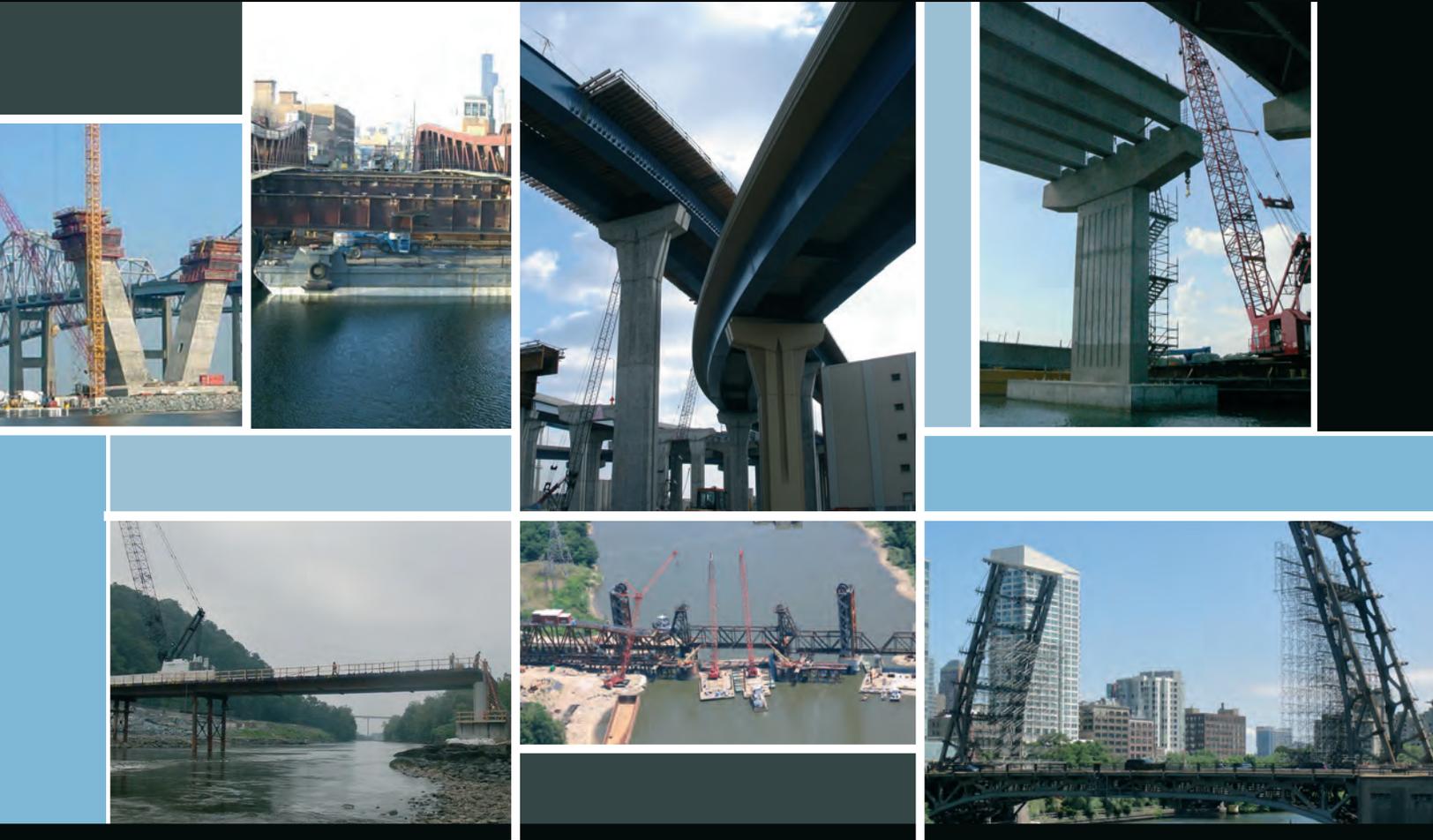


Engineering for Structural Stability in Bridge Construction

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Federal Highway Administration



NATIONAL HIGHWAY INSTITUTE

NHI Course Number 130102
Reference Manual

April 2015

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16. Abstract: This manual is intended to serve as a reference. It will provide technical information which will enable Manual users to perform the following activities: <ul style="list-style-type: none"> • Describe typical erection practices for girder bridge superstructures and recognize critical construction stages • Discuss typical practices for evaluating structural stability of girder bridge superstructures during early stages of erection and throughout bridge construction • Explain the basic concepts of stability and why it is important in bridge erection • Explain common techniques for performing advanced stability analysis along with their advantages and limitations • Describe how differing construction sequences effect superstructure stability • Be able to select appropriate loads, load combinations, and load factors for use in analyzing superstructure components during construction • Be able to analyze bridge members at various stages of erection • Develop erection plans that are safe and economical, and know what information is required and should be a part of those plans • Describe the differences between local, member and global (system) stability 			
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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				
in²	square inches	645.2	square millimeters	mm ²
ft²	square feet	0.093	square meters	m ²
yd²	square yard	0.836	square meters	m ²
ac	acres	0.405	hectares	ha
mi²	square miles	2.59	square kilometers	km ²
VOLUME				
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft³	cubic feet	0.028	cubic meters	m ³
yd³	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	pound-force	4.45	newtons	N

lbf/in²	pound-force 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				
mm²	square millimeters	0.0016	square inches	in ²
m²	square meters	10.764	square feet	ft ²
m²	square meters	1.195	square yards	yd ²
ha	hectares	2.47	acres	ac
km²	square kilometers	0.386	square miles	mi ²
VOLUME				
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m³	cubic meters	35.314	cubic feet	ft ³
m³	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/m²	candela/m ²	0.2919	foot-Lamberts	fl
FORCE and PRESSURE or STRESS				
N	newtons	0.225	pound-force	lbf
kPa	kilopascals	0.145	pound-force per inch ²	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.

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GLOSSARY OF ACRONYMS

3D	Three Dimensional
3D-FEM	Three Dimensional Finite Element Method
2D	Two Dimensional
AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
AISC	American Institute of Steel Construction
ASCE	American Society of Civil Engineers
ASD	Allowable Strength Design
ASTM	American Society for Testing and Materials
BDS	Bridge Design Specifications
DOF	Degrees of Freedom
DSW	Design Speed Wind
FEM	Finite Element Method
FHWA	Federal Highway Administration
FDOT	Florida Department of Transportation
FOS	Factor of Safety
FS	Factor of Safety
IDOT	Illinois Department of Transportation
KDOT	Kansas Department of Transportation
LF	Load Factor
LRFD	Load and Resistance Factor Design
MRI	Mean Recurrence Interval
NCHRP	National Cooperative Highway Research Program
NYSDOT	New York State Department of Transportation
OSHA	Occupational Safety and Health Administration
PCI	Precast/Prestressed Concrete Institute
PennDOT	Pennsylvania Department of Transportation
PE	Professional Engineer
SSRC	Structural Stability Research Council
TXDOT	Texas Department of Transportation
US	United States
UT	University of Texas at Austin

LIST OF SYMBOLS

Symbol	Definition
\leq	less than or equal to
\geq	greater than or equal to
$<$	less than
$>$	greater than
$=$	equal to
a	overhang beyond lift point (in)
a	center-to-center spacing of top flanges in box girder (in)
a	area (in ²)
a	the distance from the shear center to the location where the lateral brace frames into the section (in)
A_b	bolt area (in ²)
A_0	the enclosed area of the cross-section of the closed shape (in ²)
A_{bar}	area of hanger bar (in ²)
A_{bf}	area of bottom flange (in ²)
A_c	area of diagonal member (in ²)
A_d	wind directionality factor
a_{di}	cantilever overhang during lift of drop-in girder (in)
A_f	exposed projected area of girder or truss (ft ²)
A_g	gross cross-sectional area of section (in ²)
A_{g_di}	gross area of concrete of drop-in girder (in ²)
A_{g_ps}	gross area of concrete of pier segment girder (in ²)
A_h	area of horizontal member (in ²)
A_n	net area of cross-section (in ²)
A_o	the enclosed area of the cross-section of the closed shape (in ²)
a_{ps}	cantilever overhang during lift of pier segment girder (in)
A_f	area of flange (in ²)
A_{tf}	area of top flange (in ²)
A_w	area of web (in ²)
a_{wc}	ratio of two times the web area in compression to the area of the compression flange
b	the distance from the shear center to the location where the lateral brace frames into the section (in)
b	the width of a plate element (in)
b_b	width of bottom flange (in)
b_{bf}	width of bottom flange (in)
b_{b_ps}	width of bottom flange of pier segment girder (in)
b_f	width of the flange (in)
b_{fc}	width of the compression flange (in)
b_{flange}	unstiffened plate width of WT flange (in)
b_{ft}	width of tension flange (in)
b_i	plate width for use in computing J (in)
b_s	width of stiffener plate (in)

b_{stem}	unstiffened plate width of WT stem (in)
b_t	width of top flange (in)
b_{t_di}	width of top flange of drop-in girder (in)
b_{tf}	width of top flange (in)
c	centroidal distance to extreme compression fiber (in)
C	torsional stiffness (in ⁴)
C_b, C_{bL}	lift adjustment factor
C_b	moment modification factor
C_d	1.0 for single curvature and 2.0 for the brace closest to the inflection point for a beam in double curvature
CDL, C_{DL}	construction dead load
C_f	wind net force coefficient
C.G.	center of gravity
C_L	length of clamp along flange (in)
C_{LE}	effective length for bending check (in)
CLL, C_{LL}	construction live load
C_{mc}	ratio of shear buckling resistance to yield strength in hanger beam
C_R	equipment reactions
c_u	depth of neutral axis at ultimate load (in)
CW, C_w	construction wind load
C_w	warping constant (in ⁶)
C_{w14}	ratio of shear buckling resistance to yield strength in strongback
d	depth of cross-section (in)
\bar{d}	depth of girder (in)
\bar{D}	radial distance to center of gravity (in)
D	girder depth (in)
d_{br}	distance from top of beam to blocking (in)
D_c	depth of web in compression (in)
D_i	depth of web measured along incline (in)
d_o	transverse stiffener spacing in hanger beam (in)
$d_{o.w14}$	transverse stiffener spacing in strongback (in)
D_{web}	depth of web (in)
$D_{web.w14}$	depth of web of strongback (in)
DC	permanent dead load
deck_width	deck width (out-to-out) (in)
DL	dead load
DWS	design wind speed (mph)
e	eccentricity (in)
e	eccentricity of overhang force (in)
E	modulus of elasticity (ksi)
E_c	concrete modulus of elasticity (ksi)
e_{di}	eccentricity of prestressing strands in drop-in girder (in)
e_i	initial eccentricity of center of gravity (in)
e_{i_di}	initial lateral eccentricity at midspan during lift of drop-in girder (in)
e_{i_ps}	initial lateral eccentricity at midspan during lift of pier segment girder (in)

e_{\max}	analysis error
e_{ps}	eccentricity of prestressing strands in pier segment girder (in)
e_s	eccentricity of prestressing strands (in)
E_s	steel modulus of Elasticity (ksi)
E_T	tangent stiffness
e_w	eccentricity from wind overturning moment (in)
f	axial stress (ksi)
f_1	smaller stress at end of unbraced length assuming linear stress distribution (ksi)
F	eccentric force (kips)
$^{\circ}F$	degrees Fahrenheit
f_0 and f_2	respectively, smaller and larger stresses at the ends of the unbraced length (ksi)
F_b	required bracing force (kips)
f_{b_ps}	compressive stress in bottom fiber of pier segment girder from gravity load and prestress (ksi)
F_{br}	required force couple in the brace (kips)
F_{br}	required strength of brace (kips)
F_{br}	factored chord forces in cross-frame (kip) (See Figure 5-7)
F_{brc}	required strength of compression brace (kips)
F_{brt}	required strength of tension brace (kips)
f_{bu}	stress in compression flange without consideration of lateral bending (ksi)
f_{buc}	stress in compression flange without consideration of lateral bending (ksi)
f_{but}	stress in tension flange without consideration of lateral bending (ksi)
f_c	concrete strength (ksi)
F_{cr}	elastic lateral torsional buckling stress (ksi)
F_{crw}	nominal web bend-buckling resistance (ksi)
F_E	Euler buckling stress (ksi)
F_L	lateral force on flange from overhang bracket (klf)
f_{L1}	first order stress due to lateral bending (ksi)
f_{lb}	local flange bending stress (ksi)
f_{LC}	approximated second order lateral bending stress (ksi)
f_{L1c}	first order stress due to lateral bending in compression flange (ksi)
f_{Lt}	first order stress due to lateral bending in tension flange (ksi)
$F_{longvert}$	longitudinal force in bolt from vertical bending stress (kips)
$F_{longlat}$	longitudinal force in bolt from flange lateral bending stress (kips)
$F_{longtotal}$	total longitudinal force in bolt (kips)
f_{mid}	stress at middle of unbraced length (ksi)
F_{nc}	controlling nominal flexural resistance (ksi)
F_{nc1}	local buckling resistance (ksi)
F_{nc2}	lateral torsional buckling resistance (ksi)
FOS	factor of safety
f_r	modulus of rupture of concrete (ksi)
F_{QPL}	quarter point lift-off force (kips)
FS_c	factor of safety for cracking
FS_f	factor of safety for rollover

FS_{c_di}	factor of safety for cracking of drop-in girder
FS_{c_ps}	factor of safety for cracking of pier segment girder
FS_{f_di}	factor of safety for failure of drop-in girder
FS_{f_ps}	factor of safety for failure of pier segment girder
ft.	feet
ft^2	square feet
f_t	compressive stress at top fiber due to gravity load and prestress (ksi)
f_{t_di}	compressive stress in top fiber of drop-in girder from gravity load and prestress (ksi)
f_{top}	flange force from strong-axis moment (kips)
F_{trans}	transverse force in bolt from flange lateral bending stress (kips)
F_u	tensile strength of steel (ksi)
F_{ub}	tensile strength of bolt (ksi)
$F_{u.bar}$	tensile strength of hanger bar (ksi)
F_y, F_y	yield stress of steel (ksi)
$F_{y.bar}$	yield strength of hanger bar (ksi)
F_{yw14}	yield strength of strongback (ksi)
F_{yc}	yield stress of compression flange steel (ksi)
F_{yf}	specified minimum flange yield strength (ksi)
g	dead load factor
G	shear modulus (ksi)
G	gust effect factor
h	exposed wind height (in)
h	height of cross-section (in)
h	clear distance between flanges less the fillet for rolled shapes; distance between flanges for welded built-up girders or adjacent lines of fasteners (in)
H	1 minus the square of y_o divided by the square of r_o
H	height of axis of rotation about the top of girder (in)
h_b	height of cross-frame (in)
$\bar{H}_{C.G.}$	depth to center of gravity (in)
h_o	distance between flange centroids of the girder (in)
h_o	girder height (in)
h_r	distance from bottom of girder to roll axis (in)
h_{set}	exposed height for single girder being set (in)
I	moment of inertia (in^4)
I_c	connectivity index (in^4)
I_{gx}	gross moment of inertia about the x axis (in^4)
I_{gy}	gross moment of inertia about the y axis (in^4)
in^2	square inches
in^3	cubic inches
in.	inches
I_p	polar moment of inertia (in^4)
I_s	skew index
I_x	moment of inertia about the x axis (in^4)
I_{x_di}	major axis moment of inertia of drop-in girder (in^4)

I_{x_ps}	major axis moment of inertia of pier segment girder (in ⁴)
I_y	moment of inertia about the y axis (in ⁴)
I_{yc}	moment of inertia of the compression flange about an axis through the web (in ⁴)
I_{y_di}	minor axis moment of inertia of drop-in girder(in ⁴)
$I_{y,eff}, I_{eff}$	equivalent moment of inertia about y axis for shape with unsymmetric flanges (in ⁴)
I_{y_ps}	minor axis moment of inertia of pier segment girder(in ⁴)
I_{yt}	moment of inertia of the tension flange about an axis through the web (in ⁴)
J	St. Venant torsional constant (in ⁴)
k	plate buckling coefficient
k	shear buckling coefficient
k	distance from outer face of flange to web toe of fillet (in)
k, kip	kilopounds
K	effective length factor
k_b	combined stiffness of two fascia girder elastomeric bearings (k-in/rad)
K_d, K_d	wind directionality factor
k_{flange}	plate buckling coefficient for flange of WT
$K_{no\ sway}$	effective length factor for columns that cannot sway
K_{stem}	plate buckling coefficient for stem of WT
k_r	height of roll center (in)
ksi	kips per square inch
K_{sway}	effective length factor for columns that can sway
k_{w14}	shear buckling coefficient in strongback
K_z, k_z	velocity pressure exposure coefficient
K_z	effective length factor for torsion
K_{zt}, k_{zt}	topographic factor
K_y	effective length factor for buckling about y axis
K_{Θ}	sum of rotational spring constants of supports (k-in/rad)
l	beam length (in)
L	length (in)
L	span length (in)
l_1	distance between lift points (in)
\bar{L}	length along girder to center of gravity (in)
lb	pounds
l_b	length of bearing (in)
L_b	unbraced length (in)
L_b, L_{lift}	total length of a girder pick (in)
lbf	poundforce
lbf/in ²	poundforce per square inch
$L_{b.w14}$	unbraced length of strongback (in)
L_c	length of diagonal member (in)
L_{di}	length between lifting points for drop-in girder (in)
L_g	girder length (in)
L_{g_di}	length of drop-in girder (in)

L_{g_ps}	length of pier segment girder (in)
L_{id}	eccentricity of W_{id} (in)
L_{Lift}	average length from the lift points to the ends of the girder (in)
L_{lift1}	length along girder to Lift point 1 (in)
L_{lift2}	length along girder to Lift point 2 (in)
L_p	limiting unbraced length to achieve the maximum nominal flexural resistance under uniform bending (in)
$L_{p,w14}$	limiting unbraced length to achieve nominal plastic moment (in)
L_{ps}	length between lifting points for pier segment girder (in)
L_r	limiting unbraced length to achieve the onset of nominal yielding in either flange under uniform bending with consideration of compression flange residual stress effects (in)
$L_{r,w14}$	limiting unbraced length to achieve nominal moment based on inelastic lateral-torsional buckling (in)
L_s	length between supports (in)
L_S	span length at the bridge centerline (in)
L_y	length for buckling about y axis (in)
L_Z	the spacing between locations restrained from twist (in)
m	a constant taken to equal 1 for simple-span bridges and 2 for continuous-span bridges
M	the bending moment at the particular cross-section (k-in)
M	
M_A	moment at the quarter point of the unbraced length (kip-in)
M_B	moment at the middle of the unbraced length (kip-in)
M_{br}	moment in the brace (kip-in)
M_{br}	in-plane flexural resistance of brace (k-in)
$M_{br\ skew}$	moment in the brace for a skewed bridge. (kip-in)
M_C	moment at the three quarter point of the unbraced length (kip-in)
M_{cr}	elastic critical buckling moment (kip-in)
M_f	maximum moment within the span (kip-in)
M_{fl}	lateral flange moment (kip-in)
M_g	applied gravity moment (k-in)
M_{g_di}	gravity moment at midspan during lift of drop-in girder (k-in)
M_{g_ps}	gravity moment at midspan during lift of pier segment girder (k-in)
M_{gs}	buckling capacity of a girder in the system buckling mode (kip-in)
M_L	lateral moment in flange from overhang bracket (k-in)
M_{lat}	service level weak-axis moment that would cause cracking in top flange of girder (k-in)
M_{lat}	moment in flange due to lateral bending from curvature (k-in)
M_{lat}	lateral moment capacity of PPC girder (k-in)
M_{lat_di}	lateral moment capacity of drop-in girder (k-in)
M_{lat_ps}	lateral moment capacity of pier segment girder (k-in)
M_{Lu}	factored lateral moment in flange from overhang bracket (k-in)
M_{max}	maximum moment in the beam segment (kip-in)
M_n	nominal flexural resistance (k-in)

M_{nc}	nominal flexural resistance based on compression flange (k-in)
M_{ny}	nominal weak-axis bending resistance (k-in)
$M_{n.w14}$	nominal flexural resistance for strongback (k-in)
M_o	overturning moment (kip-in)
M_o	plastic moment (k-in)
M_p	plastic moment resistance (k-in)
$M_{p.w14}$	plastic moment for strongback (k-in)
M_r	resisting moment (kip-in)
M_r	required flexural strength (k-in)
M_{rx}	flexural resistance about strong axis (k-in)
M_{ry}	weak axis flexural resistance (k-in)
mph	miles per hour
M_u	elastic buckling strength (k-in)
M_u	factored maximum moment (k-in)
$M_{u.w14}$	maximum moment in strongback (k-in)
$M_{u.mc}$	maximum moment in MC hanger beam (k-in)
M_x	applied unfactored strong-axis moment (k-in)
M_{ux}	applied factored major axis bending forces (k-in)
M_{uy}	applied factored minor axis lateral bending forces (k-in)
M_w	applied wind moment at midspan (k-in)
N	lateral bending constant
n	number of braces in a span, excluding supports
N	newtons
n_{cf}	number of intermediate cross-frames in the span
N_{fb}	number of flange bolts
n_g	number of girders connected by cross-frames
N_s	number of shear planes
N_{wb}	number of web bolts
P	prestress force (kips)
P	lifting clamp load (kips)
P_{br}	required brace strength (kips)
pcf	pounds per cubic foot
P_{e_FB}	flexural buckling resistance (kips)
P_{cr}	critical buckling load (kips)
P_{di}	prestress force in drop-in girder (kips)
P_{di}	design load for MC hanger beam at drop-in segment (kips)
P_e	governing elastic critical buckling resistance (kips)
P_{e_FTB}	governing flexural-torsional buckling resistance (kips)
P_{ey}	flexural-torsional buckling resistance about y axis (kips)
P_{Ey}	flexural buckling capacity
P_{ez}	flexural-torsional buckling resistance about z axis (kips)
P_E	critical elastic buckling (Euler) load (kips)
P_{Ex}	flexural buckling capacity about the x axis (kips)
plf	pounds per linear foot
P_{max}	yield capacity of bracing (kips)
P_n	nominal buckling strength (kips)

P_o	equivalent nominal yield resistance (kips)
P_{ps}	prestress force in pier segment girder (kips)
P_r	elastic torsional buckling capacity (kips)
P_{rc}	axial compression resistance (kips)
P_r	axial tension resistance (kips)
P_{ref}	applied force (kips)
P_{rt}	axial tension resistance (kips)
$P_{r,u}, P_{ru}$	axial tension resistance for fracture (kips)
$P_{r,y}, P_{ry}$	axial tension resistance for yielding (kips)
psf	pounds per square foot
psi	pounds per square inch
P_T	elastic torsional buckling capacity (kips)
P_u	applied factored axial force (kips)
P_{uc}	axial compression force (kips)
P_{ut}	axial tension force (kips)
P_y	force resulting in full yielding of the cross-section (kips)
Q	slender element reduction factor
Q	total force effect
Q_{flange}	slender element reduction factor for WT flange
Q_i	appropriate force effect
Q_{stem}	slender element reduction factor for WT stem
Q_s	slender element reduction factor for unstiffened elements
Q_{s_flange}	slender element reduction factor for unstiffened WT flange
Q_{s_stem}	slender element reduction factor for unstiffened WT stem
q_z	velocity pressure at height z above grade (psf)
Q_z	net wind pressure (psf)
Q_{zset}	one day girder setting net pressure (psf)
q_{zset}	one day girder setting velocity pressure (psf)
q_{zsh}	shielding velocity pressure (psf)
r	radius (in)
r	radius of stability (in)
r	radius of gyration (in)
R	radius of curvature of the girder (in)
$R_{approxmax}$	maximum of the component forces determined by the approximate analysis minus the corresponding estimate from the 3 Dimensional Finite Element Analysis, and that difference divided by the estimate from the from the 3 Dimensional Finite Element Analysis.
Ratio1	web slenderness in hanger beam
Ratio1 _{w14}	web slenderness in strongback
Ratio2	limiting web slenderness in hanger beam
Ratio2 _{w14}	limiting web slenderness in strongback
R_b	web load shedding factor
R_c	service level concentrated force at each flange edge (kips)
R_h	hybrid factor
R_n	nominal resistance of the web to local web yielding (Eq 7. 17) (kips)
R_n	nominal shear resistance of bolt (kips)

r_o	polar radius of gyration (in)
R_p	reduction factor for holes
R_r	shear resistance of bolt (kips)
r_t	effective radius of gyration (in)
R_u	total force in bolt (kips)
r_x	radius of gyration with respect to the x axis (in)
r_y	radius of gyration with respect to the y axis (in)
r_z	radius of gyration for minor principal axis (in)
S, s	girder spacing (in)
S_b, S_{bx}	elastic section modulus of the bottom fiber about the x axis (in ³)
S_f	elastic section modulus of flange (in ³)
S_t, S_{tx}	elastic section modulus of the top fiber about the x axis (in ³)
S_x	elastic section modulus about the x axis (in ³)
S_{xc}	elastic section modulus about the x axis to the compression flange (in ³)
S_y	elastic section modulus about the y axis (in ³)
S_{yc}	elastic section modulus of compression flange about y axis (in ³)
S_{yt}	elastic section modulus of tension flange about y axis (in ³)
t	centroidal distance to extreme tension fiber (in)
t	thickness of a plate element (in)
T	short tons (2000 lb)
T_{bar}	tension in hanger bar (kips)
t_{bf}	thickness of the bottom flange (in)
t_f	thickness of the flange (in)
t_{fc}	thickness of the compression flange (in)
t_{flange}	thickness of WT flange plate element (in)
t_i	plate thickness used in computing J (in)
t_s	thickness of the stiffener plate (in)
t_{stem}	thickness of WT stem plate element (in)
t_T	thickness of the top flange (in)
t_{ft}	thickness of the tension flange (in)
t_{tf}	thickness of the top flange (in)
t_w, t_{web}	web thickness (in)
$t_{web.w14}$	thickness of web of strongback (in)
U	shear lag reduction factor
V	basic wind speed (mph)
V_m	wind velocity modification factor
V_n	nominal shear resistance (kips)
$V_{n.w14}$	nominal shear resistance in strongback (kips)
V_p	plastic shear force (kips)
$V_{p.w14}$	plastic shear force in strongback (kips)
V_u	maximum shear (kips)
V_{u_mc}	maximum shear in MC hanger beam (kips)
V_{u_w14}	maximum shear in strongback (kips)
w	girder weight per unit length (klf)
W	total weight of beam (kips)
W	wind load

W_1	force to girder group during partially-erected or fully-erected condition (plf)
W_b	width of elastomeric bearing (in)
W_{bracket}	total uniform load at overhang bracket (klf)
W_{di}	weight per unit length of drop-in girder (klf)
W_{DC}	uniform permanent dead load
W_{forms}	form weight at overhang bracket (klf)
W_o	weight of beam + haunch (kips)
W_{ps}	weight per unit length of pier segment girder (klf)
W_{fascia}	concrete weight at overhang bracket (klf)
W_g	weight of the girder (kips)
w_g	width of the section (in)
W_g	width of the bridge measured between fascia girders (in)
W_{id}	weight of half of slab between fascia girder and 1 st interior girder (kips)
W_{rail}	screed rail weight at overhang bracket (klf)
W_{set}	force to first girder during its setting (plf)
W_x	weight of cross-frame (kips)
W_w	line load from wind (plf)
x	additional overhang distance of deck relative to bottom flange of box girder (in)
x	approximate location of splice (in)
X	distance from the center of curvature to the axis of rotation (in)
\bar{x}	distance from the center of curvature to the center of gravity of the curved girder segment (in)
x_o	distance between the shear center and the geometric centroid measured about x axis (in)
x_{di}	distance from lift point to midspan during lift of drop-in girder (in)
x_{ps}	distance from lift point to midspan during lift of pier segment girder (in)
y	height of center of gravity of beam above roll axis (in)
Y_b, y_b	centroidal distance to bottom fiber (in)
y_{bf}	centroidal distance to bottom flange, referenced to bottom of girder (in)
y_o	distance between the shear center and the geometric centroid measured along y axis (in)
y_{b_di}	distance from centroid to bottom fiber of drop-in girder (in)
y_{b_ps}	distance from centroid to bottom fiber of pier segment girder (in)
y_r	height of the roll axis above the center of gravity of the beam (in)
y_{r_di}	height of roll axis above centroid of drop-in girder (in)
y_{r_ps}	height of roll axis above centroid of pier segment girder (in)
Y_t, y_t	centroidal distance to top fiber (in)
y_{t_di}	distance from centroid to top fiber of drop-in girder (in)
y_{t_ps}	distance from centroid to top fiber of pier segment girder (in)
y_{tf}	centroidal distance to top flange, referenced to bottom of girder (in)
y_w	centroidal distance to web, referenced to bottom of girder (in)
z	height of top of bridge deck above grade or water (ft)
Z_{max}	maximum resisting moment arm (in)
Z_o	lateral deflection of center of gravity of beam with the full dead weight applied laterally (in)

$\bar{z}_o, Z_{o.bar}$	lateral deflection of center of gravity of beam (in)
$\bar{z}'_o, Z_{o.bar.p}$	lateral deflection of center of gravity of beam including rotation effects (in)
$Z_{o.di}$	lateral deflection of center of gravity of drop-in girder with full dead weight applied laterally (in)
$Z_{o.p.di}$	lateral deflection of center of gravity of drop-in girder including rotation effects (in)
$Z_{o.p.ps}$	lateral deflection of center of gravity of pier segment including rotation effects (in)
$Z_{o.ps}$	lateral deflection of center of gravity of pier segment girder with full dead weight applied laterally (in)
$Z_{x.w14}$	plastic section modulus of strongback (in ³)
Z_x	plastic section modulus about x axis (in ³)
Z_y	plastic section modulus about y axis (in ³)
α	angle of overhang bracket relative to fascia girder web (rad)
α	angle of overhang bracket relative to horizontal line (rad)
α	superelevation or tilt angle of support (rad)
α	one-half of the degree of curvature of the curved girder segment (rad)
β	stiffness
β_b	attached brace stiffness (k-in/rad)
β_{br}	required brace stiffness (k/in)
β_{bskew}	attached brace stiffness for a skewed bridge
β_{conn}	connection stiffness
β_g	in-plane girder system stiffness(k-in/rad)
β_i	ideal stiffness (k-in)
β_L	lateral bracing stiffness
β_r	reduced weak axis flexural stiffness accounting for imperfections (k-in)
β_{sec}	web distortional stiffness (k-in/rad)
β_{skew}	attached brace stiffness for a skewed bridge
β_T	torsional bracing system stiffness (k-in/rad)
β_{Treqd}	required nodal bracing system stiffness (k-in)
β_{Treqd5}	required nodal bracing system stiffness for Case 5 (k-in)
$\bar{\beta}_T$	continuous bracing stiffness (k-in/rad-in)
β_w	warping stiffness (k-in)
γ_i	appropriate load factor (LRFD)
δ_h	horizontal deflection (in)
δ_v	vertical deflection (in)
Δ	deformation (in)
Δ_h	horizontal deformation (in)
Δ_L	chord length to lift point 2 (in)
Δ_o	initial deformation or imperfection (in)
Δ_w	lateral deflection at midspan due to wind on uncracked section (in)
ϵ	strain (in/in)
ϵ	ratio of solid area of truss members to gross area of truss

θ	skew angle measured from a line perpendicular to the tangent of the bridge centerline (deg)
θ	equilibrium roll angle (rad)
θ	lateral deflection (in)
θ	roll angle of major axis of beam with respect to vertical (rad)
θ	inclination angle of web plate in box girder, measured from vertical (rad)
$\bar{\Theta}$	angular distance to center of gravity (rad)
θ'	angular distance from lift points to center of gravity (rad)
θ_1	angle of rotation at liftoff (rad)
θ_1	skew angle at first support (deg)
θ_2	angle of rotation after liftoff (rad)
θ_2	skew angle at second support (deg)
θ_3	skew angle at third support (deg)
θ_{BrY}	= angle of rotation at reinforcing bar rupture (rad)
Θ_{di}	equilibrium roll angle of drop-in girder (rad)
θ_i	e_i/y_r = initial roll angle (rad)
Θ_{i_di}	initial roll angle of drop-in girder (rad)
$\theta_{lift\ 1}$	angular distance to lift point 1 (rad)
$\theta_{lift\ 2}$	angular distance to lift point 2 (rad)
Θ_{major}	roll angle of major axis with respect to vertical (rad)
Θ_{max}	tilt angle at which cracking begins (rad)
Θ_{max_di}	maximum roll angle for cracking of drop-in girder (rad)
$\Theta_{max.p} \ \& \ \Theta'_{max}$	tilt angle at failure (rad)
$\Theta_{max.p_di}$	maximum roll angle for failure of drop-in girder (rad)
$\Theta_{max.p_ps}$	maximum roll angle for failure of pier segment girder (rad)
Θ_{ps}	equilibrium roll angle of pier segment girder (rad)
Θ_{max_ps}	maximum roll angle for cracking of pier segment girder (rad)
Θ_{i_ps}	initial roll angle of pier segment girder (rad)
Θ_{total}	out-of-plumb total twist at end of girder (deg)
λ	eigenvalue
λ	slenderness ratio
λ_{eff}	effective slenderness ratio
λ_{limit}	limiting slenderness ratio
λ_f	slenderness ratio of the compression flange
λ_{pf}	limiting slenderness ratio for a compact flange
λ_{rf}	limiting slenderness for a noncompact flange
λ_{rw}	limiting slenderness ratio for a noncompact web
μ	Poisson's Ratio
π	pi, irrational number equal to circumference of a circle divided by its diameter
σ	bending stress (ksi)
σ_{cr}	elastic buckling stress (ksi)
σ_{diff}	warping stress in flange (ksi)
σ_p	proportional limit stress (ksi)
σ_{sum}	vertical bending stress in flange (ksi)

$\sigma_{\text{Top LT}}, \sigma_{L1}, \sigma_{L2}$	stress at the top left corner of the girder (ksi)
$\sigma_{\text{Top RT}}, \sigma_{R1}, \sigma_{R2}$	stress at the top right corner of the girder (ksi)
$\sigma_{\text{Bot LT}}$	stress at the bottom left corner of the girder (ksi)
$\sigma_{\text{Bot RT}}$	stress at the bottom right corner of the girder (ksi)
σ_{Total}	stress at the center of gravity of the girder (ksi)
σ_y	yield strength (stress)(ksi)
ϕ	resistance factor
ϕ_b	resistance factor for lateral torsional buckling
ϕ_{bk}	resistance factor for girder system buckling
ϕ_{br}	resistance factor for steel girder bracing
ϕ_f	resistance factor for flexure
ϕ_{fx}	resistance factor for flexure about x axis
ϕ_{fy}	resistance factor for flexure about y axis
ϕ_C, ϕ_c	LRFD system factor
ϕ_c	resistance factor for axial compression
ϕM_n	ultimate bending resistance (k-in)
ϕ_r	factored resistance
ϕ_u	resistance factor for axial fracture (tension)
ϕ_y	resistance factor for axial yielding (tension)

CHAPTER 1 INTRODUCTION

SECTION 1. MANUAL SCOPE

The design and construction of bridges has changed over the years to reflect the needs of the traveling public and advances in structural analysis techniques and materials developments. Highway geometrics, particularly in urban areas, often require curved girder structures, and the availability of higher strength materials has led to lighter weight members and longer spans. Current design practice also looks to minimize lateral bracing, particularly in steel bridges, where bracing connections must be detailed for fatigue.



Figure 1-1 Collapse of Steel Girder Bridge during Construction

Along with changes in bridge design, the ability of bridge contractors to set longer and heavier girders has increased with new high capacity mobile crane availability. Increased use of curved girders and composite construction, where the concrete deck contributes to lateral load resistance as well as providing lateral stability for the top flange, places added responsibility on contractors to understand bridge superstructure performance during construction.

This manual provides guidance to bridge erection engineers, resident/construction engineers, and design engineers to assist in the design and evaluation of bridge superstructures during construction. It is intended primarily to cover common steel and concrete multi-girder I-girder and tub, or box-girder, bridges. Proper assessment of bridge superstructure performance during construction is critical to ensure that member

instability or deformations do not lead to unsafe conditions or to poor geometry control that may be reflected in the finished structure.

For many bridge structures, the most critical conditions for design occur during construction, particularly when concrete decks intended to provide lateral support to the girders are not yet in place. Bridge girder collapses during construction, due to instability, have occurred in the United States as well as other countries, and several case studies are presented in Chapter 2. In some cases, excessive girder deformations have necessitated superstructure bracing modifications and even removal and re-erection of girders. The survey results reported in Section 3 indicate that 75% of the responding states have experienced such occurrences. Bridge superstructure collapses due to instability have resulted in injuries and deaths to construction personnel as reported in numerous news reports (WECT, 2009; Roads & Bridges, 2005; Yura, Ji and Windeanto, 2005; E^xponent, 2011; Star News Online, 2008), as well as deaths to the public (NTSB, 2006). Collapses have occurred during reconstruction as well as during construction of new structures (KCCI News, Des Moines, 2011).

These collapses and related girder erection problems not only affect the public due to fatalities and injuries, but such instances increase construction costs, either directly or indirectly, due to rework, extended project completion, and inconvenience to motorists. The case studies included in Chapter 2 show several failures within the United States within the 2002 – 2010 time frame, without even accounting for instances of excessive deformations or related lesser problems. Clearly, this incidence of failure is too high.

Considerable research has been conducted over the past 20 years to provide a better understanding of the performance of bridge superstructures during construction, and establish or confirm methods of analysis and member and system design. Many of these studies are cited herein or listed as additional references. Much of this work includes effects and design requirements related to structural stability, an area of structural theory that generally receives limited course time in undergraduate engineering programs.

General theory of structural stability is addressed in numerous standard structural mechanics and analysis, as well as design, textbooks. Works on classic stability theory include the *Theory of Elastic Stability* by Timoshenko and Gere, and *Buckling Strength of Metal Structures* by Bleich, which is no longer in print. *The Guide to Stability Design Criteria for Metal Structures*, R. Ziemian ed., addresses not only theoretical aspects of stability, but the development of currently utilized design equations. Various technical papers addressing member and system stability have also been published in the journals of the American Institute of Steel Construction (AISC), the American Society of Civil Engineers (ASCE) Structural Engineering, Bridge Engineering, and Structural Design and Construction journals, the American Concrete Institute, (ACI), the Precast-Prestressed Concrete Institute, (PCI), and others.

Numerous reports and research papers addressing issues related to bridge erection and stability are available. In many cases these documents may be obtained

electronically, often at no cost. Among those which provide information on bridge girder erection and stability are the following:

- National Cooperative Highway Research Program (NCHRP) Synthesis 345, Steel Bridge Erection Practices (2005) examines and discusses issues relating to steel I-girder, tub-girder and box-girder bridges that influence bridge erection. The synthesis reports results from questionnaires and interviews with bridge owners, fabricators and erectors on issues such as girder fit-up, stability, erection sequencing and owner requirements.
- NCHRP Report 725, “Guidelines for Analysis Methods and Construction Engineering of Curved and Skewed Steel Girder Bridges” (2012) evaluates the accuracy of one dimensional (line-girder based) and two dimensional (grid type) analysis procedures to predict constructed geometry and evaluate bridge constructability as opposed to three dimensional models. Recommendations relating bridge configurations and the desired analysis output data to appropriate analysis models are provided. The effects on bridge locked-in forces from cross-frame detailing using either steel dead load fit (SDLF) or total dead load fit (TDLF) are also examined and procedures to account for these in the analysis are provided. The report also provides recommendations for analysis and plan submittal content for erection engineering.
- “Stability Analysis of Single and Double Steel Girders during Construction”, Coffelt, et. al, University of Tennessee (2010) looks at torsional buckling of single and double girder systems. This work indicates that under lateral wind, the lateral girder deflection of each girder of a pair is essentially the same and thus the weak axis moment of inertia of the girder pair is twice that of the single girder. Results of the work support Yura’s buckling equation (Yura, J. and Windeanto, 2005) for twin girders, but suggest use of an unbraced length factor, C_b , of 3 for the cantilever case.
- “Stability of Precast Prestressed Concrete Bridge Girders Considering Sweep and Thermal Effects” Georgia Transportation Research Council Project No. E-20-860, Zurieck, et al., (2009), is a report prepared by Georgia Institute of Technology for the Georgia Department of Transportation. This report includes a comprehensive summary of prior research and design recommendations for stability of both reinforced and prestressed concrete girders, both under lifting conditions and after setting. Recommended provisions for design based on theoretical and load test data are presented. The thermal effects of the performance of a 100 foot long 54 inch bulb tee are also modeled and discussed. It is noted, for the case studied, that internal thermal deflections were approximately 0.5 inch; however, instability due to this deformation was not predicted by in the finite element model. The model did not, however, include initial sweep or the effects of uneven support conditions.
- “Guidance for Erection and Construction of Curved I-Girder Bridges”, Stith, et al., Report No. FHWA / TX–1010–5571–1, Center for Transportation Research, University of Texas at Austin (2010). The report summarizes the results of a research investigation on the behavior of horizontally curved girders during

construction. The role of bracing in preventing girder buckling and excessive deformations is addressed theoretically and compared to field research results. Effects of holding cranes and shoring towers are included, as well as lifting considerations. The report also discusses computer programs to analyze girders during lifting, UT Lift, and steel I-girder bridges during erection, UT Bridge, that were developed as part of the overall project.

- “Impact of Overhang Construction on Girder Design”, Yang, et. al., Report No. 0-5706-1, Center for Transportation Research, University of Texas at Austin (2010). This report presents research results and analytic studies to investigate the effects of deck overhang brackets on the performance of both concrete and steel girder bridges. System stability for concrete and steel girders is addressed as well as local web stability for steel girders, and analysis provisions are provided. A number of practical recommendations for handling field conditions are included.
- “Design Guidelines for Steel Trapezoidal Box-Girder Systems”, Helwig, et. al., Report No. FHWA / TX-0710-4307-1, Center for Transportation Research, University of Texas at Austin (2004). This report provides theoretical background and analysis and design recommendations for trapezoidal box-girders with an emphasis on bracing system requirements to control flange buckling and girder distortion in permanent and constructions stages. The use of external k-frame braces used to control relative deformation between adjacent girders during deck placement is presented along with design methodology.
- “Guidelines for Analyzing Curved and Skewed Bridges and Designing Them for Construction”, Dr. Daniel G. Linzell, et. al., The Thomas D. Larson Transportation Institute The Pennsylvania State University (2010). This report provides results of a series of parametric studies to evaluate the influence of construction methodology and sequence on stresses and deformations in both curved and skewed steel I-girder bridges. The bridge structures were numerically constructed and the influence of web-plumbness, temporary shoring location and settlement, sequencing of girder and cross-frame placement, cross-frame detailing practices (SDLF vs. FDLF), and several other variables were assessed as to their effects on the completed bridge. Models were correlated to field data, and these were then used to examine a wide range in the variables. The report provides recommendations on erection sequencing, use of cross-frames vs. solid diaphragms, web out-of-plumbness, shoring tower location and settlement limits, and cross-frame detailing practices for both curved and skewed bridges. Temperature effects during construction were found to be negligible.
- “Lateral Bracing of Long-Span Florida Bulb-Tee Girders”, Consolazio, et. al., Department of Civil and Coastal Engineering, University of Florida (2007). The stability of Florida long span bulb-tee girders during the erection process was studied. Predicted girder buckling capacities include the effects of span length, bridge skew, girder sweep, bearing pad creep, and bracing. Design equations to assess the buckling capacity including the effects of the above listed variables

were developed. Lateral wind loading effects are also discussed, as are the effects on stability of elastomeric bearing pad orientation.

During superstructure construction, it is also necessary that the engineers performing the construction stage analysis be knowledgeable in field erection practices, construction equipment, and construction load conditions — which may differ from those used in the design of the final structure.

This manual provides background information in stability concepts and provides guidance on performing stability analysis. This manual also discusses some of the advantages and limitations of bridge stability analysis methods. Recommendations are included that address construction loading conditions, and identify critical stages within the bridge superstructure construction process. Primarily girder bridges are addressed since they constitute the majority of the bridge inventory.

Chapter 2 of this manual presents several failure case histories, noting the prevalence of stability related failures, and the importance of proper temporary bracing. It is also noted that these failures are often preceded by various warning signs such as difficulty in member fit-up or visual deformations.

During bridge erection, the member support conditions, loads and stresses are affected by the erection practices such as lifting, installation of bracing, bearing conditions, temporary supports and placing sequence. Deck placing equipment, overhang brackets and staging can also have significant effects on girder stability. Chapter 3 presents information on construction practices as it relates to these considerations.

Chapters 4, 5, and 6 present the theoretical background for analyzing girder stability as well as how to integrate the theory into design practices. Both individual member stability and multi-girder stability are presented. Chapter 6 includes advanced stability analysis methods and Eigenvalue analysis.

Engineering design criteria for use in evaluating bridges during erection are presented in Chapter 7. Loading criteria and load factors for analysis are provided along with discussion of their applicability. Equations for checking member conditions during erection are included. Appendix D presents a summary of the engineering criteria for erection in a format suitable for inclusion in a specification.

Recommendations for the contents of erection engineering submittals are presented in Chapter 8. Submittal requirements are related to two categories of bridge complexity. Check lists are included to assist both the design engineer and submittal reviewer. Chapter 9 contains guidance on erection stability of several non-girder type bridges.

Appendices are provided that include the results of the erection survey sent to each state, four example problems demonstrating the application of the design methods and criteria presented in the manual, design criteria provisions provided in a specification format, and references.

SECTION 2. SURVEY OF STATE DEPARTMENTS OF TRANSPORTATION

1.2.1 Survey Description

In an effort to better understand the past experiences in bridge superstructure erection in individual states, and their requirements as they relate to superstructure erection, a survey was sent to all states that are AASHTO members. Distribution of the survey was facilitated by Technical Committee T-4, Construction, of the AASHTO Subcommittee on Bridges and Structures (SCOBS). The survey included 18 questions, starting with any past problems related to girder erection (either steel or concrete), but concentrating on erection standards, erection design criteria, and submittal and review practices utilized. Additional survey results for steel bridge fabrication and erection practices can be found in NCHRP Synthesis 345, Steel Bridge Erection Practices, and Guidelines for Design and Safe Handling of Curved I-Shaped Steel Girders (Stith, Petruzzi, et. al).

1.2.2 Summary of Responses

Survey responses were provided by 33 states, as shown in Figure 1.2. A compilation of the survey responses is included in Appendix C. A summary of the responses is given below.

1. How often have you experienced any collapses or near collapses due to lifting, handling or instability of a member during construction?
 - Though nine respondents noted this rarely or never happens, 20 states provided at least brief descriptions of some type of problem. Exterior girder rotation during concrete placement, girder lifting difficulties, and insufficient temporary bracing were the primary problems.
2. How often have you experienced member deformation / stability / alignment problems during deck placement?
 - Six respondents noted never having this problem, while 25 indicated it happens rarely too often. Problems for skewed girders and insufficient bracing were identified as well as fascia girder rotation.
3. How often have you experienced problems in the final geometry / alignment of superstructures during or at the end of erection?
 - Twenty-eight respondents indicated never or rarely experiencing such problems, while five replied occasionally or often. Fifteen respondents noted problems related to survey errors or alignment / fit-up.
4. When checking a girder for stability during handling and erection, do you require AASHTO LRFD be used?
 - Eighteen states replied yes. Of those replying no, eight use a variety of AASHTO 17th edition, AASHTO Guide Specifications for Temporary Works, or State requirements.
5. Do you require an erection procedure to be submitted by the bridge erector?

- Of the 32 respondents, 27 answered yes.
6. If the answer to question 5 is yes, are they required for all bridges?
 - Of the 26 respondents, 16 who answered yes to question 5 replied yes.
 7. If the answer to question 6 is no, is there a size, span length, geometry feature or other threshold consideration that triggers the requirement?
 - Fifteen respondents provided criteria. Triggering mechanisms included: erection over traffic, railroads, steel plate girders, long spans / curved / skewed, or as required by design engineer.
 8. Do you have requirements for the erection procedure contents and format?
 - Of the 32 respondents, 19 replied yes, 14 replied no.
 9. Do you specify criteria to erectors for design wind load considerations during erection?
 - Only six respondents specify criteria. California has a table of wind pressures, varying with height, while two reference the AASHTO Guide Specification for Bridge Temporary Works.
 10. Do you specify criteria to erectors for maximum lateral deflection of girders due to wind load?
 - One state, Pennsylvania, provides criteria.
 11. Do you specify a minimum safety factor with regard to global stability of a partially erected or demolished structure for global structural stability?
 - None of the respondents specify this.
 12. Do you provide guidance or design criteria for the strength and stability checks for cantilever girder sections during lifting and placement?
 - Two states, Kansas and Virginia, replied yes.
 13. Do you require a bridge demolition procedure to be submitted for bridge removal?
 - Twenty-seven respondents do require a demolition plan; only six do not.
 14. If the answer to 13 is yes, is there a size, span length, geometry feature or other threshold consideration that triggers the requirement?
 - Nineteen respondents indicated requirements. The most common triggering considerations are bridges over, or adjacent to, live traffic or railroads, structure length, or it is done on a case specific basis.
 15. Do you have requirements as to the qualifications of those who prepare the erection plan?
 - Twenty-five respondents require the person to be a registered engineer.
 16. Do you have requirements as to the qualifications of those who prepare the demolition plans?

- Twenty respondents have requirements. Fifteen require the persons to be a registered engineer, while the other five may require a P.E. based on bridge complexity.

17. Do you have requirements as to the level / degree of engineering analysis that is performed by the erector?

- Ten respondents indicated some type of requirements; however, these appear to be of a more general nature.

18. Who reviews erector supplied erection plans?

- Eight respondents perform reviews within the construction or resident engineering group, while twenty two perform reviews within the design group or by the Engineer of Record. Pennsylvania uses consultants for larger projects. Ohio uses a peer review system with no department review. One respondent did not indicate any reviews. Kansas uses different criteria for different erection categories.

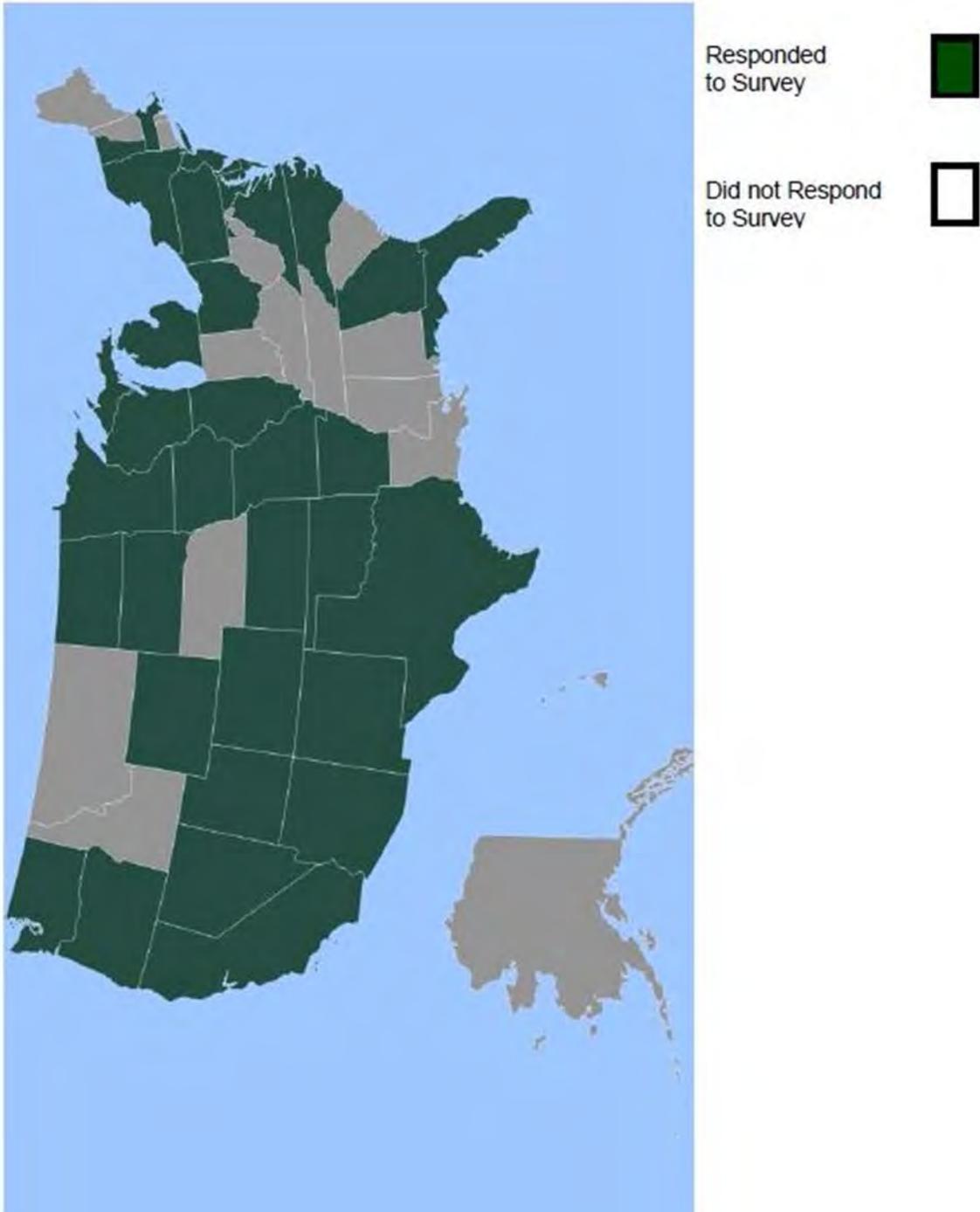


Figure 1-2 States Responding to Survey

In reviewing the overall responses, several conclusions may be drawn:

- Though each of the respondents generally considers the occurrence of collapse, near-miss, or problems with stability and alignment to be infrequent, taken together, such incidents appear to be rather common.
- The requirement to perform erection design checks to the AASHTO LRFD is referenced by the majority of respondents.
- Most respondents do require an erection and demolition procedure, though the requirement may be dependent on the bridge complexity, or adjacent traffic.
- Approximately one half of the respondents provide requirements for the erection procedure.
- Detailed design requirements for erection design provided by the owner are rare.
- Virtually all respondents require erection or demolition procedures to be prepared by a registered professional engineer.
- Review of erection procedures is most often performed in-house by construction or design staff. When not performed in-house, the design engineer-of-record performs the review.

It is evident that problems do occur as a result of bridge erection operations and that, at least from a procedure and submittal process, the respondents require engineering for the erection or demolition work. Guidance for that engineering is considered to be provided by AASHTO along with the engineers' experience.

Survey responses show that owner requirements on girder deflections, global stability safety factors, strength, and stability of cantilever sections and wind load criteria during erection are rarely provided. Design criteria and the related discussion contained in Chapter 7 are provided so that consistent criteria are available to guide design in these areas.

CHAPTER 2

CONSTRUCTION FAILURE CASE STUDIES

SECTION 1. INTRODUCTION

The failure of the Quebec Bridge over the St. Lawrence River in 1907 demonstrates that bridge failures during construction are not a recent phenomenon. Indeed, the collapse of bridges both during construction and while in service have been a prime motivator in the development of the design and construction specifications that we use today. However, despite advances in engineering knowledge and specifications, failures during construction continue to occur as the case studies that follow demonstrate. Failures not only include cases of total or partial collapse, but also include bridge deflection or local member buckling that require remedial measures. The failures discussed illustrate the importance of adequately addressing superstructure stability through both analysis and attention to field construction practices and erection details, noting that warning signs of potential problems are often present.

SECTION 2. ROUTE 17 (FUTURE I-86), NEW YORK (2010)

The structure under construction consisted of a two-span continuous parallel steel plate girder bridge that was planned to carry I-86 in New York State. The girders were each fabricated in three segments, with a center section spanning over the pier, and two end pieces, which originated at field splices and ended at the abutments. The girders were erected on temporary steel shoring frames located on each side of the pier. Each frame consisted of two steel towers and a single steel W33 shoring beam spanning between the towers in a simple span configuration. The W33 beam rested on a W12 transfer beam at the top of each tower. Each frame was founded on two timber distribution mats, one at the base of each tower. See Figure 2-1.

The center five girder segments over the pier were erected on a Thursday during the summer and reportedly had all cross-frames and bottom flange diagonals installed, but with partially bolted connections. Due to a poor weather forecast on Friday, no additional work was planned.

On Saturday evening, the engineer in charge was notified of a loud bang and arrived at the site to find the bridge girders sitting precariously on the pier cap and partially collapsed shoring, as shown in Figure 2-1. The weather from the time of erection to the collapse was not unusual, with winds averaging 7 to 10 mph and occasional gusts as high as 25 mph.

A subsequent forensic analysis by the owner uncovered multiple issues with the temporary shoring. Deficiencies were identified in the capacity of the W33 shoring beam with respect to lateral torsional buckling and for local web crippling and yielding where the bridge girders rested on the top flange of the W33. Additionally, the investigation

determined that the W12 transfer beam at the top of each tower had inadequate flexural strength, and the pins used to splice the shoring tower segments together were significantly overstressed. A review of survey data from before and after the failure determined that one of the distribution mats supporting one of the temporary shoring towers had settled 3½ inches between the time of erection and the forensic investigation.



Figure 2-1 Route 17 Collapsed Shoring and Girders

SECTION 3. MARCY BRIDGE, NEW YORK (2002)

The Marcy Bridge was designed to span approximately 170 feet across an expressway in New York State and was to serve as a pedestrian bridge. The bridge was designed as a composite structure incorporating a steel tub-girder with a 14-foot-wide, cast-in-place concrete deck. The bridge cross-section including the deck formwork is shown in Figure 2-3(Corr, et. al 2004). The structure collapsed during the concrete deck placement in the fall of 2002, at a time when the concrete placement had reached approximately midspan. The collapse resulted in one fatality and nine serious injuries. A photograph of the bridge after the collapse is shown in Figure 2-4.

The design consisted of a single trapezoidal steel tub-girder with a composite reinforced concrete deck. The bridge was straight in plan. A subsequent forensic investigation determined that the failure mode was lateral-torsional buckling of the steel tub-girder in which the entire girder cross-section participated, as opposed to the typical failure mode of the compression flanges only as shown in Figure 2-4. Such bridges are particularly

susceptible to this type of failure because they are very flexible in a twisting mode prior to deck hardening. A plan view of the bridge, showing the locations of the struts and K braces is shown in Figure 2-2. The spacing of the internal bracing was sufficient to prevent lateral flange buckling (Yura, J.A. and Widiyanto, 2005)

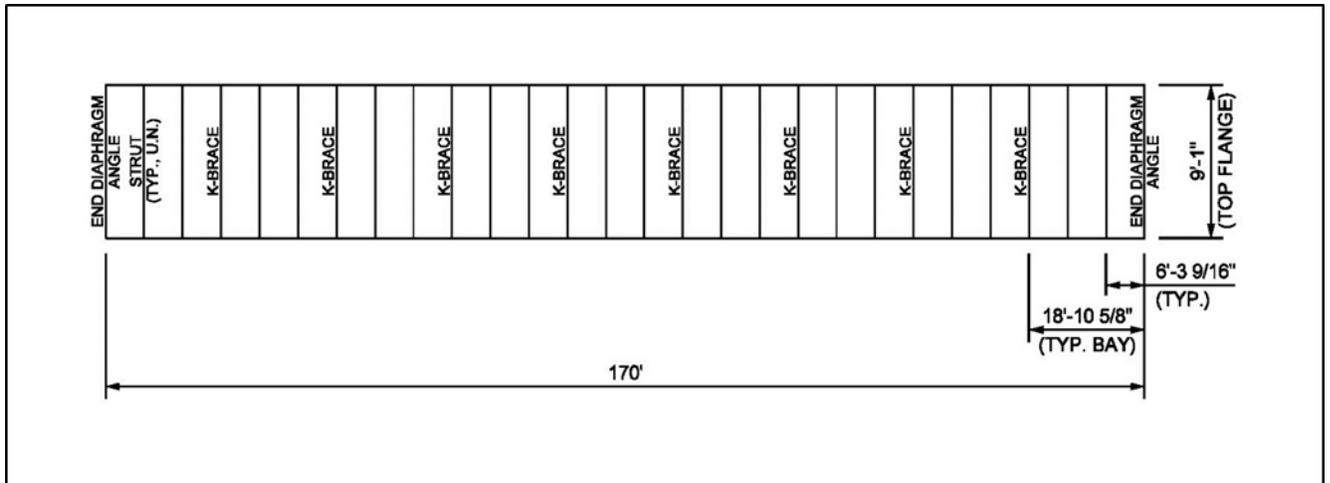


Figure 2-2 Marcy Bridge Plan

Since the shear center of the tub-girder is located below the tub-girder cross-section, construction loads can generate torsional loads in the open cross-section. The ratio of the moment of inertia about the vertical, y, axis over the moment of inertia about the horizontal, x, axis for the girder was 1.75, and hence lateral torsional buckling was not considered as a mode of failure. However, the Marcy bridge tub-girder behaved like a double I-girder bridge due to the absence of top flange bracing. The cross-frames did not provide out-of-plane rigidity, allowing each girder to deflect individually, and the entire tub girder cross-section failed in global lateral-torsional buckling.

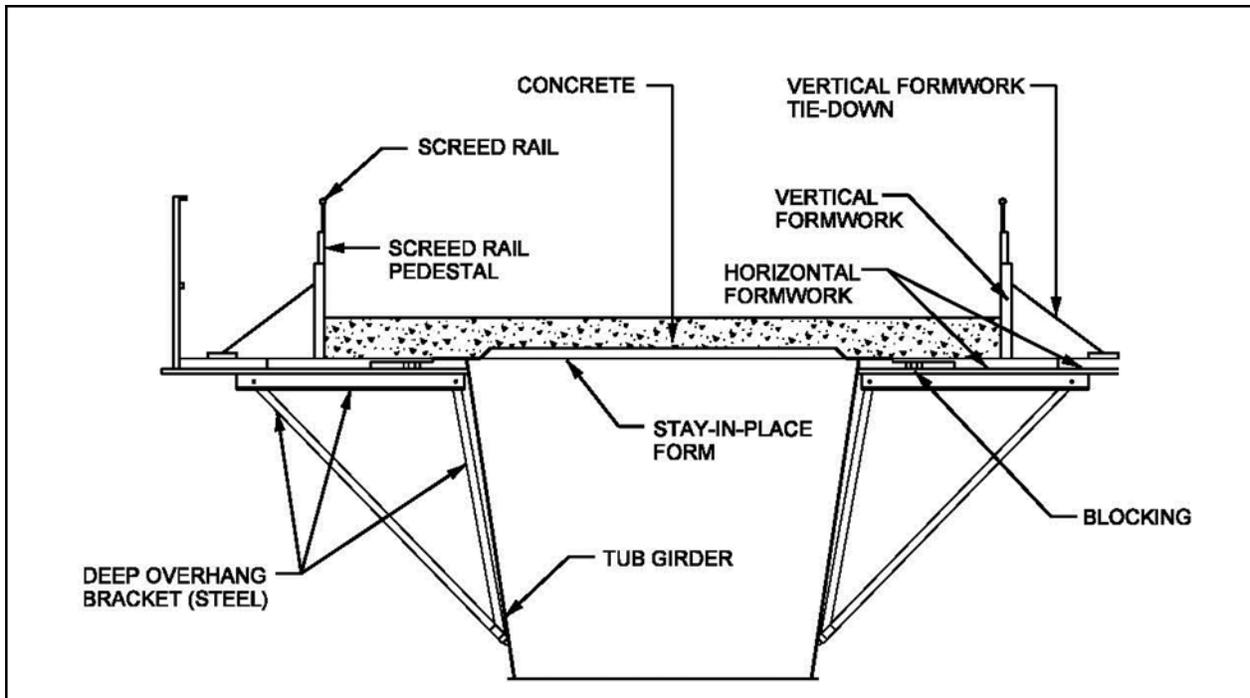


Figure 2-3 Marcy Bridge Cross-section – Including Formwork

Several things could have been done to avoid this collapse. A structural analysis of the bridge behavior during concrete placement would have indicated that temporary falsework, such as shoring towers, was required to ensure stability until the concrete attained sufficient strength. Optionally, additional top flange lateral bracing could have been added to create a quasi-closed cross-section and increase the global lateral-torsional stiffness to prevent buckling. The 1978 US Steel publication titled, *Steel/Concrete Composite Box-Girder Bridges, A Construction Manual*, recommends a full length internal top flange lateral system in spans over 150 feet. A third option could have utilized stay-in-place formwork engineered to act as top flange lateral bracing. The bridge did utilize stay-in-place formwork; however, the formwork and its connection to the top flanges were not engineered to provide the lateral strength and stiffness needed to prevent the girder from buckling.



Figure 2-4 Marcy Bridge after Collapse

One outcome of the Marcy Bridge collapse was a requirement issued by the New York Department of Transportation in 2003 that all tub girders include a full length lateral bracing system. Similar provisions were also subsequently included in the AASHTO Specifications (Corr, D.J. et. al 2009).

SECTION 4. RED MOUNTAIN FREEWAY, ARIZONA (2007)

The Red Mountain Freeway collapse occurred during the construction of the Red Mountain Freeway near Power Road in Mesa, Arizona. Two bridges were under construction, an eight-span structure carrying westbound traffic and a nine-span structure carrying eastbound traffic. The collapse occurred in Span 5 of the westbound structure, which consisted of 5 foot 3 inch deep AASHTO Type V modified prestressed concrete girders in a 114-foot simple span configuration (ENR, 2007).

During July 2007, the Span 5 girders of the westbound structure had been erected and were seated on their elastomeric bearings. No temporary cross bracing had been installed to provide lateral stability. On August 9, 2007, nine of the eleven precast, prestressed concrete girders in Span 5 collapsed prior to placement of the cast-in-place concrete deck and diaphragms.

A forensic investigation (CTL Group, 2007) initiated by the Owner concluded that the likely cause of the failure was lateral instability of the one of the exterior girders, Girder A5-9. This instability led to a rolling and/or sliding failure of the girder, which triggered a progressive collapse of the eight adjacent girders. The report stated that the instability of the girder was likely due to simultaneous bearing eccentricity, bowing of the girder (sweep), and a non-level slope of the bearing surface. Additionally, thermal creep, concrete creep, wind, and construction loads between girder erection and the collapse, and the failure to fully remove the stainless steel bearing plate protection material compounded the initial eccentricity, sweep, and bearing surface slopes in both the transverse and longitudinal directions, increasing the likelihood of the collapse.



Figure 2-5 Red Mountain Freeway Showing Collapsed Concrete Girders on Ground

In their article “Rollover stability of precast, prestressed concrete bridge girders with flexible bearings,” Hurff and Kahn (Hurff, H.B. and Kahn, L.F., 2012) also conclude that the girder failure was due to rollover and not buckling. The CTL report provided recommendations to improve resistance of girders to rollover to include:

- Assuring all bearing surfaces are clean
- Assuring the girders are centered on the bearings
- Installing temporary bracing for lateral stability at the girder ends as soon as they are erected.

SECTION 5. EAST BOUND I-80/94 TO NORTH BOUND IL 394, RAMP J

2.5.1 General Description

Reconstruction of the I-80/94 / IL 394 interchange included widening and reconfiguration of the interchange to provide improved traffic flow. The improved geometrics required construction of two elevated curved ramp structures, designated Ramps G and J. Due to construction scheduling and contract packages, the two ramps were erected by different contractors. During steel erection for the three-span Unit 2 of Ramp J, a collapse occurred.

Ramp J incorporated a cast-in-place concrete deck supported on six concentrically curved, welded steel plate girders. The ramp had two, three span continuous units, and one four-span continuous unit separated by expansion joints, with an overall centerline length of 2,344 feet - 8 inches. The arrangement of the ramp and the unit designations are shown in Figure 2.6.

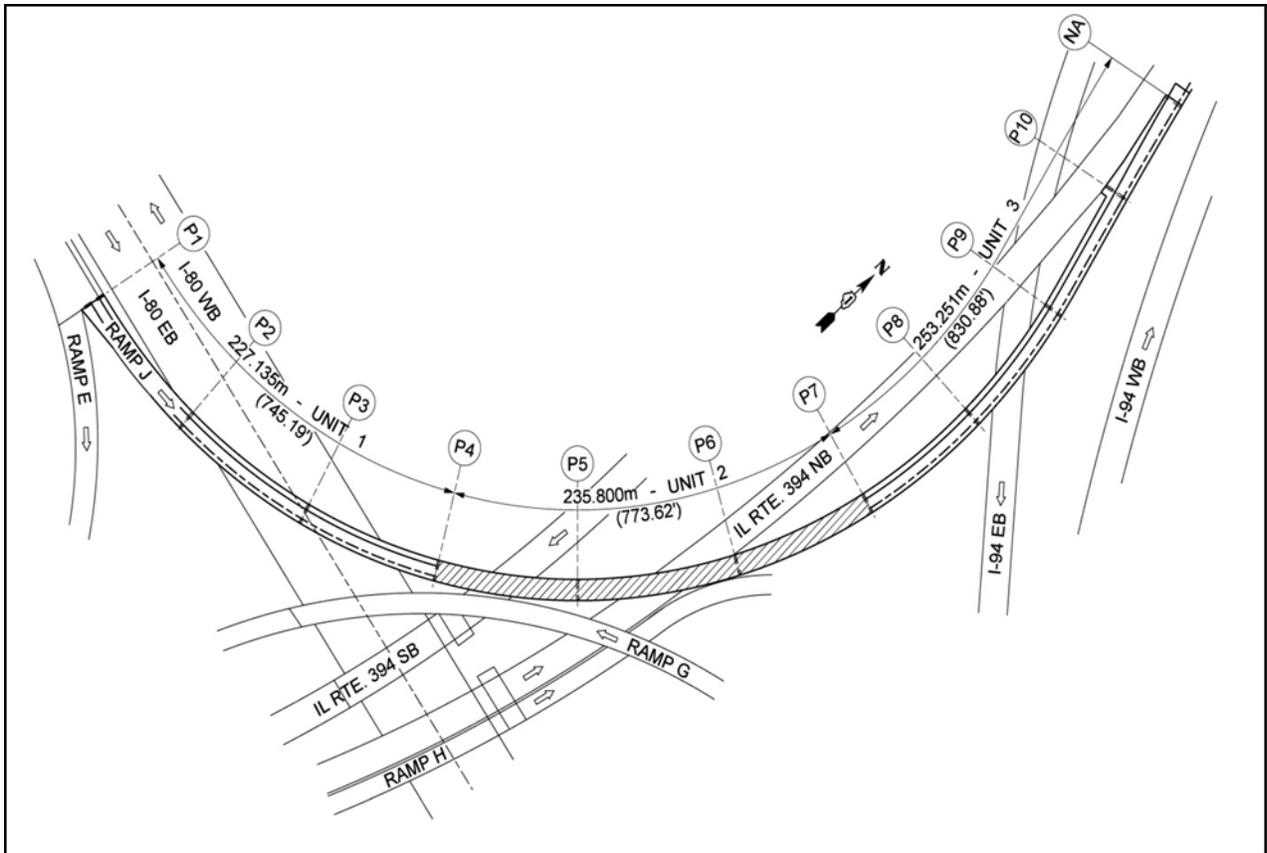


Figure 2-6 Ramp J Plan Showing Units 1 Through 3

The I-shape girders were spaced radially at 5 feet- $5\frac{3}{8}$ inches center-to-center, with bolted radial cross frames spaced at approximately 15 feet. See Figure 2-7 and Figure 2-8 for the girder layout and splice locations for Units 1 and 2 respectively, and Figure 2-9 for the typical cross-frame details. The girders' webs were 102 inches deep by $\frac{3}{4}$ inches thick, and the flange sizes varied by location. Flange width to thickness ratios varied between (approximately) 15 and 18 inches in positive moment regions. The three outer girders had larger flanges than the three inside girders. The girders were curved to approximately 990 feet radius and the ramp had a superelevation of approximately 6 percent.

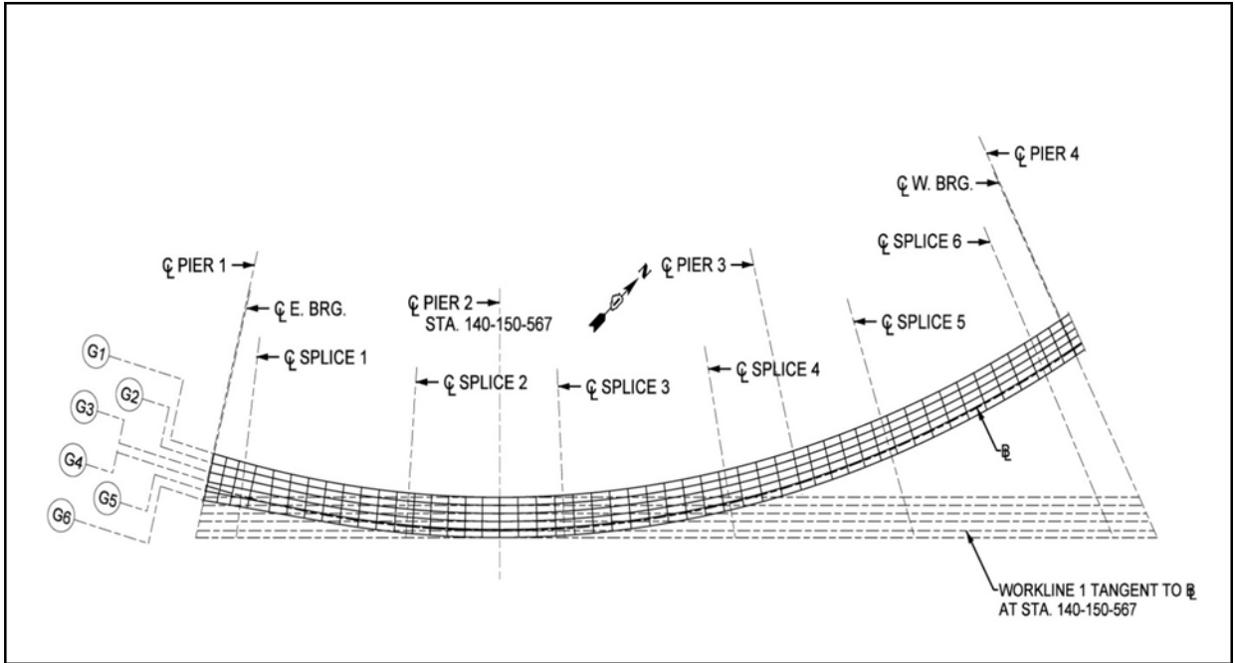


Figure 2-7 Girder Layout Plan – Unit 1 on Which Girders Misaligned

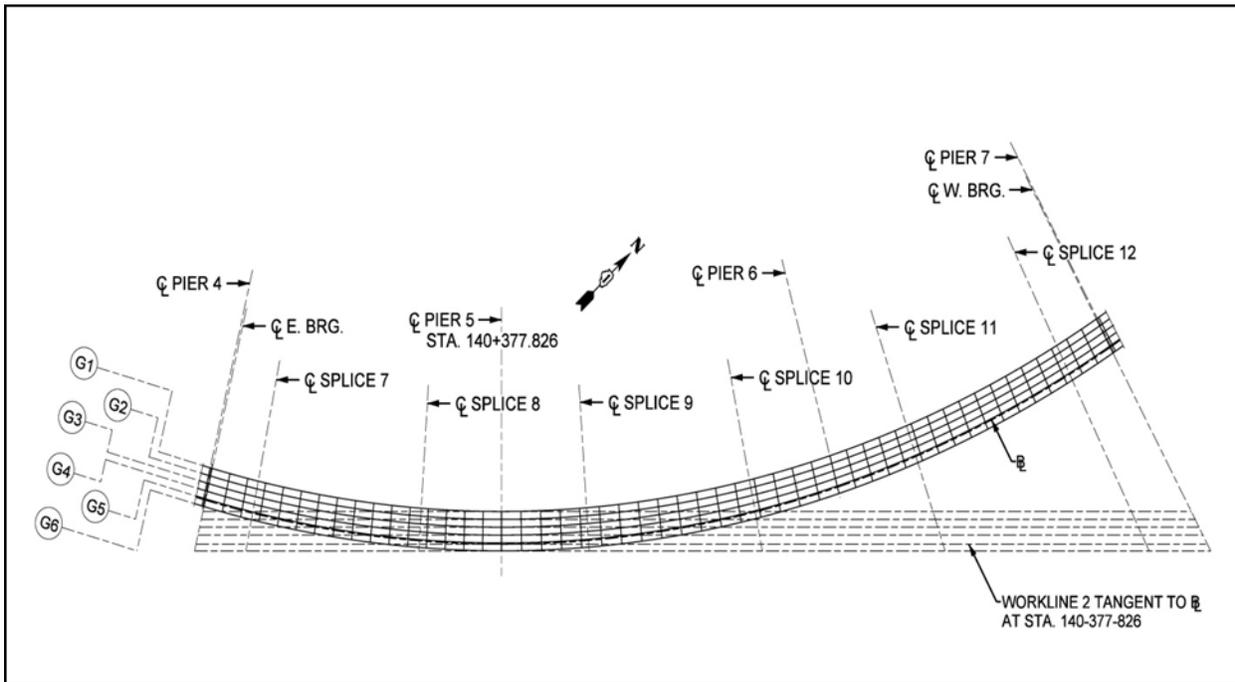


Figure 2-8 Girder Layout Plan – Unit 2. Erection Was from Pier 7.

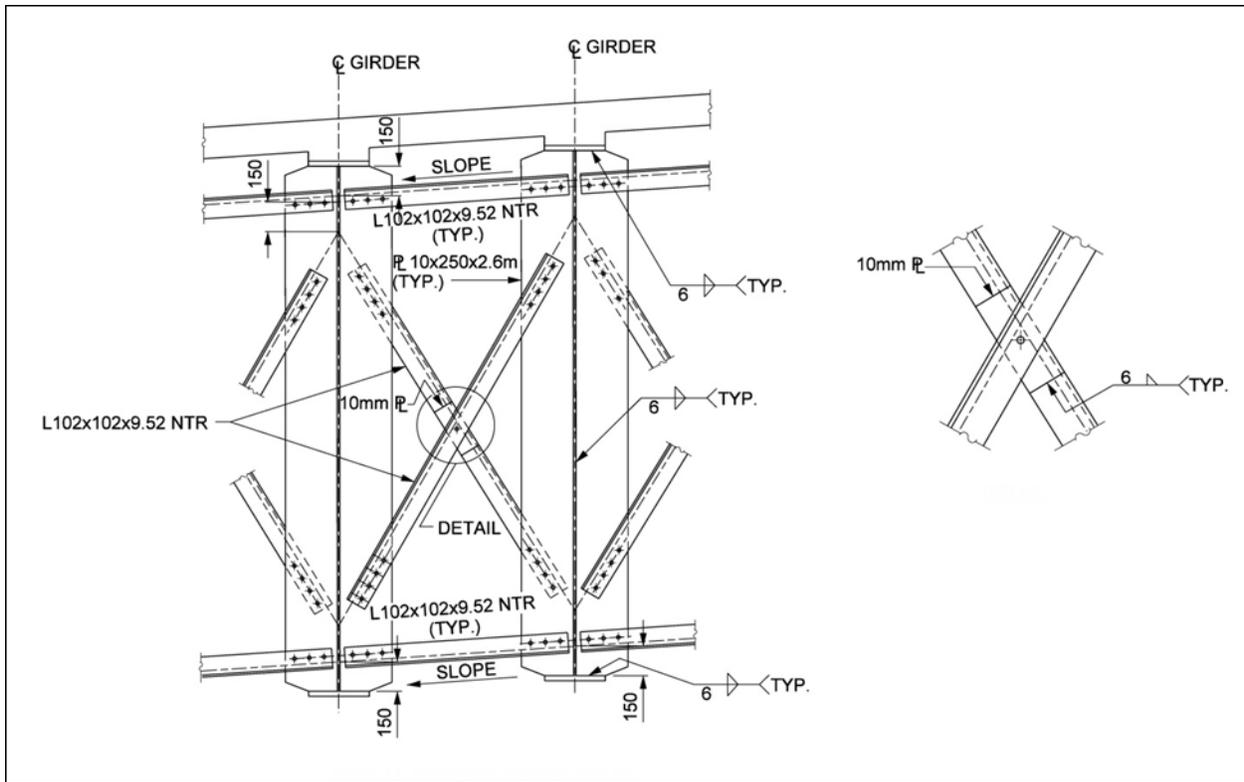


Figure 2-9 Typical Cross-frame Details (from Design Drawings)

Drawing notes indicated the bridge was designed to AASHTO Standard Specifications for Highway Bridges 2002, and the 1993 Guide Specifications for Horizontally Curved Highway Bridges (with revisions through 1997). Structural steel was AASHTO M270M, Grade 345 ($F_y = 50$ ksi). Cross-frame connections stipulated M22 ($7/8$ -in. diameter) high strength bolts in M27 ($1\frac{1}{16}$ in. diameter) oversize holes. The drawings also contained the following Construction Note:

“The stability of the partially erected curved steel components is the Contractor’s responsibility during all phases of construction. Temporary shoring towers may be required at locations determined by the Steel Erector”.

A further, Steel Erection Note stated:

“Contractor to submit for Engineer’s review, detailed erection plans and procedures taking into account traffic maintenance including but not limited to staging positions, falsework locations and sequence of girder erection and provisions for stability of girders and bearings during erection.”

A further note required bearings to be blocked during erection.

Construction of the ramp began with Unit 1, for which steel erection was mostly completed at the time Unit 2 erection began, although end cross-frames were not completed and interior cross-frame connection bolts were not tightened. The erector

planned to adjust the steel alignment and complete cross-frame bolt tightening prior to forming and casting the deck. Unit 1 shoring was left in place until all steel work was erected.

Unit 2 erection started with Span 6 from the expansion joint at Pier 7 and progressed toward Unit 1, setting the girders from the outside of the curve inward. See Figure 2-8 for the girder layout in Unit 2. Span 6 girders had two field bolted splices, and the splice nearest Pier 7 was bolted on the ground before the combined segments were lifted. A shoring tower was placed near midspan to support the two outside girders (#5 and #6). Some of the lateral bracing was bolted finger tight. The next girder segments, extending over Pier 6, were erected and the splices fully bolted.

Once all six rows of girders were in place in Span 6, the shoring tower was to be relocated. A crane applied uplift to a girder at the tower to reduce pressure and allow the shoring tower jacks to be loosened. After the jacks were released, workers began disassembling the tower, but then the Span 6 girders moved outward, became unstable, and collapsed, resulting in the death of one ironworker and injuries to three others. Figure 2-10 is a photograph taken after the collapse, looking toward Pier 6. Due to the position of Unit 2 in the interchange, the steelwork did not land on the traveled roadway, resulting in minimal traffic disruption.



Figure 2-10 Post-Collapse Photograph - Unit 2 Looking toward Pier 6

A subsequent survey of the erected steel for Unit 1 of the ramp, Figure 2-7, performed to verify alignment and elevations preparatory to placing the concrete deck showed that the in-place geometry did not match the design documents. Both the girder alignment and elevations varied from requirements. Vertical deflections were most severe in the outside girder of Span 2 of Unit 1, where deflections exceeded design values by several inches. Horizontal deflections of several inches outward were also measured in Span 2 of Unit 1. These deflections were severe enough that the Illinois Department of Transportation mandated the contractor install additional shoring until methods to realign or even re-erect the steel work were investigated and applied.

2.5.2 Ramifications of Collapse

At the time of the collapse, the erection of Unit 3 had not started. Prior to starting the Unit 3 erection, a detailed erection plan and construction procedure was developed to address both girder stability and geometry control based on a piece-by-piece erection analysis. Shoring towers were placed for temporary support and were designed with sufficient capacity to allow jacking of the girder needed for adjusting elevation during erection.

The Span 6 in Unit 2 steel was replaced with new girders, and the existing pier cap concrete was repaired. A plan for removal of the collapsed steel and erection of the new girder was also developed for this unit. The Unit 1 steel was realigned prior to deck placement.

SECTION 6. I-70 BRIDGE, COLORADO (2004)

Two steel plate girders, each 194 feet long, composed of a 154-foot span and 50-foot cantilever over the center pier were scheduled to be erected to support the widening of an existing structure during a nighttime closure of the highway below. The contractor encountered problems in setting the first girder. Reportedly, the contractor used improper tools and set the first piece backwards. These problems caused delays and only one of the two girders was erected during the nighttime closure (NTSB, 2006). Horizontal temporary steel angle lateral braces from the erected single girder to the existing bridge deck were planned to be installed. Bolts were to be used to connect the angle braces to the steel girder and expansion bolts were to be used to connect the braces to the bridge deck. The NTSB Highway Accident Report indicates that the Erector's Safety Officer, with no training or certification in engineering, developed the bracing plan for the single girder utilizing a hand drawn sketch.



Figure 2-11 I-70 Bridge Showing Collapsed Girder

A cold and windy forecast postponed the erection of the second and third girder needed for the widening, as well as the cross-frames, for more than three days. At that time, the single erected girder collapsed, rolling onto its side and causing three deaths. Figures 2-11 and 2-12, from the NTSB report, show the collapsed girder.

The subsequent forensic investigation indicated several problems with the girder installation. The girder had been installed over four degrees out of plumb at the abutment, leaning toward the existing bridge, and over two degrees out of plumb at the pier. The NTSB determined that failure of the braces occurred at the existing bridge deck, when the lateral force from the girder's distortion placed loads on the expansion anchors, separating the expansion bolts from the existing bridge deck. The expansion bolts were not installed in accordance with the manufacturer's instructions. The 0.75-inch diameter bolts were set in oversized, 0.90 inch diameter holes, and four of the five bolts were not embedded into the concrete the minimum specified depth of 3.25 inches. Only one bolt was embedded more than 2.5 inches deep, and one bolt was embedded

only 1.25 inches deep. Additionally, the five lateral braces were to be bolted flush with the existing bridge deck and none were actually flush with the deck.

A finite element analysis of the girder collapse was conducted by the FHWA Turner-Fairbank Highway Research Center (Wright, Kogler, et.al, 2005) to assist in determining the likely or possible sequence of events between the girder erection and collapse. Although the wind loading had minimal effect on the girder, cyclical forces in the braces due to lateral vibrations and wind loads were identified as primary factors in weakening the incorrectly installed expansion bolts over time. The second angle brace (from the south) of the five braces was critical in providing stability for the girder. The analysis indicated that the removal of this brace caused immediate instability for the out-of-plumb girder.

As part of the response to this accident, the State DOT issued post-accident revisions to its Standard Specifications, including:

1. An erection plan must be developed. It must be reviewed and approved by the Contractor's Professional Engineer at least 4 weeks prior to the erection of a structural steel member.
2. Details for falsework, bracing, or other connections should be shown on the erection plan.
3. A pre-erection conference must be held two weeks prior to erection, attended by the Contractor's Professional Engineer.
4. Written approval of each phase of installation must be provided by the Contractor's Professional Engineer before vehicles or pedestrians are allowed on or below the structure.
5. Daily inspections of erected steel are required by the Contractor until completion of the deck concrete.

This accident also demonstrates the need to have contingency plans in place at each stage of erection.



Figure 2-12 Girder Collapse - Note Girder Deflected Down Into Lane

SECTION 7. SR69 OVER THE TENNESSEE RIVER (1995)

The SR69 Bridge over the Tennessee River consisted of 16 spans totaling 2893 feet – 9 inches in length. The 13 approach spans consisted of prestressed precast bulb tee girders. The three river-crossing spans, 14, 15, and 16 were: 340 feet, 525 feet, and 340 feet – 6 inches long, respectively. Each was supported by three continuous steel girders spaced 20 feet - 10 inches on center transversely, with intermediate W24 steel stringers centered between the girders. The girders were braced by transverse cross-frames spaced at 24 to 25 feet on center. A cross-section of the river spans is shown in Figure 2-13.

In May of 1995, erection work was progressing on the main river span. Two girders were erected and cross-frames were installed and bolted in Spans 14 and 15. The girders were cantilevered over Pier 15 to field splice locations in Span 16. As the final girder was being erected in Span 15, the contractor had difficulty in the girder fit-up (Wiss, Janney, Elstner Assoc., Inc., 1995). The fit-up problem was related to a sweep in the girder that caused it to miss its bearing at the pier and a bolt hole pattern misalignment in matching pieces. As the erection crew made efforts to correct this sweep through come-a-longs and over-booming of cranes, they buckled a cross-frame that was in place between this last girder and the adjacent girder. When they removed the damaged cross-frame in order to replace it, the last girder erected was left with approximately a 200 foot unbraced length, and the bridge collapsed shortly thereafter. Figure 2-14 shows the progression of erection, looking downstream, at the time of the collapse. The collapse resulted in one fatality and several injuries.

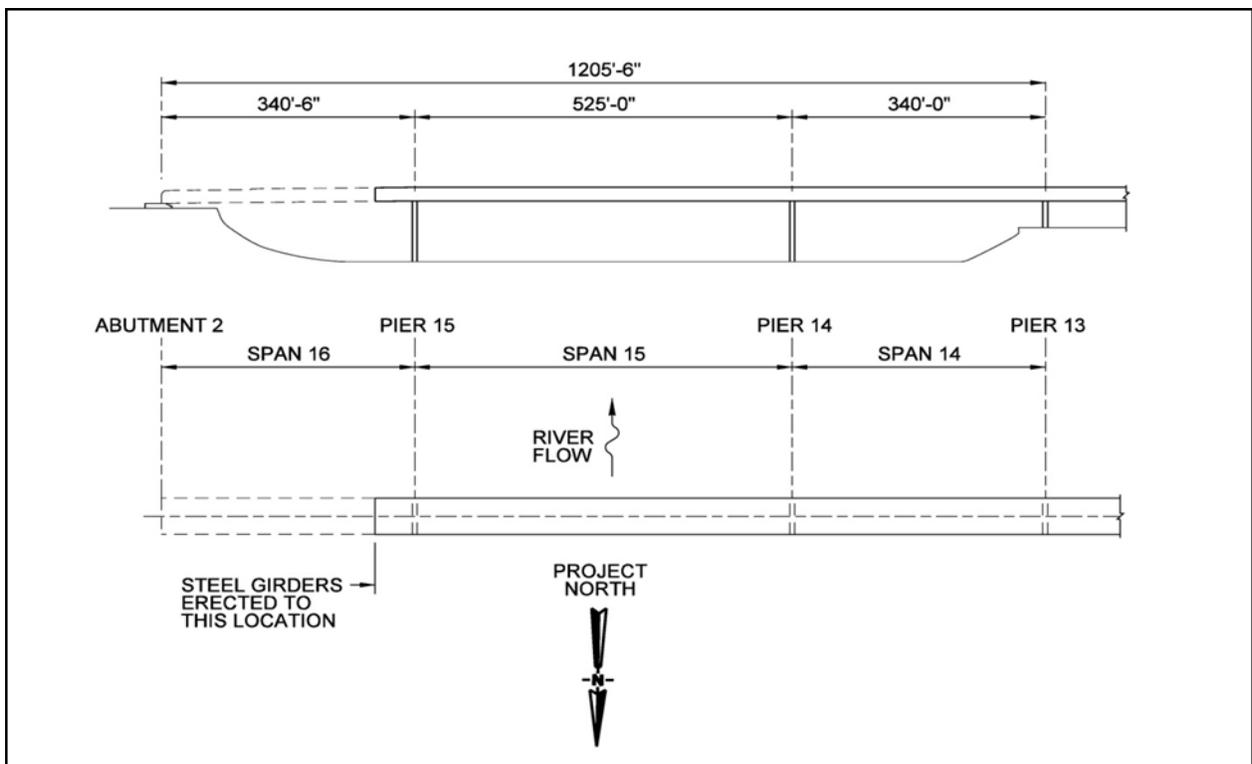


Figure 2-14 State Route 69 Bridge Erection Progress at Collapse

Figure 2-15 is a photograph from the Wiss, Janney, Elstner Associates, Inc. report, reportedly taken within hours of the collapse. The cross-frame that had been removed can be seen in the right side of the photograph still attached to the crane's whip line. A subsequent forensic analysis attributed the collapse to lateral torsional buckling of the girder because of the large unbraced length of the compression flange with the cross-frame removed.

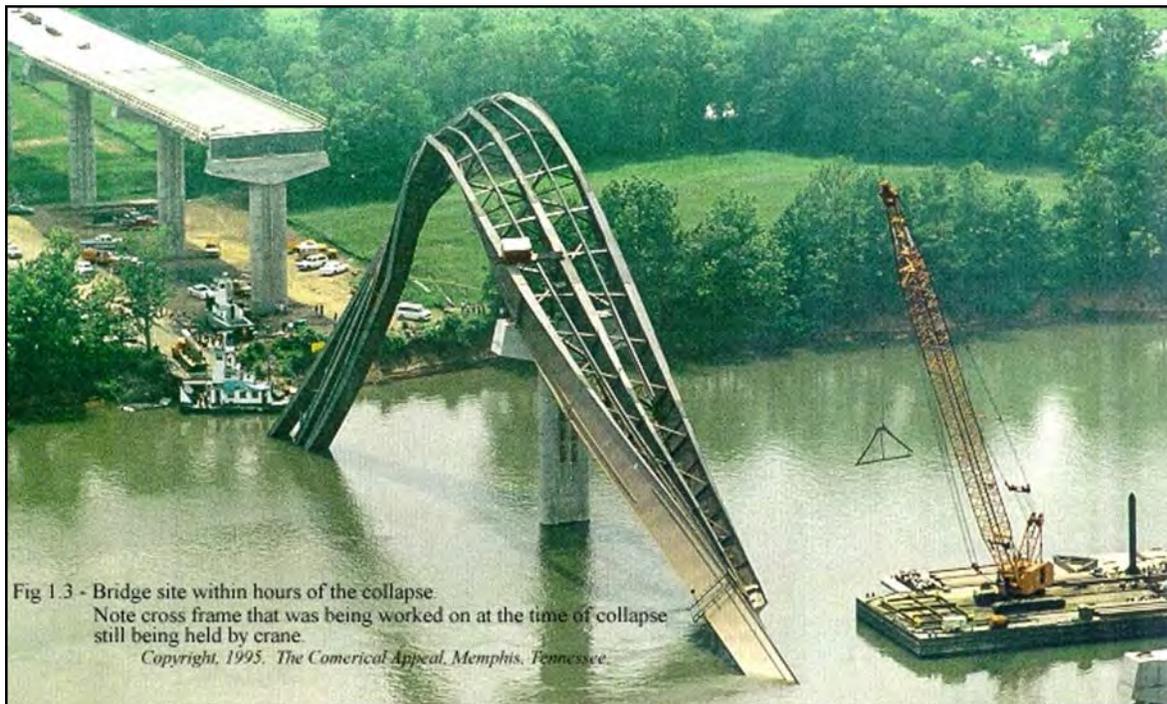


Fig 1.3 - Bridge site within hours of the collapse.
Note cross frame that was being worked on at the time of collapse
still being held by crane
Copyright, 1995. The Commercial Appeal, Memphis, Tennessee.

Figure 2-15 SR69-Bridge Site Showing Collapsed Spans

SECTION 8. SOUVENIR BOULEVARD BRIDGE, QUEBEC (2000)

The Souvenir Boulevard Bridge, which crosses Highway 15 in Laval, Quebec, was under construction in June 2000 when a partial collapse of the structure occurred. The 4-span, 472 foot long structure consisted of AASHTO Type V prestressed concrete girders and an 8-inch thick concrete deck. Per industry standard in Quebec, the structure was designed to have only one expansion joint to increase durability and reduce maintenance. To accommodate the single expansion joint, fiber reinforced elastomeric bearings were provided at the point of fixity at the west abutment (Line 1), and pot bearings designed to accommodate longitudinal movement were provided at the piers and east abutment (Lines 2-5). Due to the significant width of the bridge, the pot bearings at Girders 1-4 and 12-15 (exterior girders) were also designed to allow for transverse movement (Tremblay, R. and Mitchell, D., 2006).

On June 18, 2000, Girders 1-8 had been erected and were seated on their bearings, and threaded tie rods and timber blocking had been installed between the top flanges at

the ends of the girders, as had been used as in precast girder construction supported on elastomeric bearings. At approximately 10:45 a.m., the four exterior girders in the interior spans on the south side of the structure slid off their bearings, tilted over, and collapsed onto Highway 15, killing one person and seriously injuring another. The four exterior girders in the end spans also slid off their bearings, but were prevented from collapsing onto the highway below by the abutment wing walls.

A committee of experts was created to investigate the collapse, identify the causes of the collapse, and provide guidance on how to evaluate the adequacy of bracing for this type of construction. The committee collected mechanical, geometric, climatic, and loading data after the collapse, and a three-dimensional model of the girder-bearing system was developed. The analysis showed that sliding pot bearings do not offer significance resistance to the twisting of the girders about their longitudinal axis. This allowed the bearings to slide off their supports where transverse movements were permitted. Because the tie rods and timber blocking which only ran between the top flanges did not provide adequate bracing against girder rotation, the girders were unstable and a small load or disturbance could potentially lead to a collapse.

To mitigate the likelihood of the collapse of similar structures, the committee determined that adequate temporary bracing must be provided for girders supported by pot bearings prior to placement of the concrete deck and diaphragms. This bracing should include diagonal and lower lateral bracing to restrain rotation of the girder ends. Additionally, a stability analysis of the individual girders and the girder systems should be performed as a part of the design of the temporary bracing and to ensure the overall stability of the structure during construction.

SECTION 9. QUEBEC BRIDGE OVER ST. LAWRENCE (1907)

At 5:31 p.m. on the afternoon of August 29, 1907, the south arm of the cantilever truss bridge under construction north of Quebec over the St. Lawrence River collapsed. The New York Times reported that the sound of the collapse was clearly heard in Quebec, nine miles away. Of the 86 workers on the bridge at the time of collapse, 75 died; most bodies were never recovered (Rosenberger, J., 2004).

The Quebec Bridge was designed as a cantilever bridge with a clear span of 1,800 feet, the longest in the world at that time. The bridge had originally been designed by the Phoenix Company, who was responsible for design and erection, with a clear span of 1,600 feet, but this was increased to 1,800 feet. by Theodore Cooper on the basis that this would allow the foundations to be founded in shallower water, reduce ice loads and speed construction.

Theodore Cooper, author of the 1890 book, *General Specifications for Iron and Steel Railroad Bridges and Viaducts* and developer of Cooper's E Loading, was considered to be one of the preeminent bridge engineers in the United States. He had been retained as a consulting engineer on the project due to the lack of design experience of the Phoenix Company with a bridge of such large size.

During erection of chord members in the south anchor arm in June it was noted that some members were out of alignment (Pearson, C., and Delatte, N., 2006). This was attributed to fabrication tolerances and member adjustments were made in the field to allow riveting of the connections. As work progressed, additional member deformations were reported, including the compression chords near the pier. Jacking of members was required to achieve alignment and allow riveting of connections. Figure 2-16 shows the bridge under construction with the cantilever span to the left of the pier and anchor span to the right of the pier.



Figure 2-16 Quebec Bridge under Construction Shortly before Collapse (Modjeski et.al, 1919 – used by permission of Library and Archives of Canada)

In early August, bending was reported in two lower (compression) chords of the cantilever arm. When the deformations were reported to the chief design engineer for the Phoenix Company, he attributed them to shop fabrication, while others believed the deformations were due to accidental impacts or mishandling. Norman McClure, Cooper's on-site inspector, insisted however that the deformations were due to stresses that had occurred after the members were erected. Deformations were sufficient to cause some ironworkers to stop working, and on August 27, the construction foreman stopped work pending resolution of the observed problems.

On August 27, inspections of member compression chord A9L, the lower chord to the pier in the anchor arm, showed an increase in deflection from $\frac{3}{4}$ inch to $2\frac{1}{4}$ inch in two

weeks, and that the opposite chord also was deflected in the same direction. On the afternoon of August 28, McClure left to meet with Cooper at his New York City office. After their meeting on the morning of August 29, Cooper telegraphed the Phoenix Bridge Company's office in Pennsylvania, "*Add no more load to the bridge until due consideration of facts. McClure will be over at five o'clock*". McClure then left by train for the Phoenix Bridge Company's office to discuss the matter with Jon Deans, the Chief Engineer, and Peter Szlapka the Chief Designing Engineer for the Phoenix Bridge Company. Meanwhile, Alex Beauville, a riveting foreman on the bridge reported sheared rivets on the bottom chord, and others reported the bridge as becoming "springy".



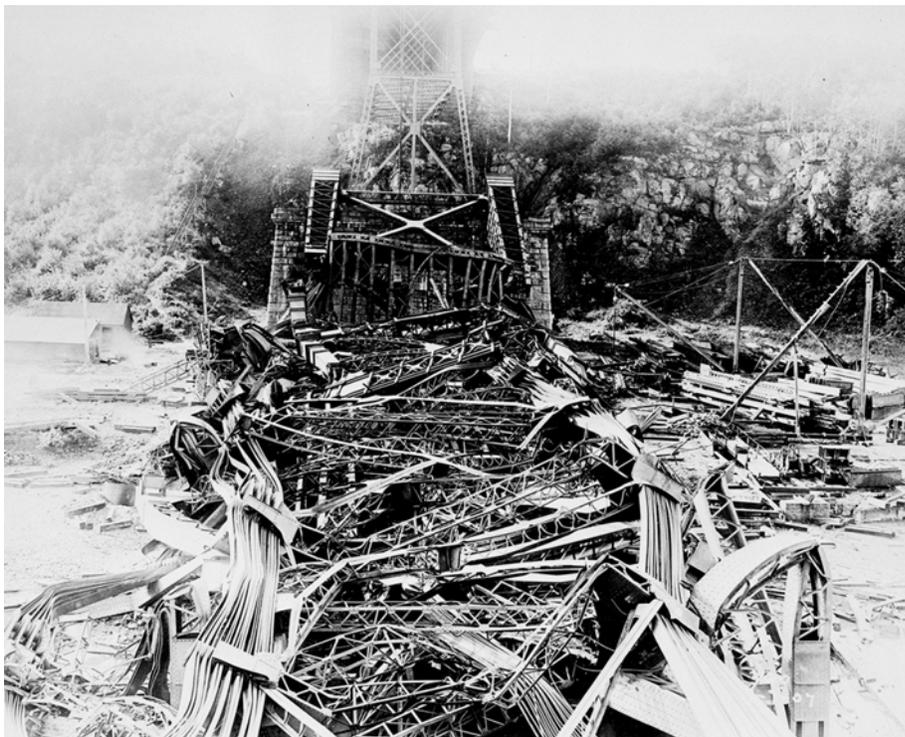
Figure 2-17 Quebec Bridge after Collapse (Modjeski et.al, 1919 – used by permission of Library and Archives of Canada)

McClure arrived shortly after 5:00 p.m. and found that Deans had received Cooper's telegram, but had not forwarded it, or other directives, to the bridge site. After preliminary discussions, they decided to await further field information before deciding on a course of action, and meet again in the morning. A few minutes after that decision was made, the bridge collapsed. Figures 2-17 and 2-18 show the collapsed bridge.

A Royal Commission was convened by the Governor General of Canada, comprised of three civil engineers. The Commission's report concluded that the immediate cause of the collapse was buckling of the A9R and A9L compression chords in the anchor arm, the design of which was found to be defective. Among factors cited as contributing to the failure were:

- Failure to adequately determine the true dead load resulting in stresses exceeding the design specification allowable. Actual dead load of the cantilever arm was 19.7% greater, and the anchor arm 30% greater than that used for design.
- The state of professional knowledge of the behavior of steel columns was not sufficient for design of the bridge.
- Lack of qualified on-site oversight.

The compression chords consisted of four ribs connected by riveted lacing bars. Testing performed after the collapse showed these behaved as four separate compression members rather than a single unit. Under load, the lacing and rivets failed, followed immediately by buckling.



**Figure 2-18 Quebec Bridge after Collapse – End View (Modjeski et.al, 1919
– used by permission of Library and Archives of Canada)**

Design of the Quebec Bridge, which would be the largest cantilever bridge in the world, pushed the limits of design knowledge in its time. The use of high allowable stresses, failure to fully redesign the bridge after increasing the length of the main span, and requirements for specialized fabrication and erection practices should have led to a heightened sensitivity to field observations and any unexpected occurrences during construction. Instead, the designers dismissed or attempted to explain observed member deformations as due to causes other than overstress, even in conflict with reports from their own field engineer. Even when the obvious signs of overstress and the onset of buckling were finally acknowledged, the designers failed to recognize the severity and immediately halt construction.

SECTION 10. I-80 GIRDER FAILURE (2005)

In March 2005, girder erection began for the construction of dual seven span bridges carrying I-80 in Clearfield County, Pennsylvania. Each 150 foot span was composed of five AASHTO, 28-by 96-inch deep prestressed concrete I-girders. The bearing types varied with location.

Figure 2-19 shows a simplified bridge elevation indicating the erection sequence. The abutments and Piers 1 and 6 used multi-rotational pot bearings with non-guided bearings at the exterior girders, and guided bearing at the interior girders. The remainder of the spans were supported on elastomeric bearings.

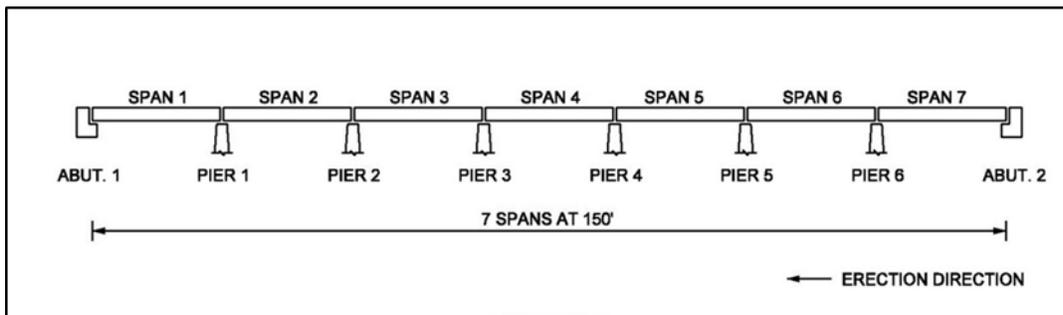


Figure 2-19 I-80 Elevation and Erection Sequence

Temporary girder bracing requirements varied with the bearing type. At Abutment 2 and Pier 6, spans supported by multi-rotational bearings, the bracing included dual diagonal bracing of the fascia girder and blocking of the non-guided bearings. This was in accordance with the erection drawings. For spans 2 through 5, supported on elastomeric bearings, the temporary bracing, as required by the erection drawings, consisted of a single diagonal brace at the fascia girder. A view of the bridge prior to collapse showing temporary bracing installed is shown in Figure 2-21.

Though the bearing type and thus temporary bracing requirements changed from elastomeric to multi-rotational at Pier 1 and Abutment 1, the temporary bracing was actually installed as though the girders were on elastomeric bearings, contrary to the erection plans. Figure 2-20 shows the bracing incorrectly used at the pot bearings on Pier 1 and Abutment 1.

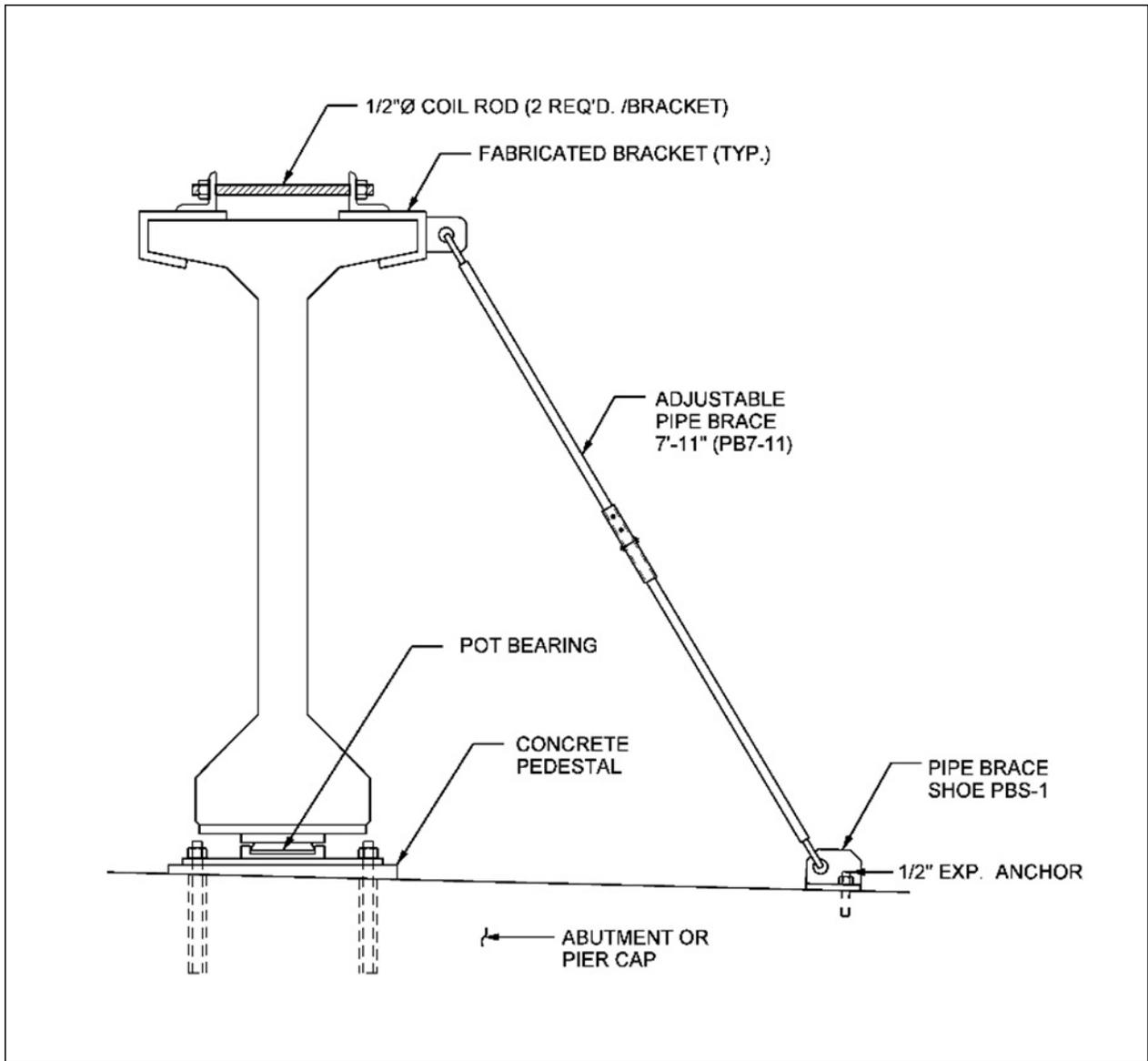


Figure 2-20 Incorrect Bracing at Pot Bearing, Pier 1 and Abutment 1

The unguided pot bearing provides no lateral restraint at the girder support. As a result, bracing only the top flange did not prevent the girder from rolling about the pot bearing which provided no lateral restraint. Investigation of the collapse included consideration of possible non-uniform thermal effects causing an increased sweep in the girders. An increased sweep would add an eccentric load due to girder self-weight that could contribute to a rolling instability about the bearing. Subsequent research work at Georgia Institute of Technology sponsored by PennDOT (Hurff, J., 2010), while indicating that thermal effects had minor effects, has provided important information regarding precast girder stability during erection. A major lesson from the I-80 collapse is the need to carefully review erection bracing details for bridges with varying bearing types to ensure adequate bracing design details and to provide field supervision to ensure that proper bracing is installed at each location.



Figure 2-21 End view of I-80 Bridge under Construction Prior to the Collapse

SECTION 11. CASE STUDIES – STEEL BOX-GIRDER BRIDGE FAILURES DURING ERECTION OUTSIDE OF NORTH AMERICA

2.11.1 Introduction

The importance of considering forces imparted during construction, in addition to the final configuration intended by the designers, is illustrated in the following case studies.

Between 1969 and 1973, five steel box-girder bridges failed during construction outside of North America. Four of these occurred within a two-year period and were well publicized at the time of each failure. The fifth failure occurred two years after the fourth and was only publicized many years later. Each of these collapses reveals specific lessons and the loss of 57 lives as a result of these failures underscores the importance of considering all possible loading combinations when developing erection methods.

2.11.2 Fourth Danube Bridge

The Fourth Danube Bridge in Vienna, Austria, was designed using three-span continuous welded twin steel box-girders with an overall length of 1,351 feet and span lengths of 394 feet, 689 feet, and 269 feet. Each rectangular box cross-section was 25 feet wide, and the overall depth of the haunched girder was 17 feet at midspan. The flanges were longitudinally stiffened by flat plates, rather than T-sections or similar configurations that would come to be used in more modern construction of this type (Akensson, Bjorn 2008).

Span 2 of the bridge was being erected by the free cantilever method in November of 1969. Spans 1 and 3 were fully in place, with cantilever sections extending from both piers. When the final section of Span 2 was ready for placement, it was determined that the warm temperatures experienced on the final day of erection caused the cantilever sections to deflect downward more than expected, due to the tops of the sections having undergone additional temperature-induced deformations relative to the bottoms of the sections. As a result, the final section did not fit as intended.

In an attempt to mitigate this problem, it was determined that the closure section would be shortened by $\frac{5}{8}$ inch at the top, which represented the difference in temperature deformation. It was recognized at the time that this would introduce undesirable constraining forces and a resulting distribution of forces and moments that the system was not designed to resist. Specifically, in this configuration, the bottom flange would experience a higher compressive force than intended. The intended solution was to lower the interior supports at the piers in order to artificially introduce tension in the bottom flange.

However, due to the required modifications of the closure section, there was not enough time remaining in the workday to lower the supports and this operation was rescheduled for the following day. During the evening and overnight hours, the bridge was not in the direct heat of the sun, and the temperature dropped to more seasonable levels. This temperature swing introduced significant compressive forces into the bottom flange. These forces were exacerbated by the poor longitudinal stiffener detailing. As a result, the superstructure buckled near the right-hand dead load moment contraflexure point in Span 2, which was not designed to resist significant bending forces. This buckle redistributed the forces in the system and a second buckle occurred approximately at the midspan of Span 1. These two locations buckled enough to form nearly pure hinges at these locations. A third buckle, at the closure section, occurred, but this was not as pronounced as the first two.

The first two hinges transformed the structure from twice statically indeterminate to statically determinate. This enabled the internal forces to redistribute and relieve the constraining forces, preventing a collapse. Had the buckle at the closure section been pronounced enough to form a third hinge, a collapse mechanism would have developed.

2.11.3 Cleddau Bridge

The Cleddau Bridge in Milford Haven, Wales was designed as a seven-span continuous welded single-cell steel box-girder with an overall length of 2,688 feet - 9 inch. The box cross-section consisted of a 41-foot wide top flange, a 22-foot wide bottom flange, and two 18 foot deep inclined webs. Other than traditional longitudinal and transverse stiffening elements, there were no interior plate diaphragms or other stiffeners, except those at the piers (Akesson, Bjorn 2008).

At the time of collapse, in June of 1970, Span 7 (the southernmost span, 252 feet long) was fully erected and shored approximately 140 feet from the abutment. Span 6 (also 252 feet long) was being erected by the cantilever method. Individual box segments were incrementally launched from Pier 6 northward toward the free end of the cantilever. When the bridge collapsed, 196 feet of Span 6 had been erected and was cantilevered out from Pier 6. An additional box section, having a length between 50 and 60 feet, was being launched, when the cantilevered section of Span 6 collapsed. The section buckled about Pier 6 and fell to the ground. At the time of collapse, the cantilever section weighed approximately 500 tons. Four people were killed in the collapse.



Figure 2-22 Cleddau Bridge Buckled over Pier

After an investigation, it was determined that the mode of failure was buckling of the pier diaphragm, which was designed to resist loads imparted in the final configuration of the bridge, but was insufficient to resist the erection loads.

2.11.4 West Gate Bridge

The West Gate Bridge in Melbourne, Australia, with an overall length of 8,530 feet, was oriented in an east-west direction. The approach spans consisted of reinforced concrete and the center five spans consisted of a continuous welded steel box-girder structure. The middle three spans were supplemented by stay cables.

The cross-section consisted of a 13-foot deep, three-cell box-girder. The top flange was 123 feet wide, and the bottom flange was 63-feet wide. The interior web plates were vertical, while the exterior web plates were inclined.



Figure 2-23 West Gate Bridge after Collapse

Due to a series of delays, construction fell behind schedule, and as a result, the contractor devised a method of erection that would help bring the project back on schedule. The chosen method of erection, along with a series of poor decisions associated with this method of erection, eventually led to the collapse of this bridge (West Gate Bridge Memorial Committee, 1990).

The method selected was to split the box-girder longitudinally into two halves and introduce a longitudinal splice. The two halves would then each be lifted separately into place by hydraulic jacks and then rolled into place and spliced. This method of erection was intended to speed construction because of the greatly reduced number of crane lifts compared to conventional construction. However, it left long unsupported lengths of flange on each half.



Figure 2-24 West Gate Bridge Construction Site after Collapse

Construction began on the east side by assembling the two halves on the ground. However, because the flanges were unsupported on their free edges due to the division of the three-celled box section into two halves, the top flange buckled while the girder halves were still on the ground in a simply supported configuration. The amplitude of these buckles was up to 15 inches. In spite of this, the contractor elected to continue with the erection.

Initially, each half was moved into place as described above. Before completing the longitudinal splice, the contractor needed to address the buckles at the free edges of the top flanges. To relieve the distortion caused by the buckles, the contractor unbolted several transverse splices, allowing the adjacent top flange plates to slip past each other in the longitudinal direction. New holes were drilled, or existing holes enlarged, in order to re-bolt the transverse splices (Morrison et.al, 1971).

Construction then proceeded to the west side. Considering the lessons learned from the free edge buckling of the top flanges that occurred on the east side, the construction team utilized additional stiffening elements on the top flanges of the two halves of the west sections, which initially controlled the free edge buckling. However, when the two halves were about to be longitudinally spliced, it was noticed that the north half was cambered 4 ½ inches more than the south half.

In order to make the longitudinal splice, it was decided that the camber of the north half would be reduced by placing concrete blocks on this portion of the structure. It was due to this loading configuration that the free edge of the top flange of the north half eventually buckled, as the supplementary stiffening elements were unable to resist the additional buckling stresses caused by the dead weight of the concrete blocks. At this point it was decided that the buckles would be taken out in a similar manner as the east side; that is, some of the transverse splices would be unbolted, which in turn would allow the top flange plates to slide past one another in the longitudinal direction. However, the additional load of the concrete blocks was not taken into account in this case, and unbolting of the splice caused additional stresses in the top flange. The entire section buckled and collapsed to the ground, taking the lives of 36 workers.

2.11.5 Rhine Bridge

The Rhine Bridge in Koblenz, West Germany was designed as a three-span continuous welded single cell steel box-girder with an overall length of 1,450 feet and span lengths of 338 feet, 774 feet and 338 feet. The box cross-section consisted of a 97-foot wide top flange, a 36-foot wide bottom flange, and two inclined webs. The overall depth of the haunched girder was 19 feet at midspan. The bottom flanges were longitudinally stiffened with T-sections.

Span 2 of the bridge was being erected by the free cantilever method. Spans 1 and 3 were fully in place, with cantilever sections extending from both piers. The crane was positioned at the end of one of the cantilever sections and was in the process of lifting the 85-ton closure section into place when this cantilever section collapsed, failing in

negative bending moment. The section that failed was not at the pier, the point of maximum moment, but rather at approximately the $\frac{1}{4}$ point of the span.

Neither the reason for the location of the failure, nor the actual cause of the failure, was immediately clear. As a part of the investigation, these became apparent. First, the location of the failure was within the section of the span for which the minimum depth of the cross-section was used. Second, this location was near the point of dead load moment contraflexure; therefore, this section would have been designed for a much lower level of stress than was experienced during the construction phase. Finally, and most importantly, the root cause of the failure was determined to be the inadequate detailing of the longitudinal stiffener at this location (Akesson, B., 2008).

The failure occurred at the location of a transverse splice. Longitudinal stiffeners were welded to the bottom flanges at these locations, but the stiffeners stopped approximately 9 inch short of the centerline of the splice in order to facilitate the welded splice. Following the splicing of the girder section, it was determined that what was labeled a supplementary T-section stiffener would be spliced to the main longitudinal stiffeners; however, this supplementary section was positioned such that a 1 inch gap was introduced between this stiffener and the bottom flange (See Figure 2-25). The gap was introduced, in part, to avoid intersecting welds; however, the resulting configuration did very little, if anything, to stiffen the bottom flange at the location of this splice. It was determined through the ensuing investigation that the bottom flange buckled at the splice location. If the longitudinal stiffener had been configured to function properly, this would very likely not have occurred.

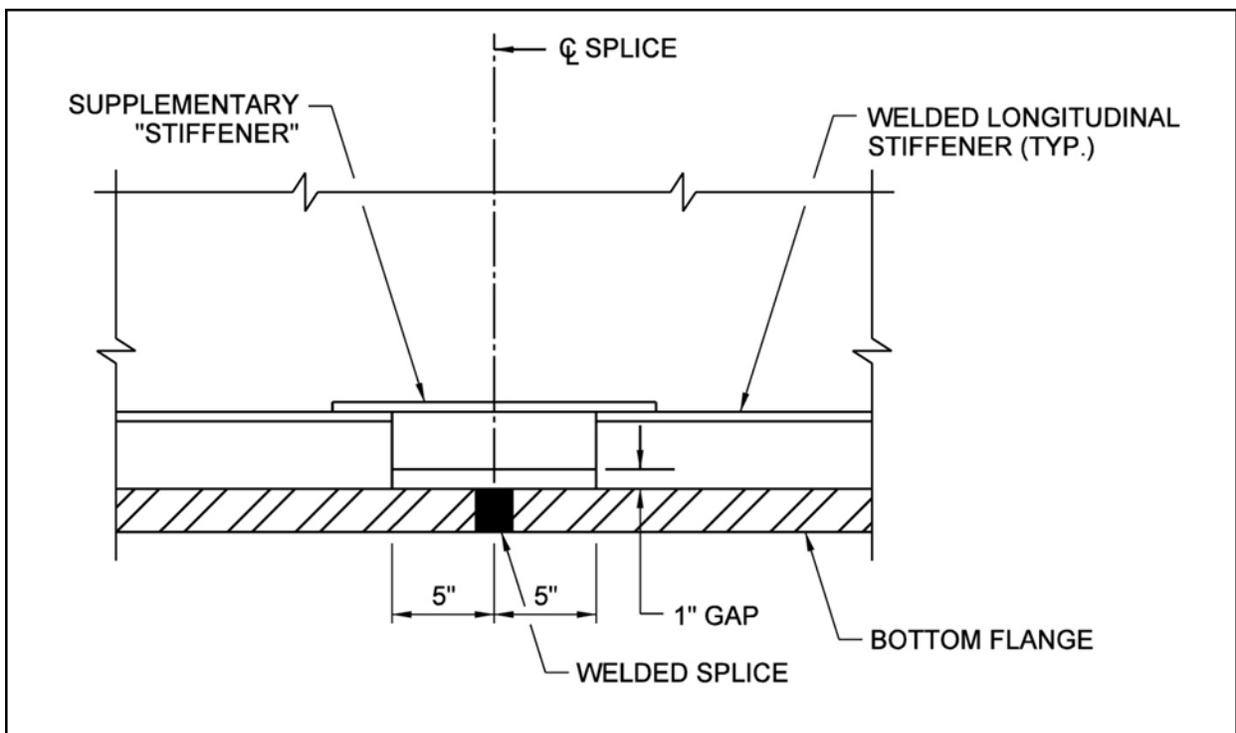


Figure 2-25 Rhine Bridge Longitudinal Stiffener Splice

2.11.6 Zeulenroda Bridge

The four bridge failures described above are well-known examples of steel box-girders failing during erection. However, a fifth failure occurred in Zeulenroda, East Germany in August of 1973. This was not reported until the archives were opened to the public in the late 1990's (Akesson, B., 2008).

This bridge was designed as a single-cell six-span continuous welded steel box-girder bridge. The overall length of the bridge was 1,188 feet, two end spans of 180 feet, and four center spans of 207 feet. The cross-section consisted of a 36-foot wide top flange, a 13-foot wide bottom flange, and two inclined webs, and an overall girder depth of 7 feet.

One of the end spans and about half of the adjacent span had been erected using the free cantilever method. Near midspan (just beyond the point that had already been erected), a shoring tower was in place to assist with the erection. However, the bridge collapsed just short of the shoring tower during the erection of one of the box-girder segments.

No formal investigation was performed regarding the cause of collapse; however, Akesson, after performing load and capacity calculations on the failed section using the speculated erection loading, concluded that the bottom flange was inadequately designed to handle the erection loads, and the longitudinal stiffeners were inadequately designed and detailed for the required loading during erection.

SECTION 12. SUMMARY AND CONCLUSIONS

Although the circumstances surrounding the failures discussed in this chapter varied widely, some common conclusions can be drawn from the summaries of these failures that were presented:

- Loading for both primary and secondary members, including loading from jacks, cranes, support towers, thermal creep, concrete creep, wind, and construction loads must be considered at every point of the construction sequence.
- The contractor's erection engineer must be cognizant of the various stages of erection, how each stage affects the loads and capacities of the sections being loaded, and how changes during construction due to fabrication errors, changes in construction methodologies and other considerations affect the manner in which load is applied and resisted.
- Secondary elements such as stiffeners must be properly designed and detailed to account for the required loading conditions during each stage of construction as well as for the final configuration of the structure.
- A stability analysis of both the individual components and overall systems should be performed as a part of the design of temporary bracing and shoring to ensure

the overall stability of the structure during construction. Lateral instability must be considered at every point during the construction.

- At each point in the construction sequence, all bracing, bolting, and framing assumed in the design of that construction sequence must be in place.
- The geometry of the structure must be monitored at each step of the construction.
- Bridges relying on the deck for structural stability must be checked for conditions when the deck is not yet providing that stability. A structural analysis of the bridge behavior during concrete placement should always be performed.
- Box girders may need additional top flange bracing to accommodate the construction loads.
- Field personnel should be alert to unexpected deformations, member fit-up difficulties, unusual sounds, etc. and report them to supervisory and engineering staff. The reason for such occurrences should be carefully investigated.
- No modifications to the permanent structure or temporary works should be made without an engineering analysis

CHAPTER 3

TYPICAL BRIDGE CONSTRUCTION PRACTICE

SECTION 1. INTRODUCTION

Various construction activities and considerations that can affect girder stability occur in the erection of bridge superstructures. Chapter 3 discusses the processes followed during erection, various options in erection methods and sequencing, and presents an overview of typical equipment used along with some of its limitations. Providing stability at all stages of erection is critical and various methods of achieving stability and geometry control may be utilized. The effect of deck placement and demolition sequencing on overall structural behavior must also be considered. Detailed designs of shoring are beyond the scope of this chapter, but references to pertinent AASHTO provisions are included.

SECTION 2. ERECTION SEQUENCING

3.2.1 Site Conditions

The site conditions often dictate the sequence and method of erection. Locations of roadways, site topography, railroad tracks, existing structures, waterways, overhead and underground utilities, and maintenance and protection of traffic through the work zone typically control where cranes and shoring can be placed and where girders can be delivered by truck. When girders cannot be erected directly from trucks or other delivery methods, adequate laydown space must be available and prepared for temporary girder storage. Based on available crane locations, a lifting or pick radius from the delivery or laydown location and a setting radius to the final girder location can be determined and an appropriately-sized crane can be chosen.

Soil pressures under crane tracks, wheels, or outriggers that develop during lifting must be compared to allowable soil bearing capacities to preclude crane tipping due to outrigger or track settlement. Compacted stone pads or timber mats are often constructed to facilitate crane operations and distribute the loads.

3.2.2 Girder Shipping

Shipping weights and piece lengths for girder segments can typically be found on the fabricator's or precaster's shop drawings. The engineer preparing the erection procedure should perform take-offs of the girder weights for determining crane requirements and preparing stability calculations and center of gravity calculations. These calculations also serve as a second check on the fabricator's estimation of girder weight.

Girders are typically shipped with webs in the vertical position and the girder is chained to the truck dollies – refer to Figure 3-1. Depending on the constraints of the project site and whether the truck has to be backed in at the site, the erector will select the orientation of the girder and relay that information to the fabricator so that the pieces arrive in the desired orientation.



Figure 3-1 Bridge Plate Girder Loaded for Transport to Project Site

Transporting the girders from the fabrication shop to the project site typically requires an in-depth analysis of possible routes, including routing clearance checks, loading analysis and permitting with applicable routing agencies. Girders may occasionally be shipped by water or rail, in which case an intermediate step of off-loading to trucks for local transport may be necessary.

Though size and weight limits for truck transport vary with available highway access, typical limits on over-the-road shipping applicable to both steel and concrete girders include:

Length – Maximum length is normally in the 120-140 foot range, though lengths up to 185 feet may be possible for some routes, and by utilizing steerable trailers.

- Width – Widths over 8 to 10 feet require permits. Maximum permitted widths normally are limited to 12 to 16 feet.
- Height – Height of trailer plus girder is normally limited to 12 feet above the roadway without an over-dimension permit and 14 feet if permitted.
- Weight – Depending upon available transportation equipment and state regulations, loads up to 110 tons may be transportable. The ability to transport a heavy load may be governed by sufficient member length to accommodate the number of axles needed for the load.

Transport of large or heavy loads requires permitting and may be limited by construction zone width restrictions on some routes and allowable travel hours. Routes should also be evaluated for high cross slopes which could result in instability of loads with high centers of gravity. Over-dimension loads will also require escort vehicles as shown in Figure 3-2. As part of the shipping plan, the supplier should complete an analysis of the girder to ensure that stresses during shipping remain within the AASHTO limits.



Figure 3-2 Concrete Girder in Transport with Escort Vehicles

3.2.3 Erection Sequence

3.2.3.1 Initial Preparation

Each bridge will require an erection sequence that meets site and design conditions for that bridge. Although the sequence is the responsibility of the bridge erector, the following outlines the concerns that must be taken into account. Prior to the start of girder erection, the bridge bearings are set and their elevation and location verified. Bearing support surfaces, particularly for elastomeric bearings, must be flat within specified tolerances in order to provide uniform bearing.

Erection often begins at a fixed bearing, or at one end of the bridge, where the bearing stiffener in the web of the girder is used to locate the bearing centerline and establishes a control point at the start of girder erection. For bridges located on a grade, erection normally starts at the abutment with the lowest elevation, and in these cases it may be necessary to block the girders to the backwall or install restraint blocking to the girder seats to maintain girder location.

On longer bridges, some of the initial pieces may intentionally not be set on the centerline of bearing but rather be offset on the bearing by one to two inches toward the backwall to provide a larger gap into which the final girder piece can be set. Once the final piece is in position the bridge is jacked longitudinally to close the gap and final bolting of the field splices is performed. When this procedure is used, the girders must be checked to ensure they possess sufficient axial capacity under self-weight to resist the jacking forces.



Figure 3-3 Splicing Girder Segments at Grade Prior to Lifting

3.2.3.2 Steel Girder Bridges

When sufficient crane capacity is available to lift the combined weight, girder segments are spliced at grade to yield longer sections for placing as shown in Figure 3-3. When the girders are erected at grade, some or all of the cross-frames are attached at that time. If girders are to be set one-by-one, the erection process usually starts with setting the fascia girder, stabilizing the girder, unhooking the crane and then setting the first interior girder with cross-frames already attached to the girder.

Once the first interior girder is in place the cross-frames are swung into place and connected to the fascia girder prior to the crane releasing the load. The number of cross-frames attached to the second girder is dependent upon what is needed for lateral stability of the girders. Remaining cross-frames and lateral bracing (if specified) are then erected. The third girder then is erected and the cross-frames hanging off of the second girder are connected to the third. Similarly, the fourth girder is erected with cross-frames attached, and then any additional girders with each pick alternating between plain girders and girders with cross-frames already attached. Figure 3-4 depicts a curved girder being erected with cross-frames attached. In this photo, the temporary shoring is actually above the girders and the girders are suspended from an adjustable chain-fall system.

An alternate erection process is to install the cross-frames between a pair of girders working at grade, and then lift and set the girder pair. While a higher capacity crane may be needed than for setting single girders, setting pairs eliminates the need to provide temporary bracing at the ends of the first girder set. Subsequent girder pairs are then set, and the cross-frames between pairs installed.



Figure 3-4 Curved Girder Being Erected with Cross-Frames Attached

When the bridge consists of multiple spans, due to the expense associated with the main lifting crane, the crane will usually be relocated to its next position to erect the subsequent span or girder segment. A smaller crane is then mobilized to complete the erection of cross-frames and any lateral bracing in the first span.

In curved girder bridges, the first individually erected curved girder will not be stable if set in place on the bearings by itself. Therefore, it is common for the first two girders to have the cross-frames installed while the girders are on the ground and lifted as an assembly. Otherwise, additional cranes and/or shoring systems are required to hold the first girder in place while a second and perhaps a third, or more, adjacent girder(s) are lifted and connected with cross-frames. Even with a paired erection scheme, temporary shoring systems may still be required. An engineering analysis of the system at each stage of erection is required to determine the arrangement of the shoring system and the applied loads and deformations as successive girders are set.

Studies of curved and skewed girder erection sequencing conducted for the Pennsylvania Department of Transportation (Linzell, David, et. al. 2010) found that setting curved girder pairs reduced vertical deflections during erection, particularly compared to erecting single girders beginning on the outside of the curve. Based on their studies, the best erection scheme for controlling deflections, especially for tight radii, is to erect girder pairs working outward from the inside of the curve.

Cross-frames must be installed as the girders are set and be secured with full-size pins and high strength bolts. Partially installed or insufficiently secured cross-frames lead to poor control of girder alignment and complicate fit-up as erection progresses.

3.2.3.3 Concrete Girder Bridges

The preferred method of erecting long precast concrete girders is with two cranes, with one located at each bridge girder support. Single cranes are generally limited to placing precast concrete girders with spans of 60 feet or less. Single-crane lifts require a sufficient boom length and lifting cables of sufficient length to keep the lifting cables at a minimum angle from the horizontal, typically specified as 60 degrees. Spreader bars or struts can also be used to control this minimum angle.

Erection using two cranes is usually faster than with one crane. The method is normally used when long girders can be delivered along the bridge span, and cranes positioned near the girder supports can lift and swing the girders from the delivery vehicle directly to their final position. It is also used where precast girder sections are spliced at grade. Dual-crane lifts can utilize shorter booms than single-crane lifts, which can be important where headroom is limited.

Girder erection normally begins with the fascia girder which is set and braced to the abutment or pier cap. The adjacent girder is then set and braced individually or, more commonly, to the previously set girder.

Once the girders are positioned, required temporary bracing remains in place until the end diaphragms, and any required intermediate diaphragms, are placed / cast.

For spliced girder and similar erection, the initial pier girder segments are set on their piers, with supplemental support from shoring towers or bracing extending back to the pier shaft. In the latter case, the bracing design must account for potential unbalanced loads to the pier. Once the pier girders are placed, the drop-in girders are placed, as shown in Figure 3-5. These are typically supported on separate or shared shoring, or are suspended by a strongback, Figure 3-6 and Figure 3-7, hanger assembly. The selection of a temporary support system depends upon bridge elevation, presence of waterways, and contractor preferences. A work platform may also be provided to allow for completion of the girder splicing operations.



Figure 3-5 Setting Drop-in Span with Strongbacks Attached



Figure 3-6 Girder Supported on Strongback (Shown on the Right)

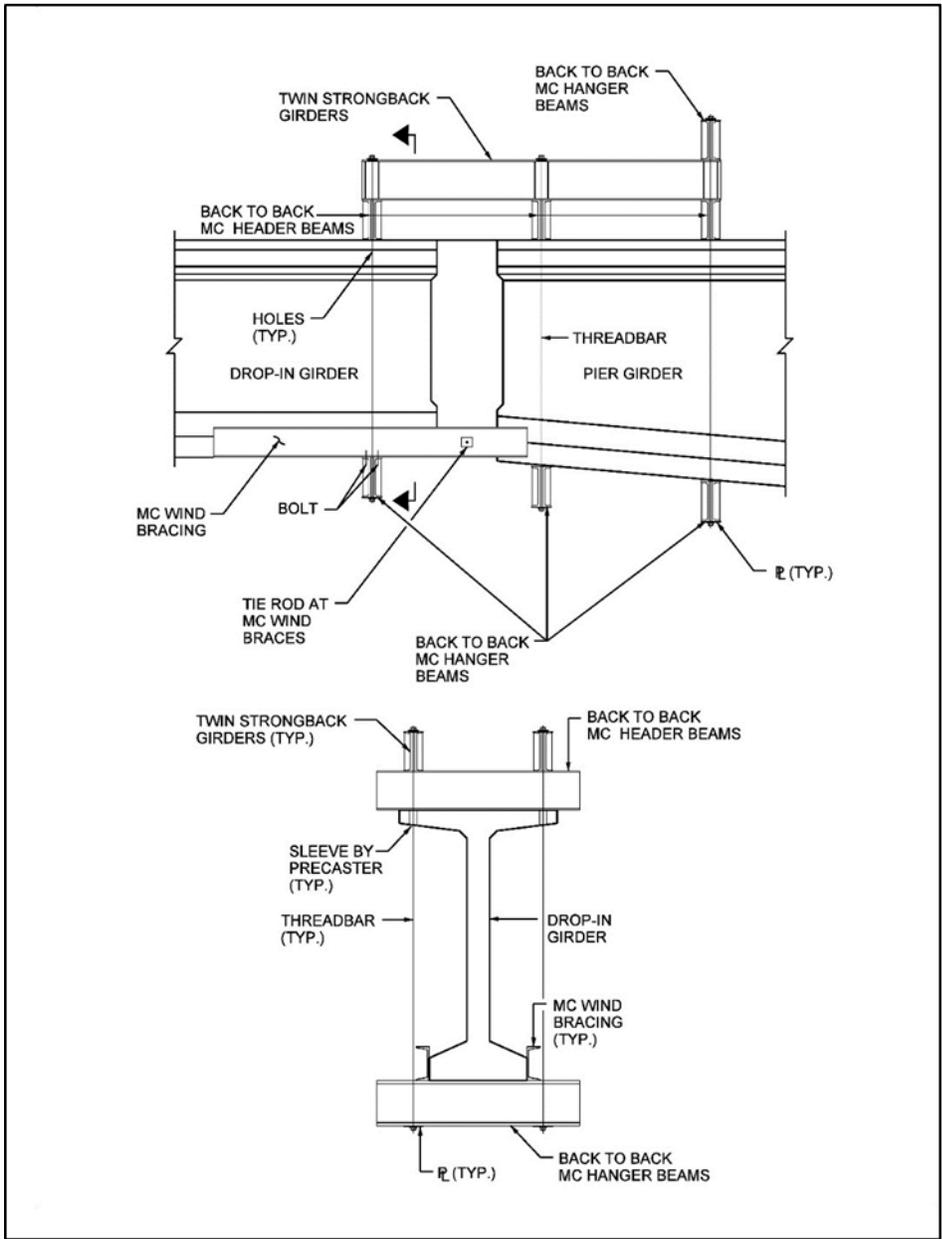


Figure 3-7 Typical Strongback Arrangement

SECTION 3. GIRDER HANDLING

3.3.1 Center of Gravity

A critical component to erecting the bridge girders and other components is determining the center of gravity of each piece. The center of gravity is a point on the girder where the total weight of the piece may be thought to be concentrated. If the girder was supported at the center of gravity, it would be in a balanced condition and the moment from the weight of the right portion of the girder about the center of gravity would equal the moment from the weight of the left portion of the girder.

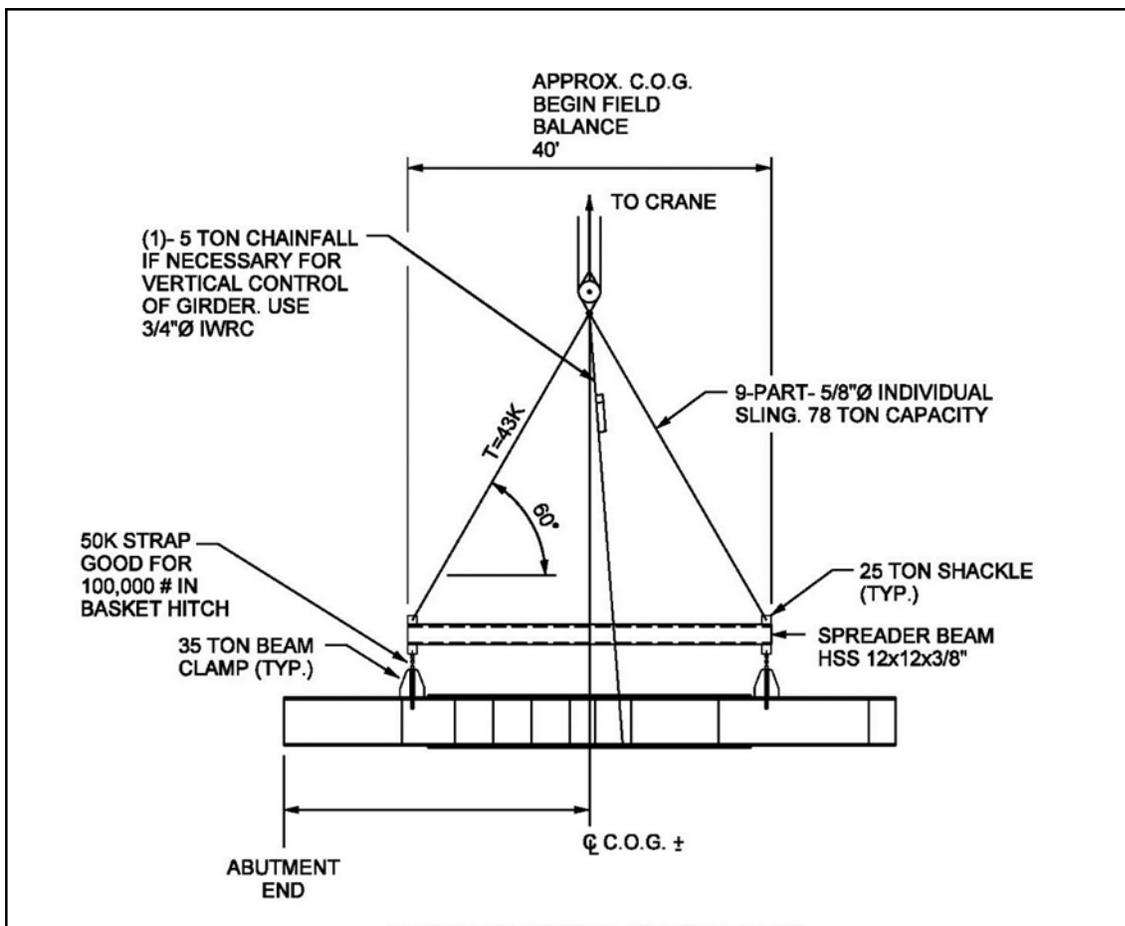


Figure 3-8 Girder Rigging with Spreader Beam

Erectors will often verify the calculated location of the center of gravity in the field by lifting the girder a few inches off the delivery truck to see if it hangs level. Slight adjustments to the theoretical center of gravity that was calculated can then be made in the field.

3.3.2 Rigging for Straight and Curved Steel Girders

On single crane picks the crane hook is located over the center of gravity of the girder. On shorter pieces, a single beam clamp or pair of beam clamps with slings is often utilized, whereas on longer heavier pieces a spreader beam or lifting beam along with a pair of clamps is utilized to pick the girder segments as shown in Figure 3-8. A typical beam clamp is shown in Figure 3-9. Beam clamps are commonly available in 15 ton, 25 ton, and 35 ton capacities and connect to the top flange of the girder. Local stresses in the top flange due to the beam clamp need to be investigated, as discussed later in this chapter.

Single crane picks with a single lift point are not recommended for curved girders. Use of a lifting beam with two lift points spaced so that the lifting points lie on the center of gravity of the curved girder will minimize rotation during lifting. For larger girders, the use of two cranes with a total of four lifting points will improve girder stability and facilitate fit-up of the splices.



Figure 3-9 Beam Clamps on Steel Girder

3.3.3 Rigging for Concrete Girders

Concrete girders typically involve a two crane pick both due to their weight and the crane attachment points are at each end of the girder. Lifting loops, most often consisting of one or more prestressing strands, are typically cast into the girder near the girder ends and the loops are connected to a shackle and a sling as shown in Figure 3-10. Lifting points used to erect the girders should be the same as used by the precaster to move and ship the girders. The critical condition for the lifting loops is when the girders are removed from the casting beds since the concrete has not yet achieved its full strength.

If concrete girder segments are light enough for the available crane capacity a single crane pick using a spreader beam can be used to pick the girder. If inclined lifting cables (or straps) are used, the axial force in the girder segment due to the inclined cables must be included in the lifting analysis by the precast and erection engineers.



Figure 3-10 Crane Rigged to Concrete Girder at Lifting Loops

Other options for rigging of concrete girders include utilizing a basket hitch near the end of the girder as shown in Figure 3-11. The basket hitch is typically positioned so that it does not interfere with the abutment or pier when the girder is set.



Figure 3-11 Basket Hitch Rigging

When multiple lifting points are used, for instance, lifting at four points along the girder, techniques for equalizing the load on each lifting point are used. This may be done with roller blocks, spreader beams or lifting trusses.

3.3.4 Local Stresses from Beam Clamps



Figure 3-12 Beam Clamp

Beam clamps grip the beam at three points: the underside of the top flange on each side and the top of the girder. When properly balanced and safely guided, the beam can be handled even if the clamp is slightly away from the center of gravity of the piece. Good safety procedures providing control of the lifted beam must be used. Snubbing lines (tag lines) at each end must be used to control excessive twisting or swinging, and to guide the beam to its proper place.

The weight of the beam clamp automatically opens its tongs, which slide under the flanges of the beam (refer to Figure 3-12). When the clamp is lifting, its center plate and gripping tongs work against each other—the heavier the beam, the greater the clamping pressure. Some models of clamps have a recessed base to accept studs welded to a beam surface.

When utilizing beam clamps to pick steel girders, significant localized stresses are produced in the top flange of the girder. These stresses should be checked (see Section 7-6.3) and, if necessary, the area reinforced. This is usually accomplished by adding a loose plate on top of the flange under the clamp or adding a plate or angle between the tongs and the bottom of the flange.

3.3.5 Lifting Beams & Spreader Beams

Spreader beams like that shown in Figure 3-13, or lifting beams as shown in Figure 3-14, are typically utilized to handle longer straight members and horizontally curved steel girders. The design of lifting beams and spreader beams should be in accordance with ASME BTH-1 *Design of Below-the-Hook Lifting Devices*. This reference requires all below-the-hook lifting devices to be designed for specified rated loads, load geometry, Design Category and Service Class. The resolution of loads into forces and stresses affecting structural members, mechanical components and connections used in rigging are performed using classical strength of material methods although other analysis techniques can be used.

Spreader beams are designed as compression members along with some minor bending due to self-weight and possible eccentricity from the sling attachment. Lifting beams are primarily bending members that typically attach in their center directly to the crane hook, and then suspend beam clamps or lifting cables from each end to attach to the girder.



Figure 3-13 Spreader Beam



Figure 3-14 Lifting Beam

Assembly can generally be thought of as maneuvering of the pieces with cranes into their final positions in the structure, whereas fit-up consists of aligning the members and field splice plates.

During the assembly phase, a ground crew selects pieces based on the shop or erection drawings. The crew typically dresses the girders, provides any additional rigging such as safety cables or tag lines, and hooks the crane to the pieces to be erected. The 'top men' guide the pieces into their appropriate positions and secure them to the structure or temporary shoring if required by the erection plan.

Fit-up is the process of aligning the members and the plies through the use of drift pins, and erection or 'fit-up' bolts. Erection bolts are used to draw the plates in the splice or connection into contact. Drift pins are steel pins tapered at each end and cylindrical in the middle (Figure 3-16). The cylindrical portion has the same nominal diameter as the open bolt hole. The drift pins are driven into the splice through all plies at the corners of the connection plates, thereby drifting into alignment the flanges and webs of the girders. Once this is accomplished, about 25% of the remaining holes also have drift pins driven into them uniformly throughout the connection (Figure 3-15).

Members are typically first aligned vertically through the web splice, and then aligned horizontally through the flange splice. High strength bolts are then installed as the drift pins are removed. Typically, 50% of the holes are filled with high strength bolts and erection pins prior to the crane releasing the load; however, bridge owners may have differing requirements, which must be followed. The AASHTO LRFD Bridge Construction Specifications require a minimum of 75% of the holes be filled if the structure is carrying traffic during erection, as may occur during rehabilitation projects.

Cross-frames and diaphragms should be pinned and bolted to a snug condition but not fully tightened until all of the girder lines are complete. This provides for stability and allows for adjustment as erection proceeds. On curved girder lines, the cross-frame connections also typically have 50% of the holes filled with high strength bolts or pins prior to the crane releasing its load, though some owners may require a higher percentage.

The final bolts are installed typically from the center of the connection toward the outward edges. Erection bolts can be used as many times as necessary, but cannot be used as the final bolts.



Figure 3-15 Girder Splice Bolts Being Installed and Drift Pins in Place

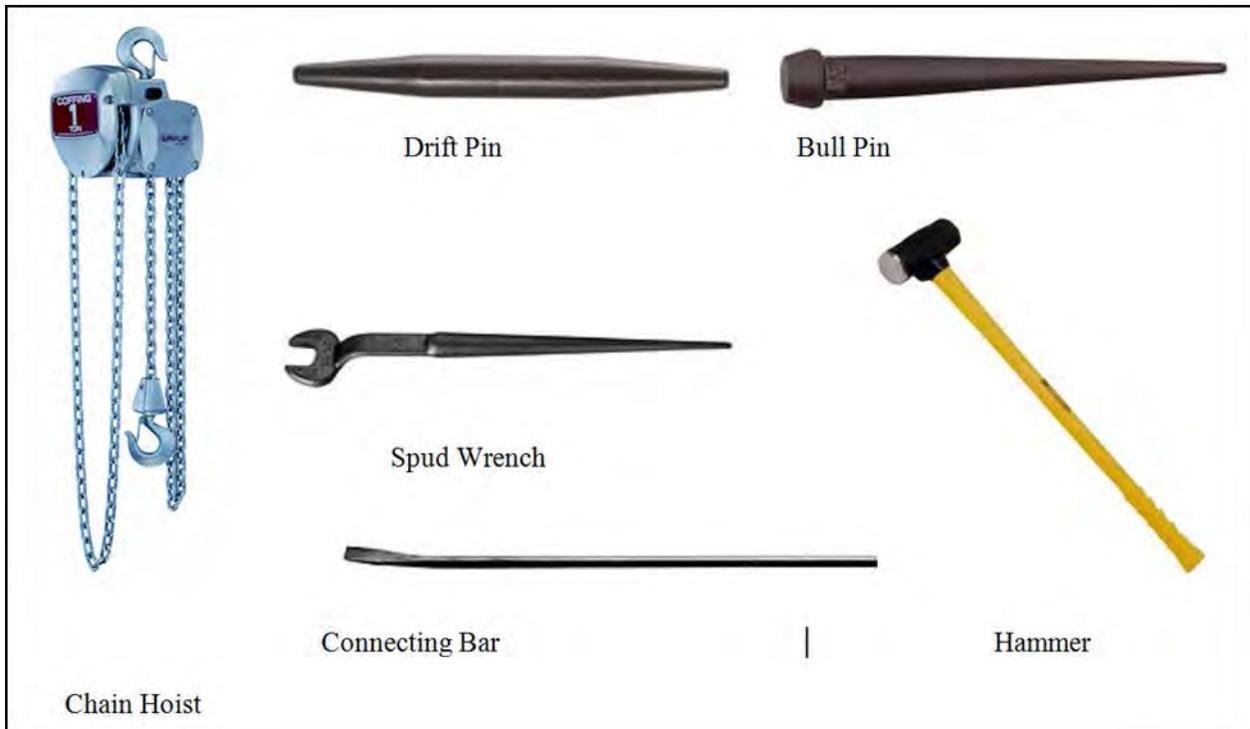


Figure 3-16 Ironworkers Tools of the Trade

3.3.7 Bolting Procedures

Bolts are typically ASTM A325, A490 or F1852 as specified in the contract documents. The bolt installation method should be in accordance with the AASHTO LRFD Bridge Construction Specifications and owner requirements.

Bridge girder lines are usually made up of several pieces that are bolted together in the field. These bolted connections must be properly aligned and tightened before the splice can perform as designed. Field splices are generally located near a point of zero dead load bending stress. The field splices are labeled on the erection sheets of the shop drawings and may differ from those shown on the design plans if the fabricator or erector requests, and has received approval for an alternate location to facilitate erection or shipping.

Access for the ironworkers to assemble and bolt the splice may utilize man lifts, suspended platforms, cages or other methods. Providing room for this equipment may preclude installation of bracing or cross-frames near the girder splice until the splice is complete. Though splice size varies widely, an initial estimate of the time required to make up a splice is around 30 minutes. Pins and bolts placed at the edges of a splice are more effective in maintaining member alignment (and in web splices, moment capacity). This is of particular importance where splices are initially only partially bolted. In order to reduce the time that large cranes are required on the jobsite for erection, contractors may initially erect the structure using partially bolted connections and then

complete bolting as a follow-up activity. Similar situations occur when erection work periods are of limited duration due to traffic or access restrictions.

The common requirement for installation of at least 50% of the high strength bolts in a connection, prior to releasing it from support, is recognition that the self-weight of the steelwork produces lower loads on the splice than do the final design loads. These bolts must be properly tightened, not only to carry their loads, but to prevent connection slippage and maintain member alignment. The erection engineer should review connection configuration and location and ensure that loads during erection do not exceed the assumption of the 50% bolted criteria for each erection stage. For instance, continuous girders cantilevering beyond a support pier develop stresses on the splice in the trailing span that differ from the steel self-weight stresses in the final configuration. Partially bolted connection capacity evaluation must recognize that the splice bolts constitute a bearing connection until final tightening.

3.3.8 Field Welding Considerations

Field welded connections are not widely used in bridge structures. However, field welded girder splices and cross-frame connections may be encountered. When field welds are part of the load resisting system, the welded members should not be loaded until welds are fully completed, unless an engineering stress analysis is performed that demonstrates sufficient capacity for intermediate conditions. Temporary supports and guides for field welded members must be sufficiently rigid to assure that the required weld joint geometry is maintained during welding.

Erectors may wish to field weld connections for temporary bracing, tie-offs, temporary stiffeners, etc. Field welding of such brackets, lugs, reinforcing bar ties, screed rails, etc. to permanent members should not be allowed unless approved by the Engineer of Record. High strength rod conforming to ASTM A722 is often used in temporary works for bracing or hangers. Welding of A722 rods can result in brittle fracture and should be prohibited. All welding must conform to owner requirements.

Girder temporary bracing connection plates or stiffeners required for erection should preferably be shop installed. When this is not possible, their field welding should conform to the Bridge Welding Code, AWS D1.5.

3.3.9 Concrete Girders

Various methods may be used to create continuous girder systems from precast concrete beams. Methods for coupling individual beams to create continuous girders include post-tensioning and coupling with either high-strength rods or prestressing strands. Drop-in spans are discussed in Section 3.2.3.3. Continuity of the girders does not take place until all bar splices, post-tensioning and concrete placement at joints is complete; thus any temporary works must remain until continuity is achieved. In some cases, staged post-tensioning may be used.

Temporary girder support and bracing must provide for proper alignment of splice bars or strands so that bar connections may be made. Alignment of ducts is also critical to avoid additional friction losses during tendon stressing.

SECTION 4. CRANES

3.4.1 Typical Crane Data

Typically, mobile cranes are utilized to erect and demolish bridge superstructures. During lifting, the crane is supported either by its tracks, for a crawler crane, or for truck cranes, on outriggers that extend out from the crane frame to provide discrete support points. For mobile cranes, the stability-limited rated load for a crawler crane is 75% of the tipping load. The stability-limited rated load for a mobile hydraulic or lattice-boom truck crane supported on outriggers is 85% of the tipping load. The American Society of Mechanical Engineers specifies these limits along with additional safety-related aspects of the crane design.

It is common for mobile cranes to be mounted on barges to facilitate erection of bridges over rivers or over large bodies of water (Figure 3-17). Typically, the published crane lifting capacity charts for barge mounted cranes are downrated 25% to 33% to account for the dynamic effect of barge vessel motion. Additionally, overall barge stability needs to be considered in the downrating, inasmuch as barges that allow greater pitch or yaw would require greater downrating of the cranes. Barge stability must be ensured for all crane operating conditions, and the barge deck evaluated for the crane loads and track pressures.



Figure 3-17 Crane Mounted on Barge

Newer cranes feature load moment indicators in plain view of the crane operator that display the crane boom radius, boom angle, rated load, actual load, wind speed and other data as well as displays with messages and alarms to alert the crane operator and prevent dangerous conditions from developing. Most new cranes are also equipped with automatic shutoffs if the load moment indicator is exceeded. The load moment indicator combines the effect of the load and its radius on the crane capacity.

The crane manufacturer publishes load charts for each crane. The charts dictate the allowable load that may be lifted based on the desired factors of safety compared to the radius of the boom of the crane. A sample load chart is shown in Figure 3-18 for a 350-ton capacity hydraulic truck crane. The top portion of the chart indicates that the chart covers boom lengths of 51 feet to 197 feet, the crane has 220,400 lbs. of counterweight, the outriggers are fully extended and set, and that the ratings listed are good for any pick geometry over a 360 degree swing of the crane. The lower portion of the chart is similar except the counterweight is 110,200 pounds.

load charts

 51-197 ft.
 (15.5-60.0 m)
  220,400 lb.
 (100 Tonnes)
  100%
 27' 11" Spread
  360°

 Pounds (thousands) 

Feet	51'	51	69	87	106	124	142	160	178	197
8	700.0†									
10	542.0	486.0	422.0	342.0						
15	412.0	370.0	366.0	342.0						
20	332.0	302.0	302.0	300.0	242.0	185.0	139.0			
25	286.0	254.0	258.0	254.0	242.0	185.0	139.0	108.0		
30	219.0	218.0	222.0	222.0	220.0	174.0	137.0	106.0	78.0	66.0
35	185.0	185.0	188.0	188.0	187.0	174.0	137.0	106.0	78.0	66.0
40			162.0	163.0	161.0	161.0	126.0	106.0	78.0	66.0
45			142.0	143.0	141.0	144.0	116.0	102.0	78.0	66.0
50			126.0	126.0	128.0	128.0	106.0	95.0	78.0	66.0
55			113.0	113.0	116.0	114.0	98.0	88.0	78.0	66.0
60			102.0	102.0	105.0	103.0	91.0	82.0	75.0	66.0
65			92.0	95.0	95.0	93.0	84.0	76.0	70.0	65.0
70			85.0	85.0	86.0	85.0	78.0	71.0	66.0	62.0
75			79.0	77.0	77.0	73.0	66.0	62.0	62.0	59.0
80			72.0	70.0	70.0	67.0	62.0	62.0	58.0	56.0
85			67.0	65.0	65.0	64.0	58.0	58.0	54.0	52.0
90			62.0	60.0	60.0	61.0	54.0	54.0	50.0	49.0
95				55.0	55.0	57.0	50.0	47.0	47.0	46.0
100				51.0	51.0	53.0	48.0	45.0	45.0	43.4
105				47.0	47.0	50.0	45.0	42.4	42.4	41.0
110					45.0	46.0	42.4	42.4	40.2	39.0
115						43.2	40.4	40.4	38.2	37.0
120							40.6	38.6	36.2	35.2
125								37.0	34.4	33.6
130								34.6	32.6	32.0
135								33.4	31.2	30.0
140								32.4	30.0	27.8
145									28.8	26.0
150									27.2	24.8
155									25.6	23.8
160									24.4	22.6
165										21.8
170										20.6
175										19.2
180										17.2

* Over rear only, 28'6" x 19'8" outrigger base. Lifting capacities greater than 425,000 lbs require additional equipment. † 700,00 lbs is a comparative rating

 51-197 ft.
 (15.5-60.0 m)
  110,200 lb.
 (50 Tonnes)
  100%
 27' 11" Spread
  360°

 Pounds (thousands) 

Feet	51'	69	87	106	124	142	160	178	197
10	448.0	422.0	342.0						
15	352.0	352.0	342.0	242.0					
20	278.0	282.0	282.0	242.0	185.0	139.0			
25	222.0	224.0	224.0	224.0	185.0	139.0	106.0		
30	180.0	183.0	183.0	182.0	182.0	139.0	106.0	78.0	66.0
35	147.0	150.0	151.0	155.0	150.0	137.0	106.0	78.0	66.0
40		129.0	131.0	130.0	126.0	118.0	106.0	78.0	66.0
45		111.0	113.0	112.0	108.0	102.0	97.0	78.0	66.0
50		95.0	97.0	97.0	94.0	94.0	87.0	78.0	66.0
55		81.0	83.0	83.0	82.0	83.0	79.0	73.0	66.0
60			73.0	72.0	75.0	73.0	70.0	68.0	66.0
65			64.0	63.0	66.0	64.0	64.0	63.0	59.0
70			57.0	59.0	59.0	57.0	58.0	56.0	53.0
75				54.0	53.0	52.0	52.0	51.0	48.0
80				48.0	48.0	48.0	47.0	46.0	43.0
85				43.8	42.8	43.8	42.6	41.2	39.0
90				39.6	38.8	39.6	38.4	37.0	35.4
95				36.2	36.0	36.0	34.8	33.4	31.8
100				33.0	32.8	31.6	30.2	28.6	28.6
105				30.0	29.8	28.8	27.4	25.6	25.6
110					27.6	27.4	26.2	24.8	23.2
115						25.0	23.8	22.6	20.8
120						23.0	21.8	20.4	18.8
125							19.8	18.6	16.8
130							18.2	16.8	15.0
135							16.6	15.2	13.4
140							15.0	13.8	12.0
145								12.4	10.6
150								11.2	9.4
155								10.0	8.2
160								9.0	7.0
165									6.0
170									5.0
175									4.2

Lifting capacities greater than 425,000 lbs require additional equipment. Note: Above chart is available with reduced outriggers

THIS CHART IS ONLY A GUIDE AND SHOULD NOT BE USED TO OPERATE THE CRANE.
 The individual crane's load chart, operating instructions and other instructional plates must be read and understood prior to operating the crane.

GMK 6350

Figure 3-18 Sample Load Chart for 350-Ton Capacity Crane

Figure 3-19 depicts a sample load chart for a 45-ton capacity all-terrain crane. Like the previous chart, this load chart identifies allowable load on the crane hook at various lift radii and boom configurations. The chart on the upper left is for 20 feet - 10 inches x 20 feet - 4 inches outrigger spacing, main boom lengths from 25.6 feet to 102.3 feet, and is good for any lift geometry over the 360 degree swing of the crane.

The chart in the lower left of Figure 3-19 rates the crane for a reduced outrigger configuration of 20 feet – 10 inches x 7 feet – 8 inches and limits the boom length to 64.0 feet when the crane is operating with the reduced outrigger spacing. In contrast, the chart on the upper right of Figure 3-19 rates the crane with no outriggers and a boom position of 0 degrees, which means that the chart is only good for lifts over the rear of the crane, and the boom length is limited to 44.6 feet

In selecting a crane and appropriately describing it on erection documents, it is important to be very specific. A given crane may be available in several configurations of boom types and lengths, counterweight configurations, tracks extended (or not), etc. These variations affect the crane lifting capacities, and track or outrigger loads. Maximum crane capacity can be controlled by boom structural capacity, as well as crane tipping. When determining the allowable lifted load, the weight of all rigging below the crane hook must be subtracted from the rated load capacity. Where large spreader beams are required, this can substantially reduce the allowable weight of the lifted girder.

Where multiple cranes are used to pick a single girder, or girder pair, (Figure 3-20), close operator control must be maintained to ensure load shifting does not take place, causing one crane to become overloaded.

The crane type and capacity selected for a given project depends upon many factors including the method of bridge field assembly, site access and the resulting radius to lift, turn, and set the members, and local crane availability. Mobilization and demobilization costs as well as project sequencing and staging must also be considered. Some large crawler cranes may require field assembly, even requiring use of a smaller crane to handle crane components.

General guidance on crane capacity may fall in the following ranges:

- Setting single girders or sections with weights in the 25 to 40 ton range might use conventional truck or crawler cranes with a main lifting capacity between 150 and 250 tons. To account for typical working radii, a 60 to 70 ton holding crane may also be needed. (Refer to SECTION 3-4.2 for a discussion of holding cranes.)
- For spliced segments or heavier girders in the 50 to 100 ton weight range, 300 to 550 ton capacity hydraulic cranes may be used. Smaller cranes may be sufficient if they can be positioned for minimal picking radius.

- For lifts of 100 to 200 tons such as large girders or girder assemblies, hydraulic cranes in the 500 to 650 ton capacity range or crawler cranes of 300 to 450 ton capacity might be used. These heavy lifts are often multiple crane lifts.

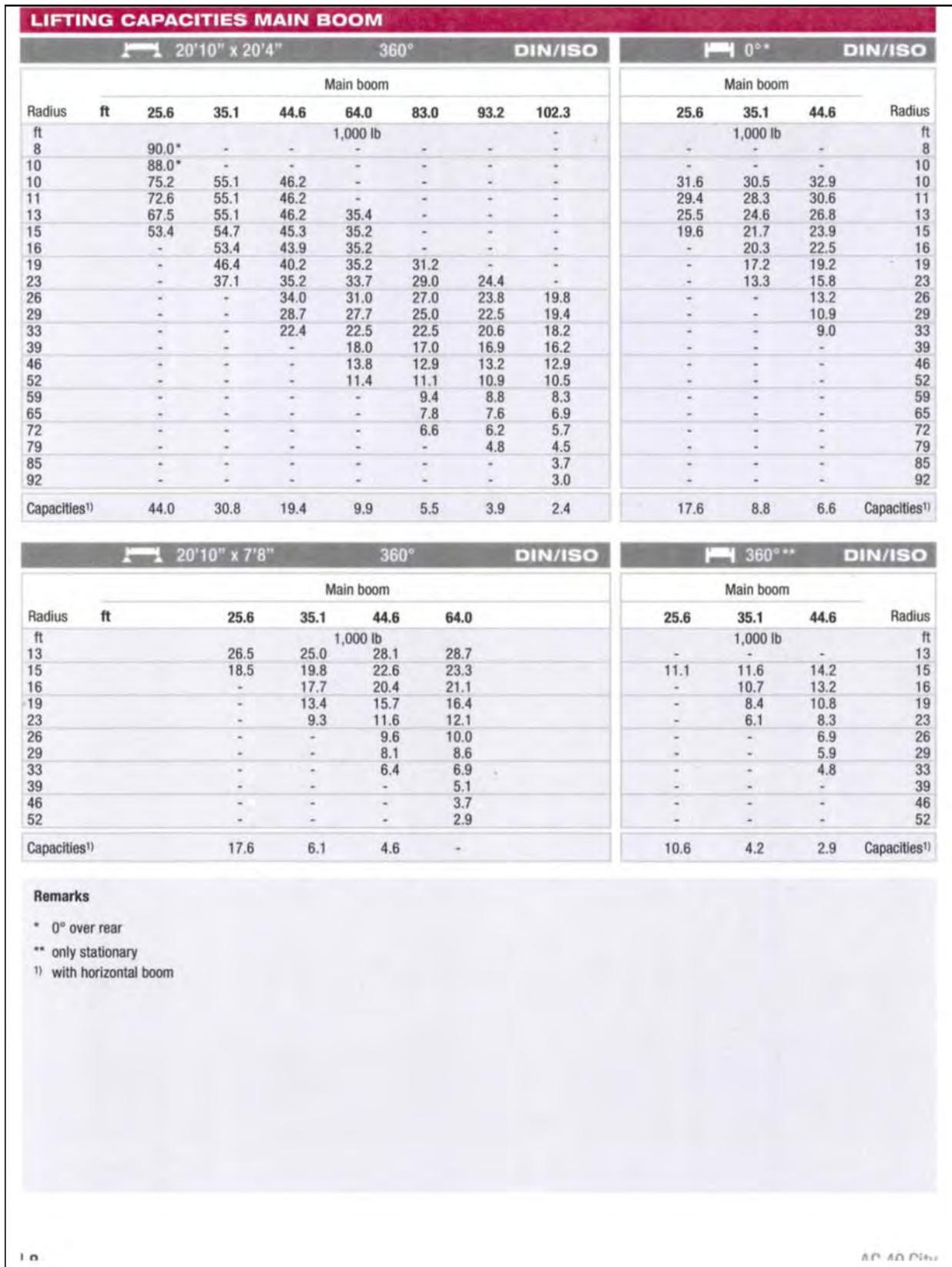


Figure 3-19 Sample Load Charts for 45-Ton Capacity Crane

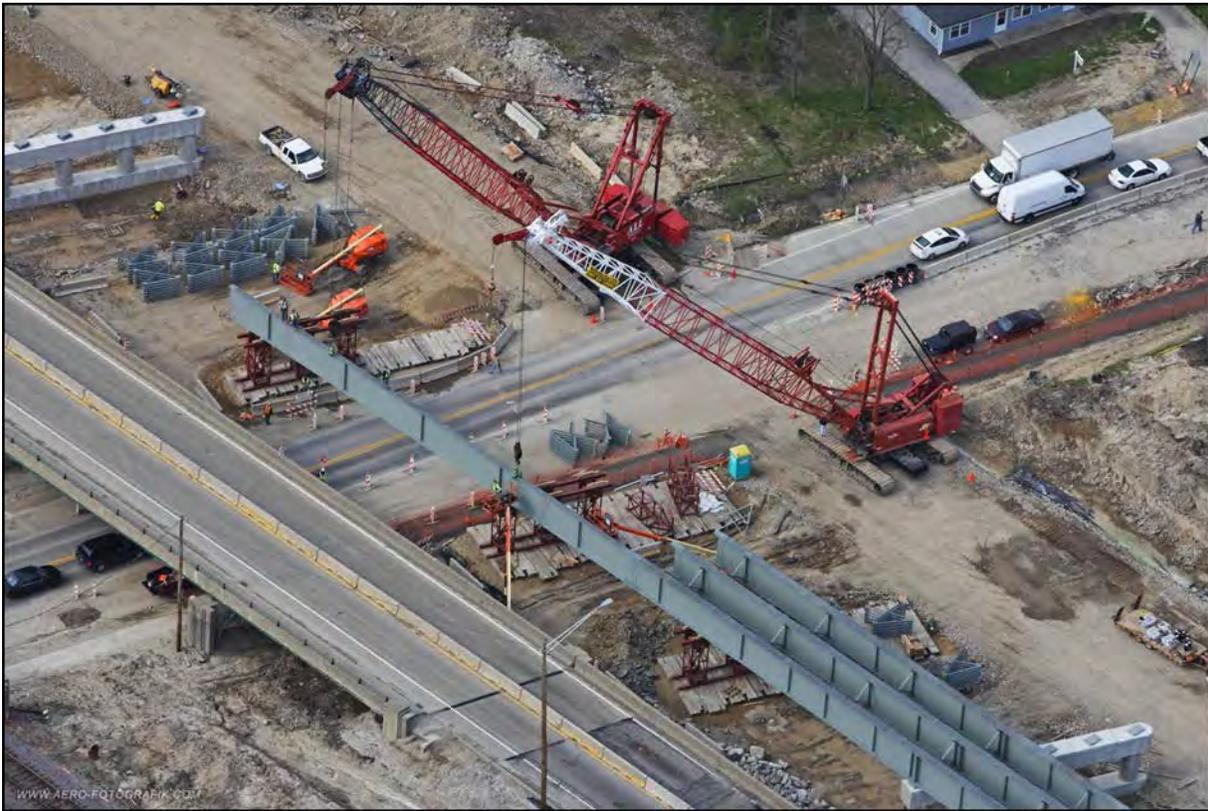


Figure 3-20 Multiple Cranes Lifting Girder – Aerial View

3.4.2 Holding Cranes

Holding cranes are often utilized to temporarily stabilize girders or control geometry during erection. The holding crane typically accepts a portion of the erected girder load while the main lifting crane releases the load in order to pick another girder segment for erection. Holding cranes are typically utilized when the use of temporary shoring is not feasible due to limitations from live rail or vehicular traffic lanes beneath the structure, waterways, and the like that would conflict with the desired placement of the temporary shoring.

Research by the FHWA/TXDOT/University of Texas at Austin, (Stith, Jason, et. al. 2010), studied various cases of curved girder bridges utilizing holding cranes during erection. These research results can also be applied to straight girders. Their research indicates that the optimum location for placing a holding crane is at the maximum positive moment location in the girder, which will maintain a web vertical and minimize displacements along the length of the girder, facilitating fit-up. Unlike shore towers, placing a holding crane at the location of maximum positive moment tends to give the lowest displacements and warping stresses of any location along the girder.

The research also showed that the load that the holding crane applies significantly affects the behavior of the girder being supported and recommends that the holding crane lift with a load equivalent to that of a rigid support reaction placed at the same location. The crane should be sized to enable it to provide the required lift capacity, accounting for its job site location relative to the girder being held.

Holding cranes are normally, and preferably, attached to the top flange of the girder, thus providing an upward force above the girder's center of gravity. Thus, if the girder rotates, a component of the holding force creates a restoring secondary moment that decreases deflections from those predicted by a linear analysis.

Because support for the girder provided by the holding crane is through the crane rigging which has minimal lateral stiffness, the holding crane does not provide a lateral or torsional brace point for the girder. However, by sizing the holding crane as though it were a rigid support, the bending moment within the supported girder is reduced. This is further discussed in Chapter 4.

3.4.3 Soil Pressures and Structures Influenced by Crane Location

Cranes may produce bearing pressures beneath tracks or outriggers of several thousand pounds per square foot, and the existing soil conditions must be evaluated for their allowable bearing capacity. The available soil bearing capacity can be determined using the AASHTO LRFD Specifications; however, the available bearing pressures should account for the short-term effect of the maximum loads. When computed bearing pressure exceeds that available, compacted stone pads or timber mats are commonly used to distribute the loads. Crane loading data is available from crane suppliers, or can be calculated from crane dimension and weight data. The book *Cranes and Derricks* by Howard I. Shapiro, et al., provides calculation guidance.

The cost of renting and operating a crane is directly proportional to the lifting capacity of the crane. In order to minimize the expense associated with crane capacity it is generally desired to have the crane as close as possible to the girder segment that is being set. This results in cranes being set up or operated directly adjacent to abutments, and wingwalls, as well as atop buried structures. The surcharge load caused by the crane self-weight and the lifted load weight on these portions of the structure are often large and well in excess of the anticipated loads accounted for during the design of these components. All structures within the zone of influence of an outrigger or the tracks of a crawler need to be evaluated to ensure adequate capacity prior to approving a crane position. Additionally, when tracks or outriggers are placed near a slope or excavation, the resulting load effects on stability and bearing capacity must be investigated.

SECTION 5. BRACING

3.5.1 Girder Bracing

Bracing is used to perform various functions in bridge superstructures. Bracing normally consists of cross-frames or diaphragms between girders and lower, and possibly upper, lateral bracing systems as shown in Figure 3-21. Cross-frames may be composed of angles, tees, or channel sections and are generally shipped to the bridge site as fabricated assemblies with high strength bolted connections to the girders. Diaphragms may be fabricated from channels, wide flange sections or bent plates, and are field bolted to the girders. Cross-frames and diaphragms provide vertical load distribution between girders, serve to laterally brace girders for stability prior to casting the deck, laterally brace the compression flange in negative moment regions, and provide transfer of lateral loads to piers and abutments.

Lateral bracing systems (Figure 3-21) provide a truss system to carry lateral loads to the piers and abutments. Figure 3-22 shows the distribution of wind loads between the concrete deck and lower lateral bracing system in the completed bridge. Without the deck or lateral bracing, the lateral loads produce weak axis bending of the girder flanges. These bracing systems also help prevent relative lateral movement of the girders, and provide added control of bridge geometry during construction. This is particularly important in curved girder spans.

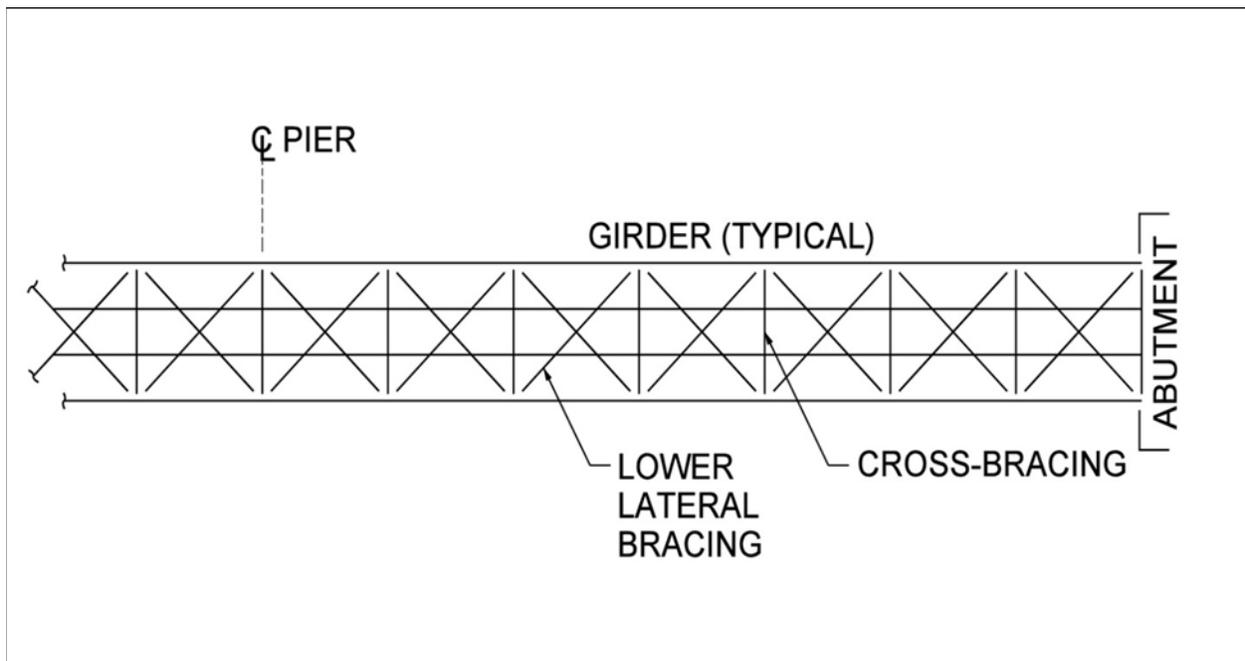


Figure 3-21 Lower Lateral Bracing

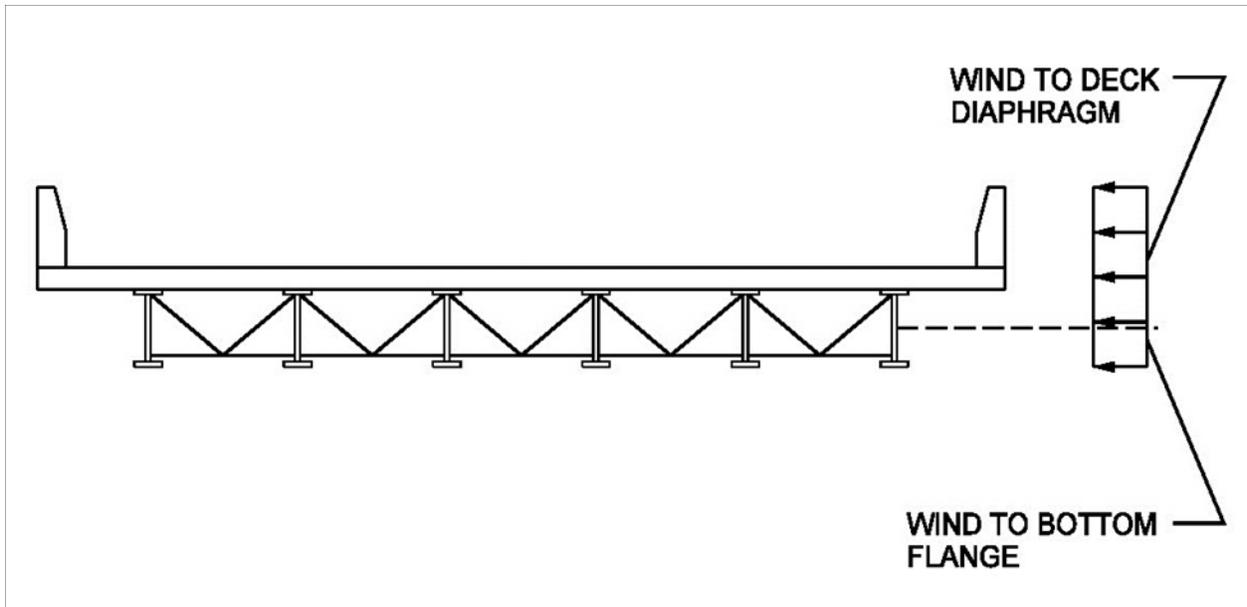


Figure 3-22 Wind Load Distribution



Figure 3-23 Box Girder Top Lateral Bracing and Diaphragms

Cross-frames or diaphragms used in box section members, as shown in Figure 3-23, perform much the same functions as in I-girder spans. Stability and geometric control during erection and casting of the deck are dependent on proper bracing design. Lateral top flange bracing should be provided to prevent flange buckling during deck placement, as well as control deformations. At least partial length bracing should be provided in straight box girders, and full length in curved box girders. External cross-frames or diaphragms between boxes may be needed to maintain proper deck geometry and thickness during deck placement, due to relative displacements resulting from girder twist.

The erection plan must consider the sequence and extent of cross-frame / diaphragm and lateral bracing installation required for each stage of erection. This erection sequence will also determine the need for any temporary bracing. The load and stiffness demands on permanent bracing may differ under erection conditions from that used for the permanent bridge design, and should be verified.

Erection schedules may limit the time available for field activities, particularly when traffic flow may need to be stopped to allow overhead operations. The erector may want to reduce the extent of bracing initially placed, or the completion of bracing connections, completing the installation at a later time. Such practices require careful evaluation and control, if the bridge's stability is to be maintained.

Temporary bracing may be required due to construction operations, such as stabilizing girders at piers or abutments, bracing of girders to an adjacent structure during widening projects, transferring loads from deck overhang brackets, and other purposes. The erection engineer must properly design any such bracing considering strength and stiffness requirements, as well as special requirements, such as allowing controlled vertical movement of adjacent members.

The bridge erector must ensure that girders are adequately braced at all times. Erectors should closely monitor weather conditions and have contingency plans and materials in place so that in the event of unforeseen wind activity or storms added bracing or hold-downs can be quickly placed. A review of bracing requirements should be conducted each day prior to leaving the site.

3.5.2 Temporary Bracing and Hold Downs

Temporary bracing is normally required to provide stability for girders, at least until the permanent bracing or diaphragms are installed. Bracing at girder ends and piers is needed to provide global stability to the girder and restraint to girder twist.

Hold down systems may be needed to restrain girders, and temporary restraint is needed to eliminate movement at rotational and sliding bearings.

Designs for these temporary items can take many forms depending on details of the bridge design, erector preferences, erection engineer experience and owner

requirements. Temporary bracing and hold-downs should be located so that they do not interfere with installing the permanent bracing or cast-in-place concrete diaphragms. Several methods of providing this temporary bracing are shown in the following subsections.

3.5.2.1 Chain Down to Standard Bearings

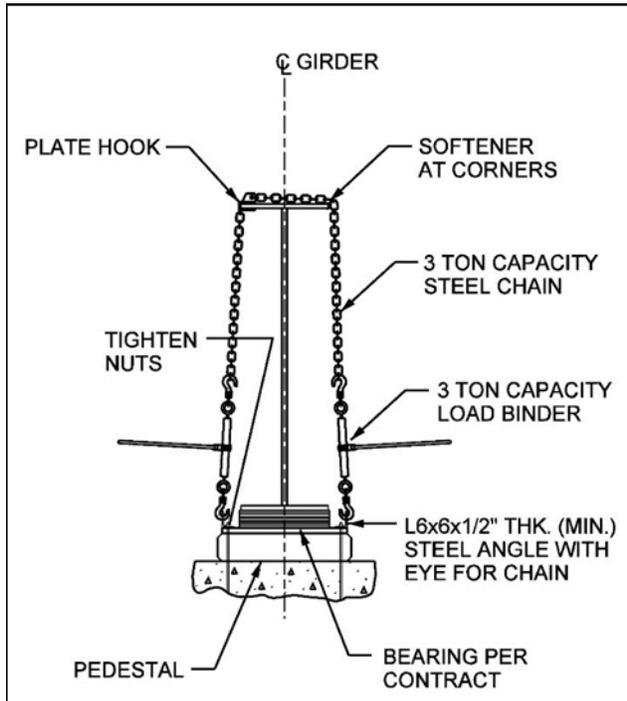


Figure 3-24 Steel Girder Tie Down Using Chain Binders to Pier

One method for holding girders in position at piers and abutments on common fixed or expansion elastomeric bearings is to chain the girders to the bearings themselves. Two details to achieve this are illustrated in Figure 3-24 and Figure 3-25.

The first detail, Figure 3-24 binds the girder to the bearing through the use of chains and load binders and incorporates the anchor bolts for the bearing. It is also common to use temporary anchor bolts embedded in the top of the pier cap or abutment seat in lieu of the bearing anchor bolts. This detail is common in situations where the girder is still supported by the crane and is in the process of having cross-frames connected between the girder and the adjacent previously erected girder line.

The second detail, Figure 3-25, is commonly used when erecting the first single girder where the girder will be subjected to wind loads.

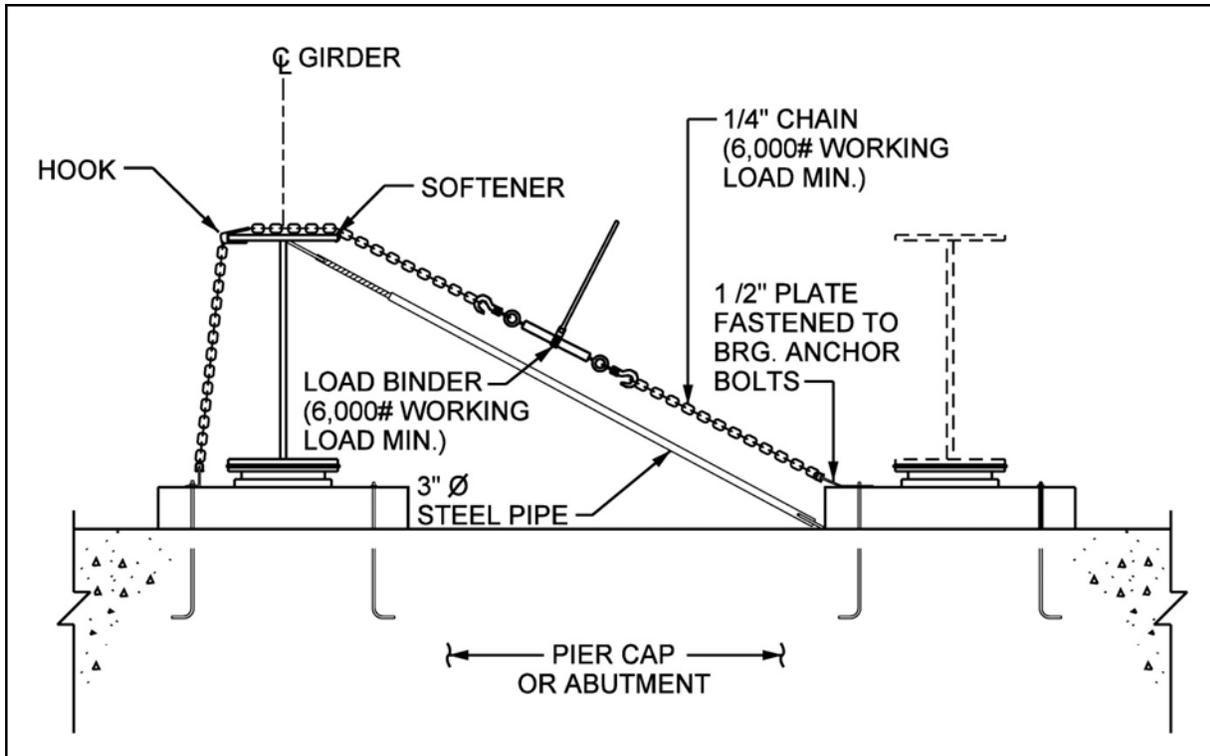


Figure 3-25 Steel Girder Tie Down Using Strut and Chain Binders

The chain and load binder will prevent the girder from rolling over when subjected to wind loads from the right, while the pipe strut will handle wind loads from the left. This detail also restrains the girder from twisting.

Figure 3-26, which is based on a Commonwealth of Pennsylvania Department of Transportation bracing detail shows a variation of the arrangement suitable for concrete girders utilizing threaded rods in place of the chains. The top channel member provides load distribution across the flange rather than at the flange tips, as would happen if chain were used on the bulb tee. Note the use of neoprene pads for load distribution.

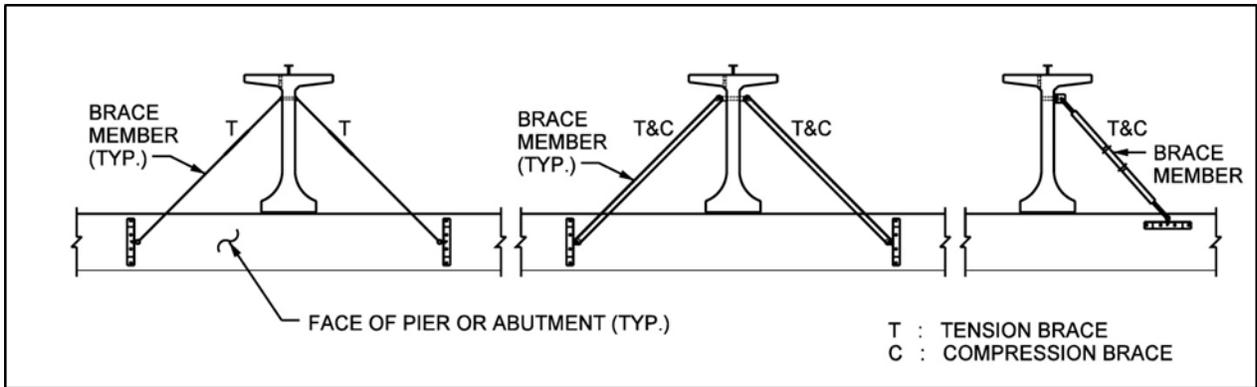


Figure 3-27 Typical Concrete Girder Bracing Arrangements at Pier or Abutment

Figures 3-28 through 3-31 show concrete girder bracing details based on Texas Department of Transportation standards. Figure 3-32 shows timber X-bracing and top flange lateral struts for I-girders at the abutment. Many bracing systems used for concrete girders utilize timber members as compression elements. It is important that these be secured by wedging or connections to adjacent members to preclude loosening and displacement due to construction and wind induced vibrations.

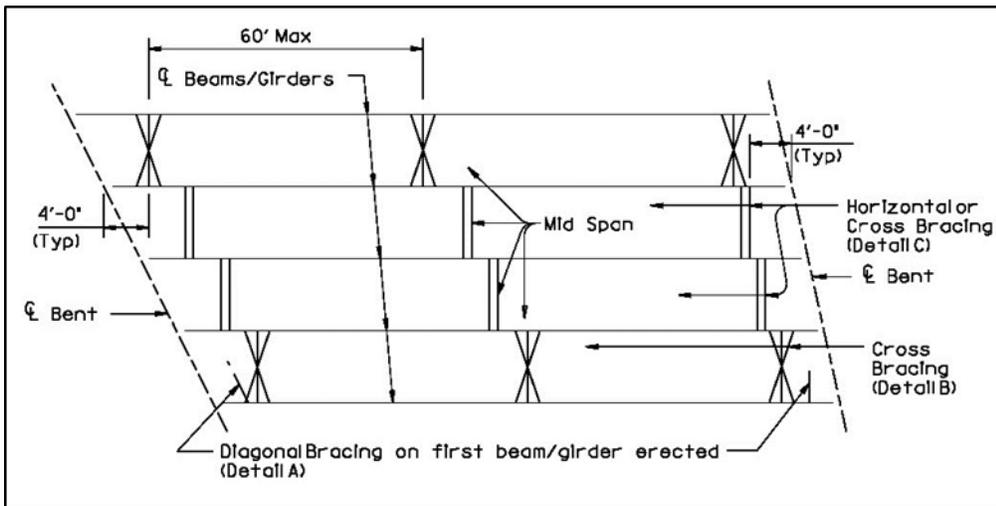


Figure 3-28 Concrete Girder Erection Bracing – Example Plan

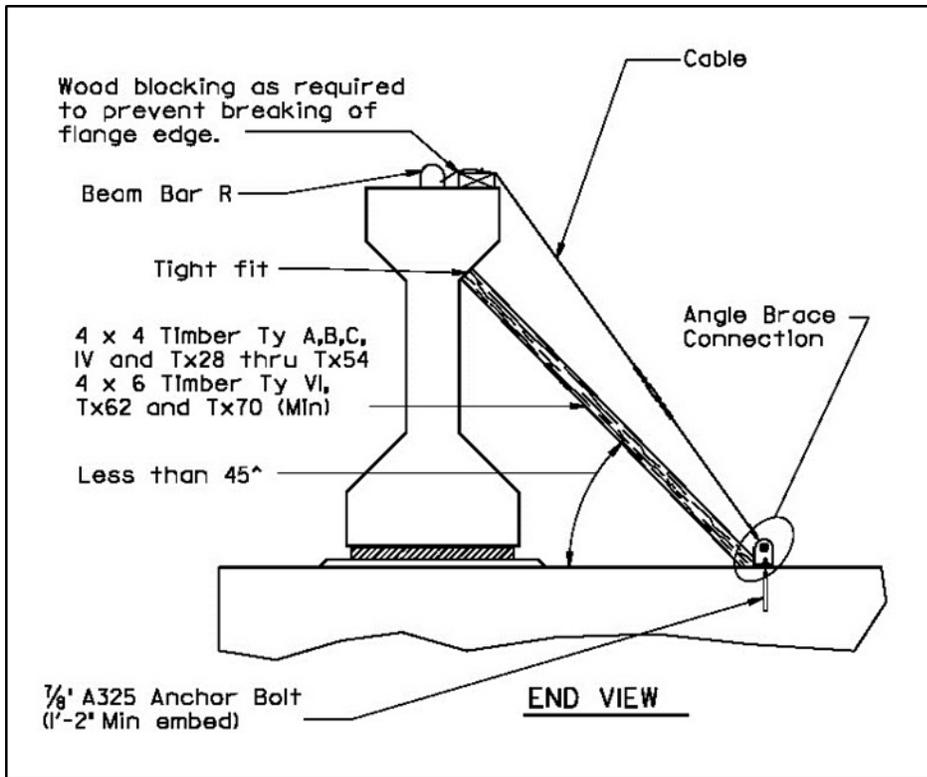


Figure 3-29 Concrete Girder Erection Bracing Detail at Abutment or Pier Using Cable and Timber Strut

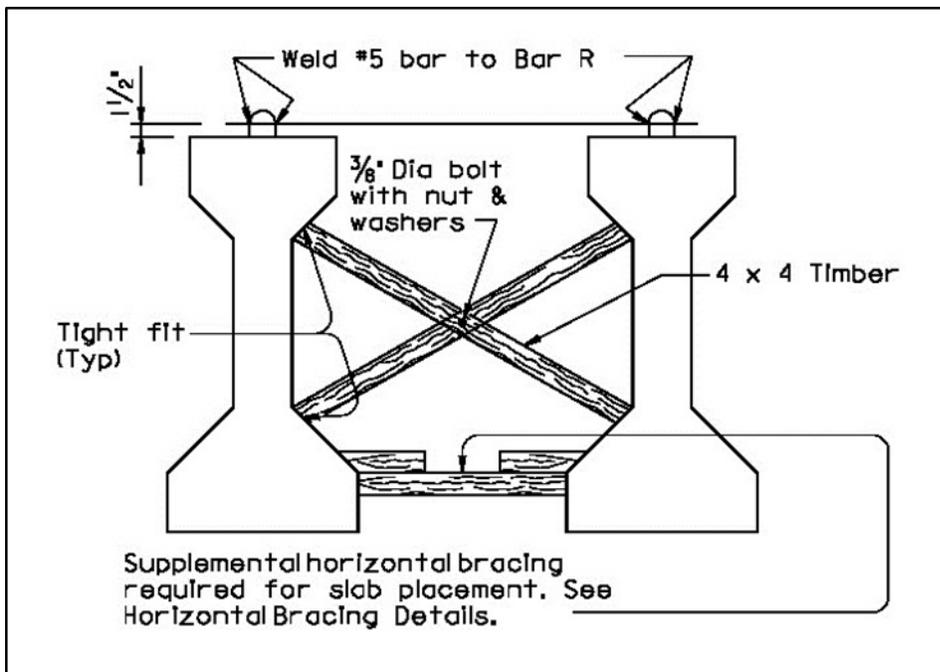


Figure 3-30 Concrete Girder Erection Bracing Detail between Girders Using Timbers and Top Tie Bar

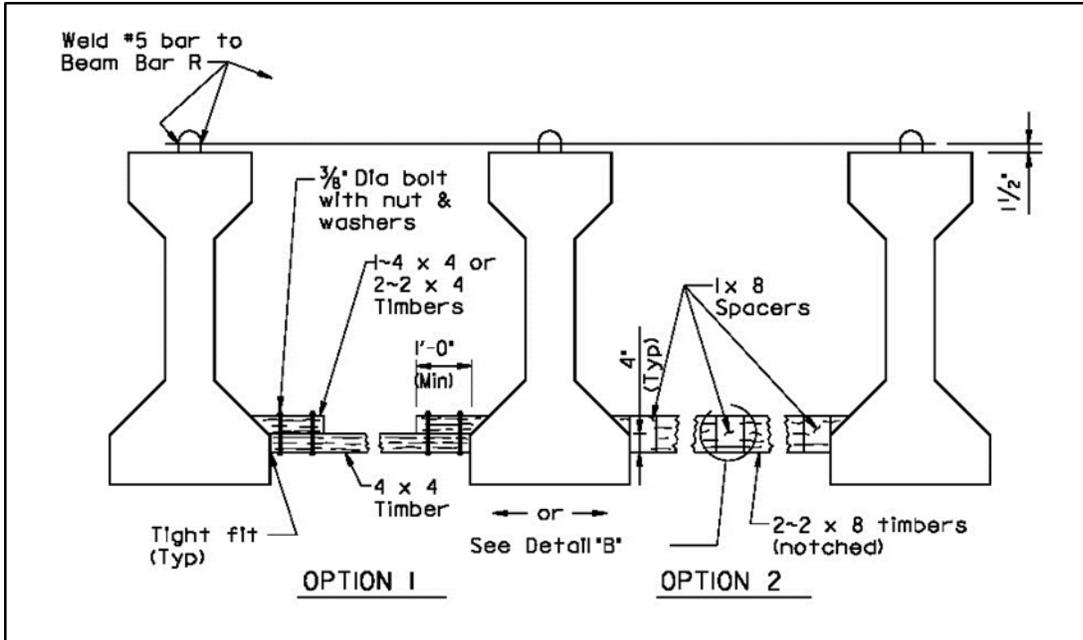


Figure 3-31 Concrete Girder Erection Bracing Detail between Girders Using Timbers and Tie Rod



Figure 3-32 Timber X-Bracing for Concrete I-girders

3.5.2.2 Erecting on Multi-Rotational Bearings

When erecting girders on multi-rotational bearings, the bearing must be restrained so the girder does not roll. Wooden or steel shims can be driven between the load plate and the masonry plate (cylinder plate) on all sides of the bearing to prevent unwanted displacements of the girder at the bearings. The girder itself would be chained down in a manner similar to the elastomeric bearing examples.

A method of locking a guided high load multi-rotational (HLMR) bearing, after a Pennsylvania DOT standard detail, is shown in Figure 3-33. The locking assembly is placed prior to setting the girder, which is then secured with local bracing.

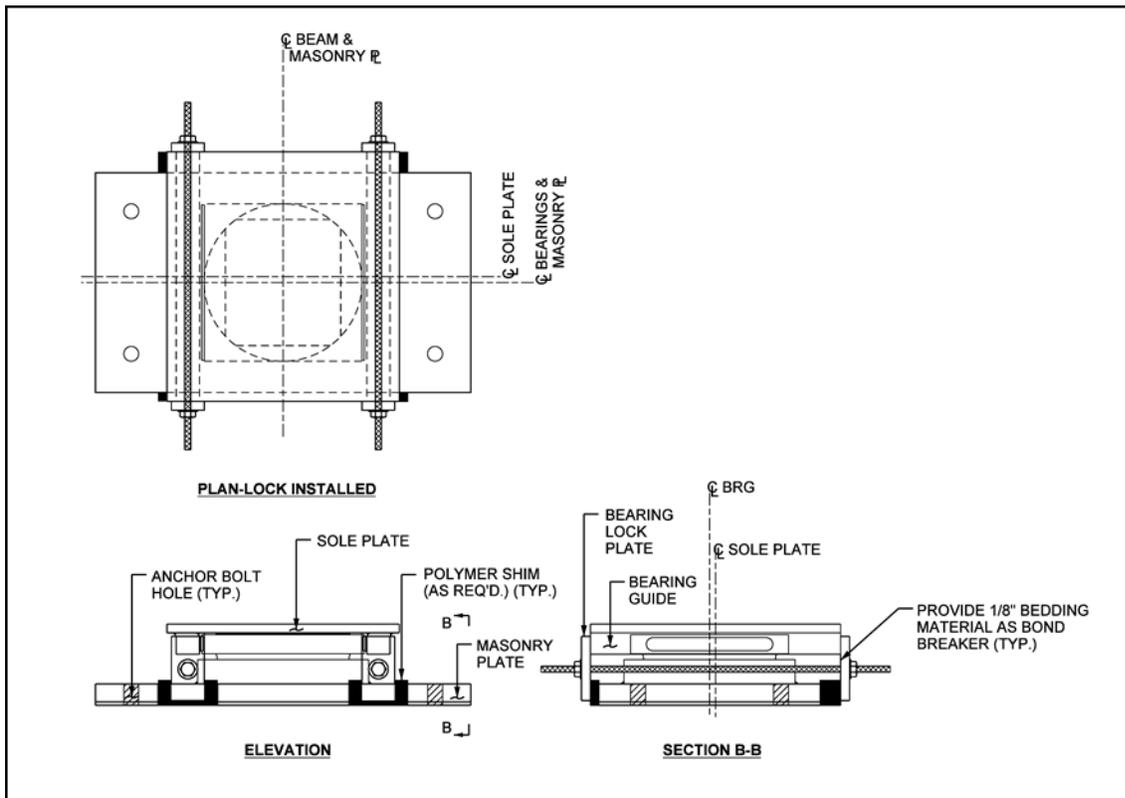


Figure 3-33 Conceptual Guided HLMR Bearing Lock

3.5.2.3 Bracing to Adjacent Structures – Phased Construction

Many bridge replacement projects are conducted without detour routes when sufficient width is available. This may require that the first new girder erected adjacent to the existing bridge be braced to that bridge. Inasmuch as the existing bridge is subject to vehicular loading, the existing structure is subjected to vibrations and deflections that the new girder is not subjected to. Special attention should be given to providing details that will allow differential deflection between the existing structure and the new structure as well as engineering connection details that will not be susceptible to vibration.

Figure 3-34 illustrates a possible connection that meets the criteria. This detail allows vertical movement while providing a lateral brace point along the new girder flange in positive moment regions. These temporary braces must be located to preclude instability of the new girder. Bracing at the abutment or pier may be as shown in Figure 3-27.

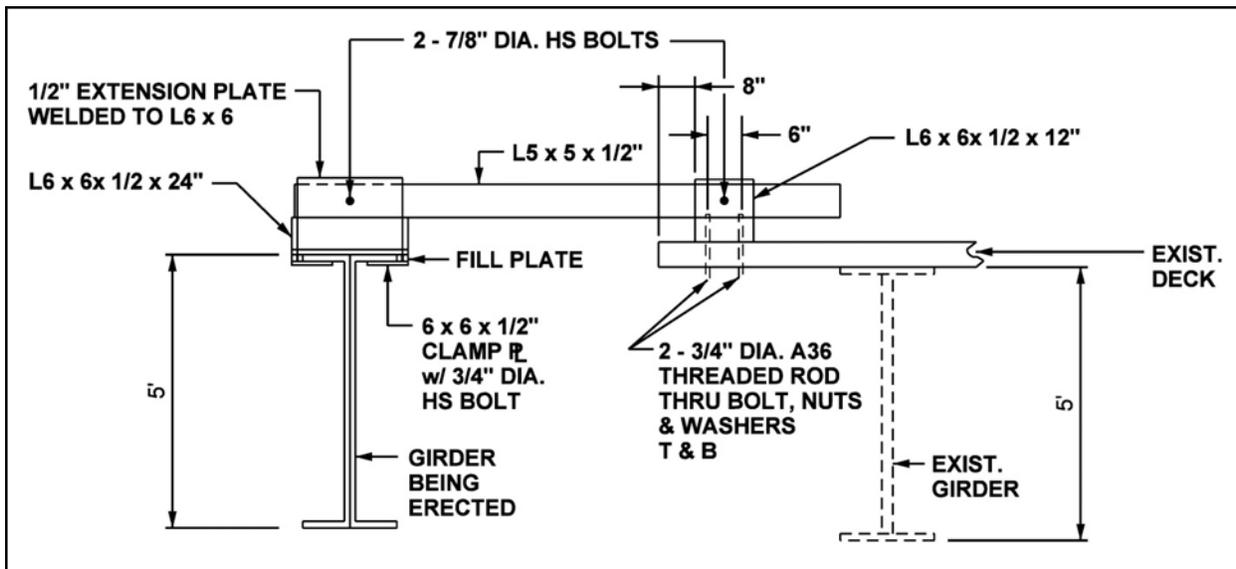


Figure 3-34 Temporary Brace to Existing Bridge Deck During Phased Construction

3.5.2.4 Uplift Hold-Down

Unbalanced load conditions while setting girders or casting the deck can cause reduced pressure from the girders on the bearings. This not only can cause alignment issues with the bearings, but also reduce the lateral restraint to the girder bottom flange, or allow sliding. A typical hold-down detail is shown in Figure 3-35. The rod can be preloaded to provide a net compression to the bearings when the uplift is present. For concrete girders, the channels will bear on the top slope of the bottom flange, which may require installation of a horizontal tension rod through the lower webs or similar tension tie configuration to prevent spreading of the girders due to the horizontal component of the vertical load.

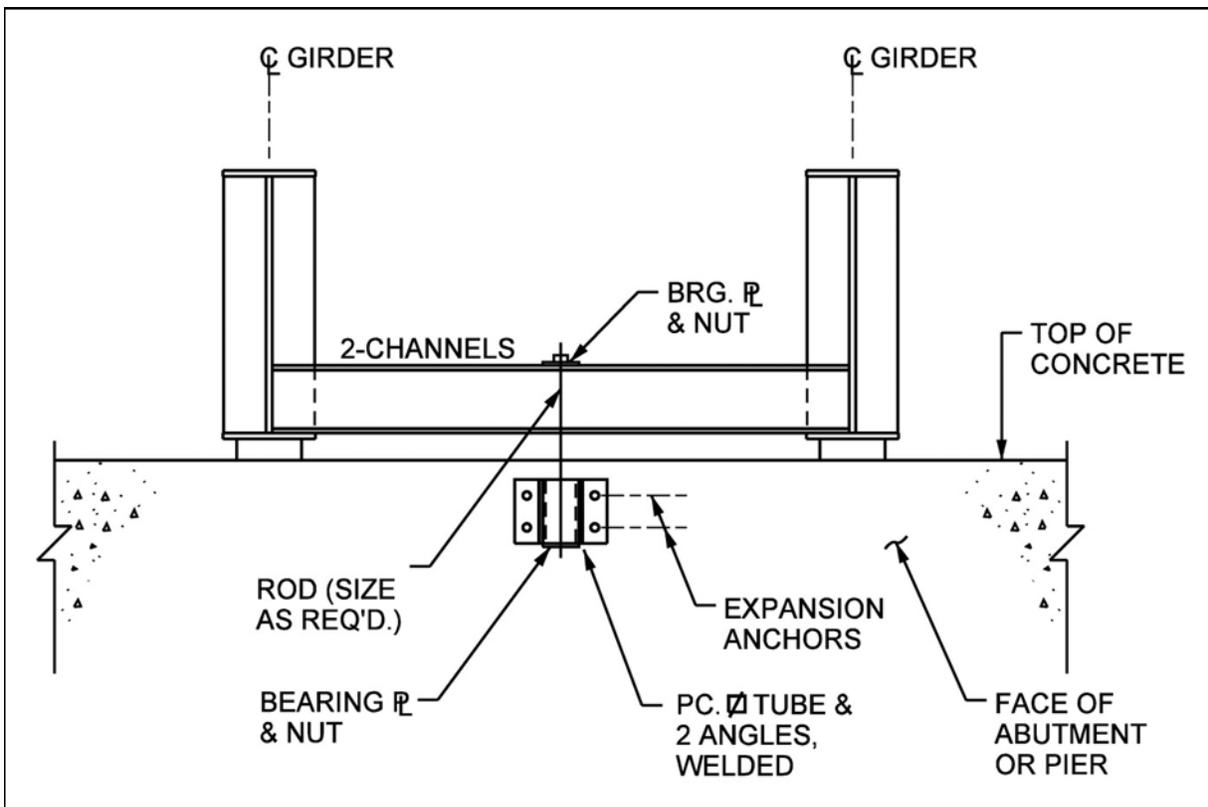


Figure 3-35 Hold-Down Assembly for Steel Girders

SECTION 6. SHORING TOWERS

Shoring towers, such as those shown in Figure 3-36, are primarily used to control deformations and stresses in the girders during erection. They may also be used to temporarily support concrete girder segments prior to those segments being made into a continuous girder. The location of a shoring tower affects the behavior of the girders as well as required design forces on the shoring tower. The specific location where a shoring tower can be placed, however, may be affected by many nonstructural restrictions such as site access, construction methodology, and girder stiffness variations.

It is recommended that shoring towers be placed at locations where the maximum positive bending moment occurs between permanent supports. This positioning will have the greatest effect on minimizing the total deflections, and in general, it places a tower near the position where it will be required to support the least load of any position along the girder. It should be noted that adding a shoring tower results in a large concentrated force and it is thus advisable for steel girders to place the shoring tower under a stiffer location, or alternatively, to check local yielding and stability of the girder web at the shoring tower location.



Figure 3-36 Shoring Towers Supporting Girders during Erection

Shoring tower loads should be computed for the various erection stages to ensure the maximum loads are accounted for. The design of shoring towers should be in

accordance with the AASHTO Guide Design Specifications for Bridge Temporary Works.

Towers have to be designed to resist the bracing force imparted by externally applied lateral forces, as well as the force imparted by the bracing member to the tower. Towers will also deflect at the top and may rotate, both of which will lead to the reduction of restraining forces, as shown in Figure 3-37. To mitigate this, this deflection needs to be accounted for in the bracing scheme, and the towers need to be designed to be stiff enough to be able to develop the required restraining force. Tower deflections should be monitored at the time of load application (member release) and periodically thereafter. Observation of any unanticipated or unusual deflections should be cause to immediately stop work until the cause of the deflections is resolved and any remedial measures completed.

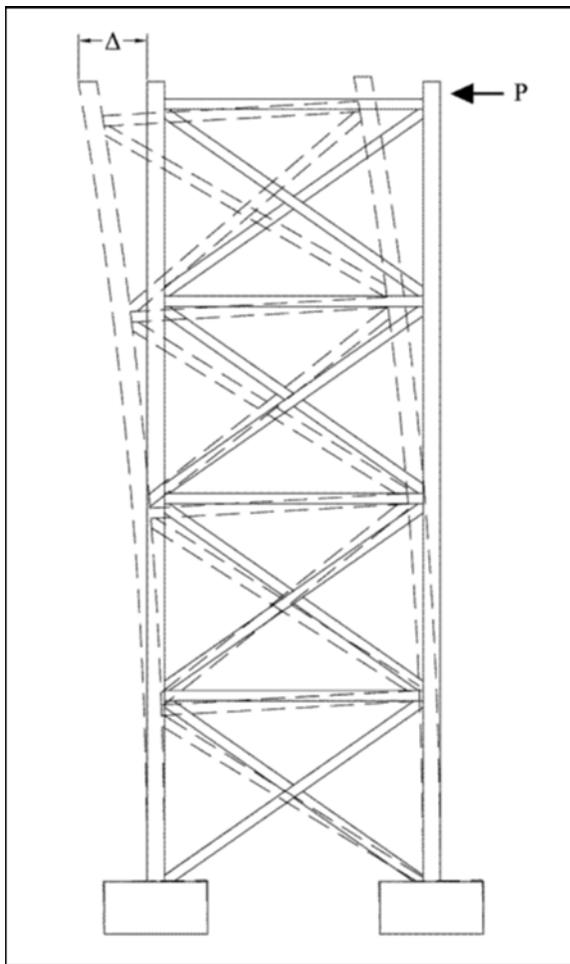


Figure 3-37 Tower Flexibility Effects Under Lateral Load

In order to ensure that towers are installed to the required elevations, both tower shortening and settlement should be evaluated. The AASHTO LRFD Bridge Construction Specifications, Article 11.8.5.2, requires top of tower elevations to be set so that they support the girders at their cambered no-load elevations. Hydraulic jacks are sometimes installed at the tops of towers to allow for adjustment in girder elevation as erection progresses. The shoring towers are normally released prior to placing the concrete deck, and a sequence for releasing should be determined. The method of release, whether by shoring tower screw jacks, hydraulic jack systems, sand jacks, or other means should provide for gradual and controlled release of loads from the shoring system.

Temporary shoring towers are often placed on timber mats or precast slabs set at grade. For poor soils or very heavy loads, cast-in-place concrete footings or pile bents may be required. Figure 3-38 shows shoring towers supported on timber mats carrying spliced bulb-tee sections.



Figure 3-38 Shoring Towers Supported on Timber Mats

SECTION 7. DECK CONSTRUCTION

3.7.1 Placing Sequence

In continuous girder bridges, a concrete deck placing sequence is normally provided in the design plans, or addressed in the owner's specifications. Typically, the positive moment regions of the deck are placed first, followed by the negative moment regions, in an effort to minimize deck cracking in the negative moment region. This results in a discontinuous deck placement, generally not favored by contractors. Contractors will often develop an alternate placing sequence, which starts at one end of the bridge and moves across. Aside from issues such as deck elevation control and concrete cracking / mix control, placing the deck in this manner can create uplift at piers or abutments that must be addressed in the erection plan. Uplift is most pronounced where a short end span is followed by a longer interior span, with maximum uplift occurring as concrete placement reaches mid-span of the longer span. Hold-down assemblies as discussed in Article 3.5.2.5 may be needed.

Skewed supports and/or curvature can complicate the deck-casting sequence. Keeping the deck placement reasonably symmetrical laterally minimizes eccentric or unbalanced loading and helps reduce differential deflections between adjacent girders. On skewed bridges where the anticipated differential deflections between girders are reasonably small, it is preferable to set up the finishing machine normal to the girders as it reduces the length of the machine. However, in cases with severe skews that may lead to large differential deflections, it may be preferable to set up the finishing machine to align with the bridge's skew. With this arrangement, as the finishing machine progresses along the bridge, each girder carries an equal concrete load with resulting similar deflection along its length.

In wide structures that have multiple girders and/or with severe skews, multiple deck placements may have to be made in the lateral direction which will result in the introduction of longitudinal construction joints.



Figure 3-39 Deck Finishing Equipment for Deck Pour

3.7.2 Overhang Brackets

Almost all steel and concrete bridges include a deck overhang beyond the fascia girder. Overhang brackets are used to support the overhang concrete loads, and often the loads from the finishing machine. Figure 3-39 shows a typical finishing machine, which runs on screed rails set atop the overhang forms or fascia girder. Figure 3-40 and Figure 3-41 show typical overhang brackets for steel and concrete girders, respectively. Overhang brackets are manufactured items for which dimensional and load capacity data are available from the manufacturer. Typical bracket spacing is approximately 4 feet. Figure 3-42 shows brackets being set on a tub-girder.

AASHTO LRFD Article 6.10.3.1 requires that the effect of forces from deck overhang brackets acting on the fascia girders be considered by the bridge design engineer. The magnitude and application of the overhang loads assumed in the design should be shown on the contract documents. Loads from the contractor's selected overhang brackets should be compared to these loads to determine if further analysis is required.

The eccentricity of the deck weight and other loads acting on the overhang brackets creates torsional moments on the fascia girders. On curved bridges, overhang bracket loads are particularly critical for the girder on the outside of the curve. As the loads are applied to the brackets, the top flange is pulled outward causing flange lateral moments of the same type as the flange lateral moments due to curvature in regions of positive flexure. The opposite is true for the girder on the inside of the curve.

The girder top flange must have sufficient capacity to resist these lateral overhang bracket loads acting in combination with the vertical loads resulting from the deck casting. The bracket should be sized so that the reaction from the bracket's diagonal strut is applied to the girder just above the bottom flange. Placing the bottom of the overhang bracket above the base of the girder web may result in web overstresses and distortion in areas between cross-frames or transverse stiffness. Temporary bracing may be needed to control these deformations. This may especially be a concern on deep steel girders due to the depth limits of available brackets.

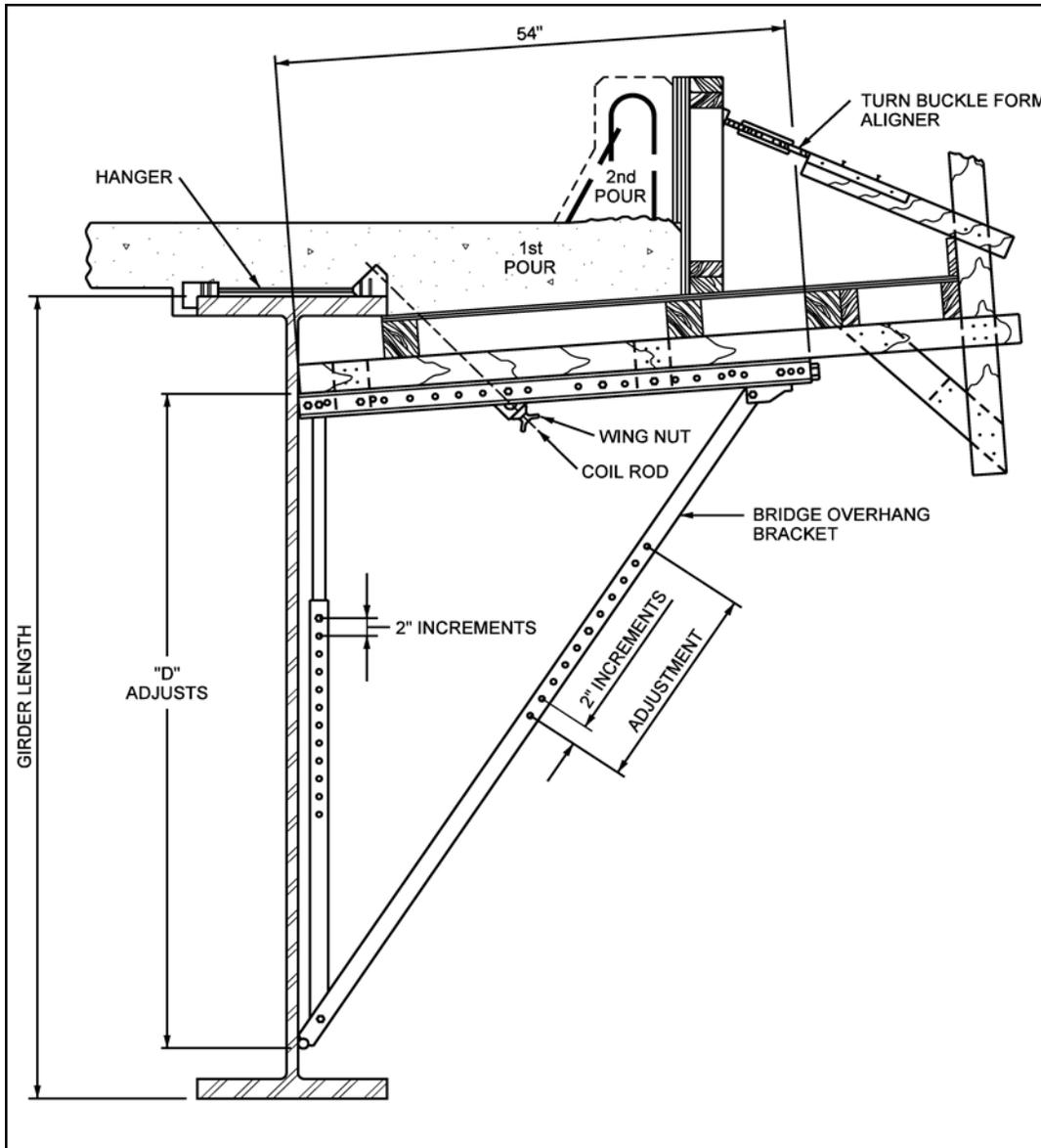


Figure 3-40 Typical Overhang Bracket – Steel Girder

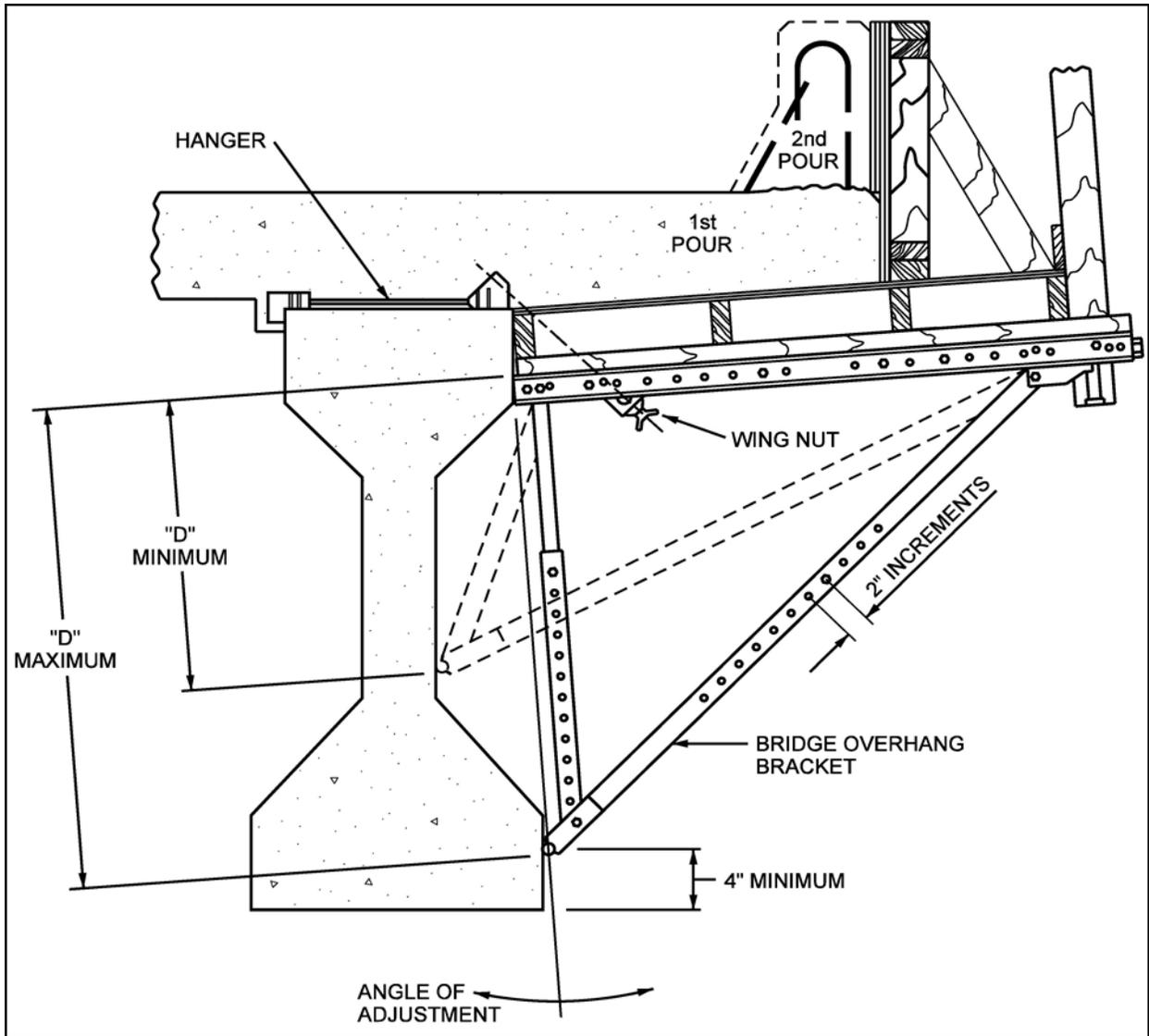


Figure 3-41 Typical Overhang Bracket – Concrete Girder



Figure 3-42 Overhang Brackets Being Set on a Tub Girder

Torsional loads can be resisted by the use of ties, usually reinforcing bars placed above the top flange, and timber compression struts located at the bottom flange. The computer design tool TAEG (Torsional Analysis for Exterior Girder), developed by the University of Kansas for the Kansas Department of Transportation (KDOT), is a public domain program, available from KDOT, that can be used to evaluate these systems including girder and cross-frame forces.

Figure 3-43 illustrates examples of temporary bracing used to control the local effects of the overhang brackets. When possible, the tie rods should run the full width of the bridge. Tie bars attached to U-stirrups of precast girders must be attached near the bottom of the U-stirrups to limit stirrup lateral deformations. Timber blocking can be carried across the bridge width, but often an angled wood brace runs from the lower flange of the outside, or perhaps first inside girder, to the underside of the top flange of the next girder to the inside. All blocking must be made tight by wedging, and be secured against loosening due to relative movements between the girders, vibration, or accidental impacts.

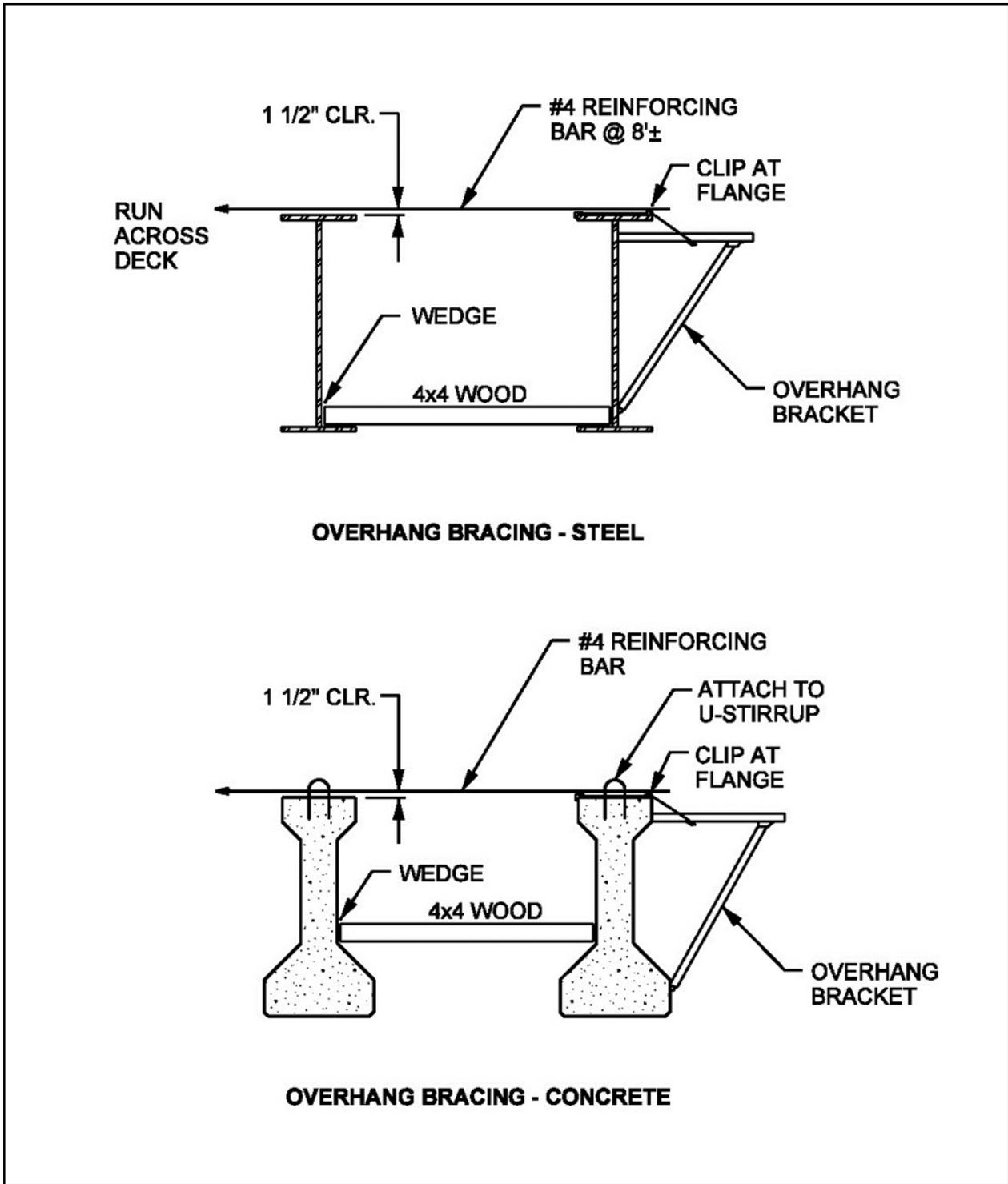


Figure 3-43 Overhang Bracket Bracing Examples

SECTION 8. BRIDGE DEMOLITION

Demolition of existing bridges may include full bridge removal to allow for new construction, or partial removal for widening, refurbishment, redecking, or similar activities. When concrete bridge decks are removed, composite action of the deck and girders is lost and lateral support for girders is reduced or eliminated. Thus, the girder capacity alone must be sufficient to prevent overstress or instability under these configurations. In some cases, temporary lateral bracing or supports were used during bridge erection; reinstallation of similar measures may be required during bridge demolition.

While of less concern where bridges are to be fully demolished, demolition activities for bridges being redecked or with girders otherwise reused should not cause girder damage that could adversely affect capacity. Concrete decks are normally removed either by demolishing the deck with hydraulic breakers with the debris retained by a temporary work deck supported on the girder lower flanges, or by saw cutting into large pieces, as shown in Figure 3-44, which are then lifted off using cranes, back-hoes or similar equipment. As the left girder in Figure 3-44 shows, when large hydraulic breakers are used, girder flanges may suffer impact damage causing deep dents, flange twisting, flange tears, and other defects that can reduce girder capacity. In removing decks in slabs, saw cuts are made both parallel and perpendicular to the girders to yield slabs of such size that they can be readily removed. If the depth of cut of the saws is not well controlled, girder flanges may experience cuts that can significantly reduce girder capacity. For redecking projects, it is recommended that the girder top flange location be marked on the deck and hammer sizes limited in order to minimize any girder damage.

Girder removal normally follows a sequence that is the reverse of erection. Unless girders are to be reclaimed for reuse, members are separated into sizes convenient for lifting by flame cutting rather than unbolting connections. Stability must be maintained at each stage of removal.

In determining crane capacities for demolition, consideration must be given to increased girder weight due to concrete that remains attached. In addition, an allowance should be made for the possibility of girders hanging up during removal due to corrosion, possible misalignment remaining from installation, possible structure movements over time, and similar factors. Thus, cranes should be sized using generous allowances for unknowns.

As seen in Figure 3-44, demolition is often performed by equipment located on the bridge. The effects of these loads must be evaluated and account for equipment loads as well as changes in girder capacity as demolition proceeds. These activities may create uplift at supports, and girder behavior may change from continuous to simple span or cantilever depending on stages in removal.

In removing decks from post-tensioned concrete girder bridges, evaluation should consider whether post-tensioning was applied before or after the deck was cast. If the deck was placed prior to stressing, the bare beams may not have sufficient capacity to resist the prestressing force when no longer composite, possibly resulting in girder cracking.



Figure 3-44 Bridge Deck Demolition Removing Deck in Sections

SECTION 9. SUMMARY

Erection schemes for girder bridges, though well developed, require evaluation considering the specific requirements of each structure. Those responsible for the design of erection methods and sequencing must be knowledgeable in how construction practices affect the bridge superstructure at each stage of erection. In presenting an overview of construction practices and noting the relationship of these to girder erection and stability, this chapter provides a background to the girder design and stability discussions presented in the following chapters.

CHAPTER 4

STABILITY FUNDAMENTALS

SECTION 1. INTRODUCTION

Much of the analysis that is performed on steel and concrete bridge systems assumes that the structure is geometrically stable and that the deformations of the structure will not have a significant impact on the geometry of the structural components. However, depending on the configuration of the structure and the applied loading, instabilities can have a significant impact on the safety and performance of the structure, primarily during construction stages. Although some limit states are similar for steel and concrete structural elements, for many situations the controlling limit states are often substantially different between the two materials. The following two sub-sections provide a general introduction to the general topics of stability that will most likely apply to steel and concrete structures.

4.1.1 Steel Structures

Stability limit states often control the design of compression members in steel structures. The stability of structural elements and systems can generally be divided into two areas: local buckling and global buckling. Local buckling modes typically involve buckling of elements of the cross-section, such as the flanges (flange local buckling) or the web (web bend buckling), both of which are usually controlled by limiting the geometric profile of the section. Global buckling modes include both individual member buckling as well as system buckling involving a group of structural members. Global member stability is a function of several parameters, including the member geometry, unbraced length, and boundary (support) conditions. Depending on the cross-sectional geometry and bracing that is provided, the potential global buckling modes for columns can consist of flexural buckling, torsional buckling, or flexural-torsional buckling. The global buckling mode for girders is always flexural-torsional (also often referred to as lateral-torsional). In addition to members buckling, interconnected girders (such as a multi-girder system with cross-frames) can also fail in a system buckling mode. System buckling of interconnected girders is covered in Chapter 5. Although this chapter is presented primarily from a steel perspective, most of the principles also apply to concrete structural members and systems. In addition to buckling modes, concrete girders (and steel girders in the absence of bracing) should be evaluated for roll stability of the girder to ensure that the member does not fail as a result of overturning. Roll stability is not discussed in this chapter but is discussed in Chapter 5.

4.1.2 Concrete Structures

Although much of this chapter is presented primarily from a steel perspective, most of the principles also apply to concrete structural members and systems. This is particularly true for concrete column elements. Although it is possible that a concrete beam may experience lateral-torsional buckling between brace points, in many situations the torsional constant of the girder is relatively large and therefore, provided twist is restrained at a few points along the girder length, the lateral-torsional buckling strength is often not an issue. However, in addition to buckling modes, concrete girders (and steel girders in the absence of bracing) should be evaluated for roll stability of the girder to ensure that the member does not fail as a result of overturning. Roll stability is not discussed in this chapter but is discussed in Chapter 5.

4.1.3 Analysis for Stability

Although many designers make use of a first-order structural analysis to determine design forces, such an analysis does not provide an estimate of buckling-related deformations, nor can such an analysis provide an estimate of the buckling strength. Two types of analyses are generally associated with stability: a critical load analysis and a large-displacement analysis. A critical load analysis is found from an eigenvalue buckling analysis as outlined in Chapter 6. Many commercially available software packages are capable of determining eigenvalues. Many of the expressions that are presented in this chapter for evaluating the buckling capacity of plates and elements are synonymous with the critical loads that are predicted by eigenvalues. The critical load provides a valuable indicator of the maximum capacity of the section under ideal conditions. However, because of local and global imperfections, the actual capacity is often lower than predicted by the critical load. Methods of stability analyses are discussed in Chapter 6. This chapter focuses on the stability buckling behavior of structural members and has been divided into six sections, including this introduction. The buckling capacity is significantly affected by geometrical imperfections and residual stresses, both of which are discussed in the following section. The subsequent sections focus on the buckling behavior of columns, plates, and girders. The final section of the chapter provides a discussion on the fundamental strength and stiffness requirements for proper stability bracing. Bracing is discussed in more detail in Chapter 5.

SECTION 2. GEOMETRICAL IMPERFECTIONS AND RESIDUAL STRESSES

There are a number of geometrical imperfections that can have a substantial impact on the buckling resistance of the section. Many of the imperfections in steel sections come as a result of the manufacturing process. The imperfections may consist of inconsistencies or distortions of the cross-section, out-of-straightness of the member, as well as plumbness limits in the erected structure. Most codes and standards including the AASHTO Specification (2012) reference the American Society of Testing and

Materials (ASTM) A6/A6M (2012) for tolerances on plates, rolled shapes, and fabricated steel sections. Although inconsistencies and distortions of the cross-section may have some impact on the global buckling behavior, the effect will generally be more severe on the local buckling capacity.

Although the manufacturing process can also lead to imperfections in concrete structures, imperfections can also come from prestressing and long-term creep deformations. Due to the weight of the concrete members, lateral imperfections can cause large over-turning moments. Problematic situations that have occurred during construction on the concrete girders have often been due to inadequate or incorrectly installed braces that are intended to stabilize the girders. Tolerance limits for sweep, camber, cross-section and other properties of precast concrete girders are provided in the PCI Manual for Quality Control for Plants and Production of Structural Precast Concrete Products (MNL 116-99)

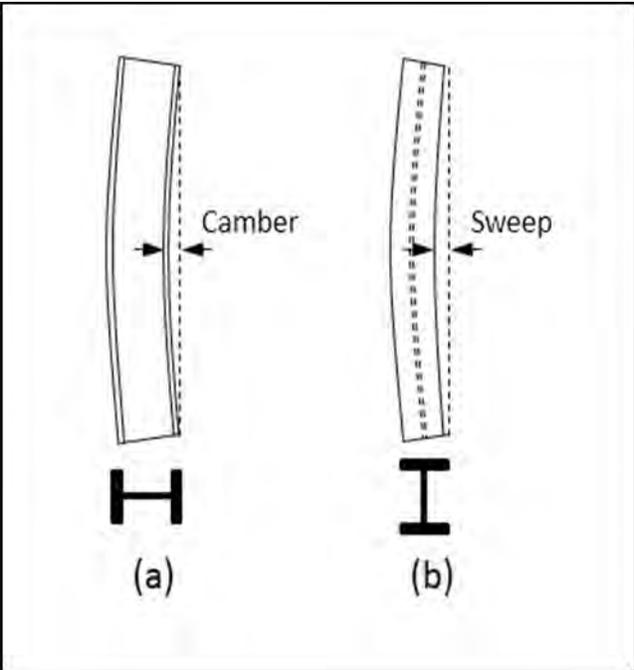


Figure 4-1 Camber and Sweep in Steel Members

Global buckling modes are much more sensitive to variations in the profile of the member along the length. For rolled shapes, ASTM A6 provides tolerances on the camber and sweep of the member as depicted in Figure 4-1. Camber is generally defined as the intentional bending of the member to account for dead load deflections, while sweep is an out-of-plane imperfection from the member's designed geometry. The ASTM A6 tolerance on sweep for sections that are likely to be used in bridges is equal to $L/960$.

In addition to the magnitude of the imperfection, the shape of the imperfection has an impact on the buckling capacity. If the shape of the imperfection is similar to the controlling buckling mode, the imperfection has a more substantial effect on the buckling capacity. When analyzing members for stability using a large displacement analysis, the potential imperfection should be included in the geometry of the member. The critical shape of the imperfection in these analyses is dependent on the controlling buckling mode. For buckling modes that primarily include lateral translation, a translation such as those depicted in Figure 4-1 are generally critical. However, for mode shapes involving a torsional deformation, the critical imperfection includes twist of the section as well. An effective method of incorporating the critical shape imperfection is to use the eigenvalue from an eigenvalue solution to create a seed imperfection for the large displacement analysis. Modeling the imperfections in the large displacement analysis is covered in more detail in Chapter 6.

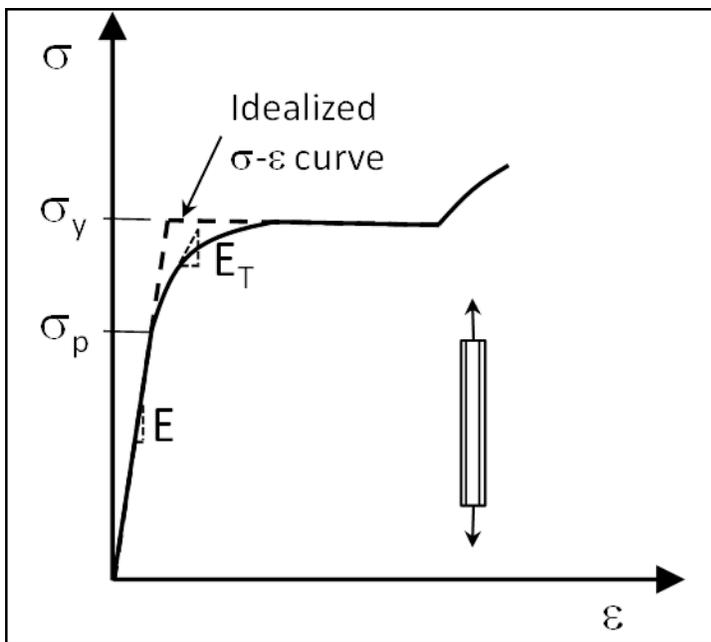


Figure 4-2 Effects of Residual Stresses on Stress-strain

The buckling strength of a member is often significantly affected by residual stresses, which are stresses that are locked into the cross-section and are typically caused during manufacturing or fabrication. The locked-in stresses consist of a combination of tensile and compressive stresses with force resultants that are in equilibrium. Although residual stresses will cause a member to experience inelasticity at an applied stress lower than the yield strength of the material, because the tensile and compressive force resultants are in equilibrium, the ultimate strength of a member in tension is not generally reduced. This is demonstrated in Figure 4-2, which depicts a stress-strain curve for a steel section with residual stresses. The dashed line represents the idealized stress-strain curve for mild-carbon steel with a well-defined yield plateau. The idealized stress-strain curve typically comes from a test on a coupon of the material cut from the main member. The residual stresses are released when the coupon is cut from the material, and therefore, the resulting curve is often not representative of the behavior of the entire

cross-section. The presence of residual stresses leads to yielding of parts of the cross-section at applied stress levels lower than the yield strength of the material. However, because the tensile residual stresses are counter-balanced by compressive residual stresses, the total yield capacity of the entire section is still the same: P_y equals σ_y times A_g , where P_y is the force resulting in full yielding of the cross-section, σ_y (in practice also sometimes denoted by F_y) is the yield strength of the material, and A_g is the gross cross-sectional area of the member. The magnitude of the maximum tensile residual stress in the cross-section is the difference between the yield strength, σ_y , and the proportional limit stress, σ_p , which is the stress where the σ - ϵ curve becomes nonlinear.

A curve similar to that shown in Figure 4-2 will result when a member is subjected to compression, provided that the length of the member is short enough (a stub column section) so that the member will not buckle. As is discussed in the following section, the buckling strength is a function of the stiffness (modulus) of the material. Above the proportional limit stress, the tangent stiffness (E_T) of the material is less than the elastic stiffness (E). As a result, the buckling strength of a member can be significantly reduced by the presence of residual stresses on the cross-section. Although the applied stress may be less than the yield strength of the material, the residual stresses lead to premature yielding of elements of the cross-section that reduce the effective stiffness of the member compared to the full elastic stiffness. Therefore, to account for the effects of residual stresses, most design specifications have both elastic and inelastic buckling solutions. The inelastic buckling solution takes into account the impact of residual stresses on the stiffness of the member.

As noted earlier in this section, residual stresses are often caused by the manufacturing or fabrication processes of the steel member. For example, in rolled W -shapes residual stresses are primarily caused during the manufacturing process at the mill. To facilitate the rolling process, a steel billet for the W -shape is raised to a high temperature so that the modulus of the material is relatively low, thereby making the shape easy to roll without excessively wearing the mechanical rollers. Once the W -shape has been rolled and achieved a shape within applicable tolerances, the member is left to cool.

As the member cools, the steel begins to regain the full elastic stiffness and also shortens as dictated by the thermal properties of expansion and contraction. However, thermal gradients exist as the shape gradually cools. The first portions of the cross-section to cool are the regions that have the most surface area exposed to the atmosphere, such as the flange tips. As these regions cool and contract, the modulus of elasticity of the material increases to the full elastic value.

Although some parts of the section may have cooled and possess nearly the full elastic modulus, other regions are still hot, and as they cool, will continue to contract. However, in order to contract, regions of the section with significant material stiffness must be stressed to accommodate the thermal contraction of the hotter regions — which therefore leads to residual stresses. In general, portions of the section that cool first develop compressive residual stresses, while portions that cool last develop tensile residual stresses. As noted earlier, the tensile and compressive force resultants from

the stresses must be equal for static equilibrium. There have been a number of previous studies on the magnitude and distribution of residual stresses in steel members (SSRC 2010). A typical residual stress pattern in a rolled W-shape is depicted in Figure 4-3. Because the tips of the flanges and mid-depth of the web cool first, these sections experience compressive residual stresses while the flange to web intersection cools last and material in that area typically experiences tensile residual stresses.

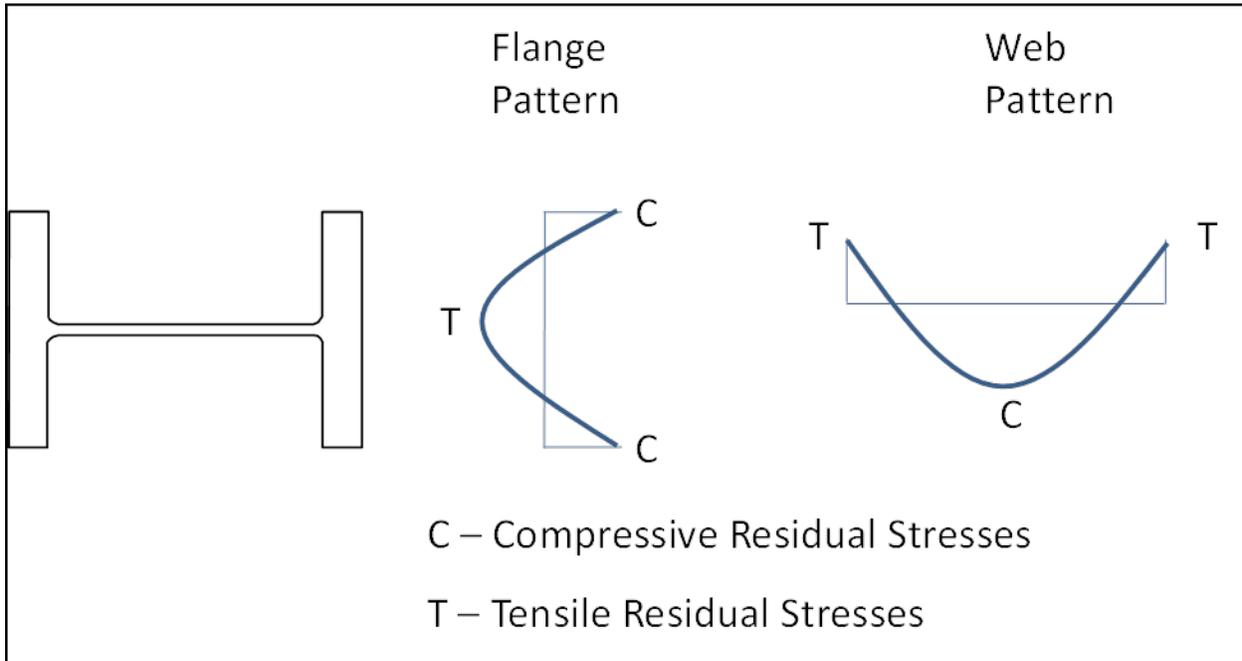


Figure 4-3 Typical Residual Stress Distribution on a Rolled Wide Flange Section

Sections built-up from welded plates also develop substantial residual stresses. The high localized heat input from the welding process coupled with the high thermal conductivity of steel can result in a quenching effect in the welds as the steel plates draw heat away from the weld. The residual stresses that develop from welds can be significantly higher than those found in rolled shapes. Preheating the plates in the weld area helps to minimize the large thermal gradient and can reduce the magnitudes of the residual stress as well as the plate distortions in the welded regions. Residual stress distributions are also affected by the straightening and cambering of girders by cold working the steel material. Rolled sections can be straightened by either a rotarizing process or by gag straightening. Many mills use the rotarizing process in which the shape is passed through a series of rollers that cold work the section back and forth to achieve a straight section.

On a W-shape, the rollers typically react on the fillets (often referred to as the K-zone) of the shape. Although rotarizing is a quick method to straighten the section, the large reactions from the rotarizing on a localized region of the shape may affect the ductility of the steel at the roller contact point and can lead to a region of low fracture toughness for

crack initiation. The rotarizing process produces residual stresses all along the length. The other straightening process is referred to as gag straightening in which hydraulic presses act at a few locations and deform the member inelastically to achieve a member that satisfies the required sweep (or camber) tolerances. The method of gag straightening is a manual process that is more labor intensive than the automated rotarizing process; however gag straightening does not generally lead to the issues with low fracture toughness at localized regions of the member as outlined for rotarized sections.

The impact of residual stresses on the buckling behavior differs depending on the axis of the section. For example, the effects of residual stresses have a more significant impact for buckling about the weak axis of a wide flange section compared to the buckling about the strong axis. Some past studies have recommended different buckling solutions depending on the axis that the section is buckling about; however the suitability of these different solutions is questionable. Although there have been numerous studies in the past on the magnitude and distribution of residual stresses, there is a great deal of variability. Numerous factors play a role in the actual distribution and magnitude. Although there is significant scatter in the test results due to variations in the distribution of residual stresses, inelastic buckling solutions have been developed for design specifications that provide a reasonable estimate of the impact of residual stresses on the buckling behavior. The following section focuses on the buckling behavior of column sections. Although elastic buckling behavior is discussed first, methods of accounting for inelastic effects on the buckling behavior are also covered.

SECTION 3. COLUMN BUCKLING

There are several applications for axially loaded compression members in bridge applications, including: piers, truss members, and temporary supports, such as shore towers. There are three buckling modes possible for axially loaded compression members: flexural buckling, torsional buckling, and flexural-torsional buckling. The following subsections provide an outline of the solutions for the different modes. The controlling mode is generally the value that provides the lowest capacity. Although a complete stability design considers all three modes, with a little insight into the buckling solutions, a designer can eliminate the need to evaluate some of the modes depending on the shape of the cross-section and the bracing details. A discussion of when the particular modes need to be evaluated is provided in the following subsections.

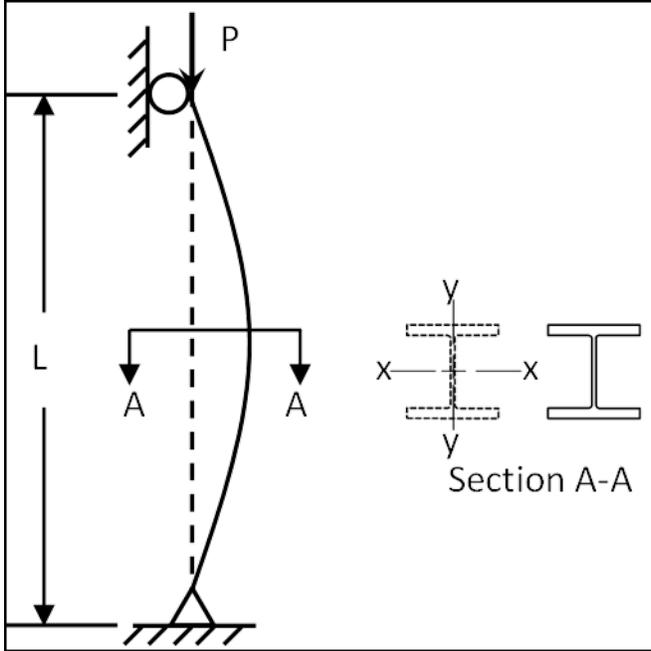


Figure 4-4 Flexural Buckling of Wide Flange Column

4.3.1 Flexural Buckling

The buckling mode that is generally covered in all courses on structural steel design is the flexural buckling mode. As a result, most engineers are the most familiar with flexural buckling, which involves a lateral translation of the cross-section about one of the principle axes as depicted in Figure 4-4. The pin-ended column in Figure 4-4 is often referred to as an Euler column as a tribute to Leonhard Euler who was the first individual to develop a mathematical formulation in the mid-1700s to predict the elastic buckling capacity of axial loaded columns.

The elastic solution developed by Euler for the pin-ended column is given in the following expression:

$$P_E = \frac{\pi^2 EI}{L^2} \quad \text{Equation 4-1}$$

Where:

- P_E = the critical elastic buckling load (Euler load) (kips)
- E = the modulus of elasticity (ksi)
- I = the moment of inertia of the section about the buckling axis (y-y axis in figure) (in^4)
- L = the unbraced length of the column (in)

The controlling buckling capacity of the column in Figure 4-4 would be the smaller of buckling about the x-x and y-y axis. If the unbraced length is the same about the two axes, the smaller moment of inertia will dictate the buckling capacity. For this reason, the y-y axis is often referred to as the weak axis since $I_y < I_x$ in I-shaped members,

generally referred to as I-girders. However, the strong axis can still control buckling if the unbraced length is larger about that axis or if the translational or rotational support conditions are different about the two axes.

Equation 4-1 is applicable to columns with pinned ends. For more general problems, an effective length factor (K) is often incorporated, resulting in the following expression:

$$P_E = \frac{\pi^2 EI}{(KL)^2}$$

Equation 4-2

The effective length factor converts a column in a structural system to an equivalent Euler column. A physical representation of the effective length factor is depicted in Figure 4-5. The columns in the portal frame are pinned at the base and restrained by the girder at the top. The effective length factor for a column in the structural system is a function of the relative stiffness between the columns and the restraining girder at the top of the frame. The effective length factor converts the columns in the frame to the equivalent length of the pin-ended Euler column in the figure.

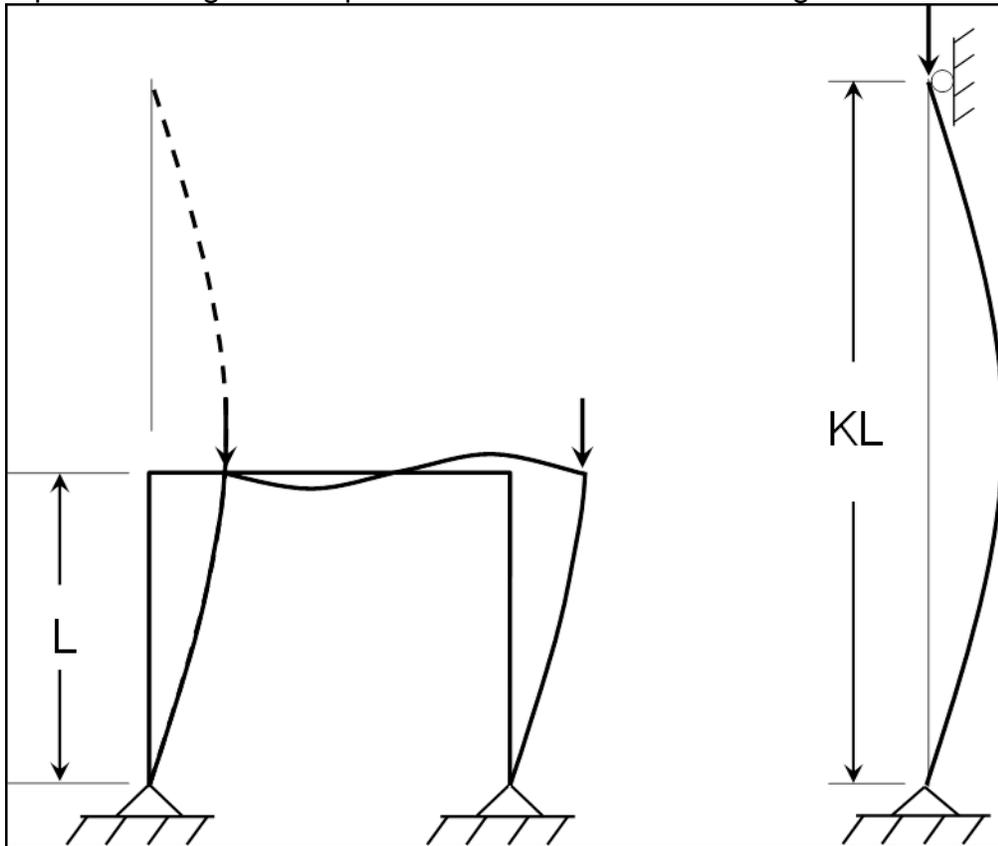


Figure 4-5 Effective Length Factor

Effective length factors for flexural buckling are often divided into two categories as a function of the bracing condition of the column. The columns in Figure 4-5 are part of an

unbraced frame that can side-sway. The range for the effective length factors for unbraced frames is $1.0 \leq K_{\text{sway}} < \infty$. For braced members in which the ends cannot translate relative to one another, the range for the effective length factor is $0.5 \leq K_{\text{no sway}} \leq 1.0$. Many specifications or the associated commentary sections provide a figure similar to the one shown in Figure 4-6 that tabulate the K-factors for idealized support conditions. Such a figure is useful to provide an indication of the range of K-factors for idealized columns; however, most structures do not match the idealized support conditions. Therefore, for design, a solution that can account for the variable support conditions is typically desired. A more general solution is available with the alignment chart that is commonly provided in most design specifications (AASHTO Section 4.6.2.5).

	Braced Column			Unbraced Column		
Column Condition						
K-Factor	0.5	0.7	1.0	1.0	2.0	2.0
Support Condition	<ul style="list-style-type: none"> Rotation Fixed and Lateral Translation Fixed Rotation Free and Lateral Translation Fixed Rotation Fixed and Lateral Translation Free Rotation Free and Lateral Translation Free 					

Figure 4-6 K factors for Idealized Support Conditions

As noted earlier, design specifications typically provide both elastic and inelastic buckling solutions. Since the solutions will typically be a function of the material yield strength, the buckling solutions are sometimes provided in terms of stress instead of force. Dividing both sides of Equation 4-2 by the cross-sectional area results in a stress and gives the following formulation:

$$F_E = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$$

Equation 4-3

Where:

F_E = the Euler buckling stress (ksi)

r = the radius of gyration which is equal to $r = \sqrt{I/A_g}$ (in)

A_g = the gross area of the cross-section (in²)

The ratio KL/r is a unitless quantity that is referred to as the slenderness of the column. Most design specifications use an elastic buckling solution that is some function of the Euler solution given by Equation 4-3.

The inelastic buckling solution in most specifications is tangent to the elastic solution at a selected column slenderness and then tends to the nominal yield strength (usually including a resistance factor, ϕ) at a column slenderness of 0. The combined elastic and inelastic solutions are usually referred to as the column curve. The AASHTO LRFD specification has adopted the column curve from the AISC Specification (2010) as given in the following equations:

$$\frac{P_E}{P_o} \geq 0.44, \text{ then: } P_n = \left[0.658^{\left(\frac{P_o}{P_E}\right)} \right] P_o$$

Equation 4-4

$$\text{If } \frac{P_E}{P_o} < 0.44, \text{ then: } P_n = 0.877 P_E$$

Equation 4-5

Where:

P_n = the nominal buckling strength (k)

P_E = the Euler buckling load given by Equation 4-2(k)

P_o = the yield load of the section given by $F_y A_g$ (k)

F_y = the nominal yield strength of the column material (ksi)

A_g = the gross area of the column (in²)

The AASHTO specification also includes a Q factor in the calculation of P_o , which is a factor that accounts for slender cross-sectional elements that may be affected by local buckling. Flexural buckling is a buckling mode that may control the design of either doubly-symmetric or singly-symmetric sections. The term doubly-symmetric refers to a section with two axes of symmetry, while a singly-symmetric section possesses a single axis of symmetry. Some typical doubly-symmetric and singly-symmetric sections are depicted in Figure 4-7.

4.3.2 Torsional Buckling

Another potential mode to be considered for axially-loaded columns is torsional buckling, in which the instability involves a pure twist of the section. The point that the section twists about is dependent on the cross-sectional properties and the bracing details that are used. The assumed center of twist for most columns is the shear center of the section; however as is subsequently discussed in this section, depending on the lateral bracing details, some columns may twist about a restrained axis. Before the buckling behavior is discussed, a brief overview of the torsional properties of the section is prudent.

The torsional resistance in thin-walled members results from either Saint-Venant torsional stiffness or warping torsional stiffness. The Saint-Venant stiffness is often referred to as uniform torsion, since the stiffness does not vary along the length and is also not sensitive to the support conditions of the section. Saint-Venant torsion results in a pure shear deformation in the plane of the plates that make up the member.

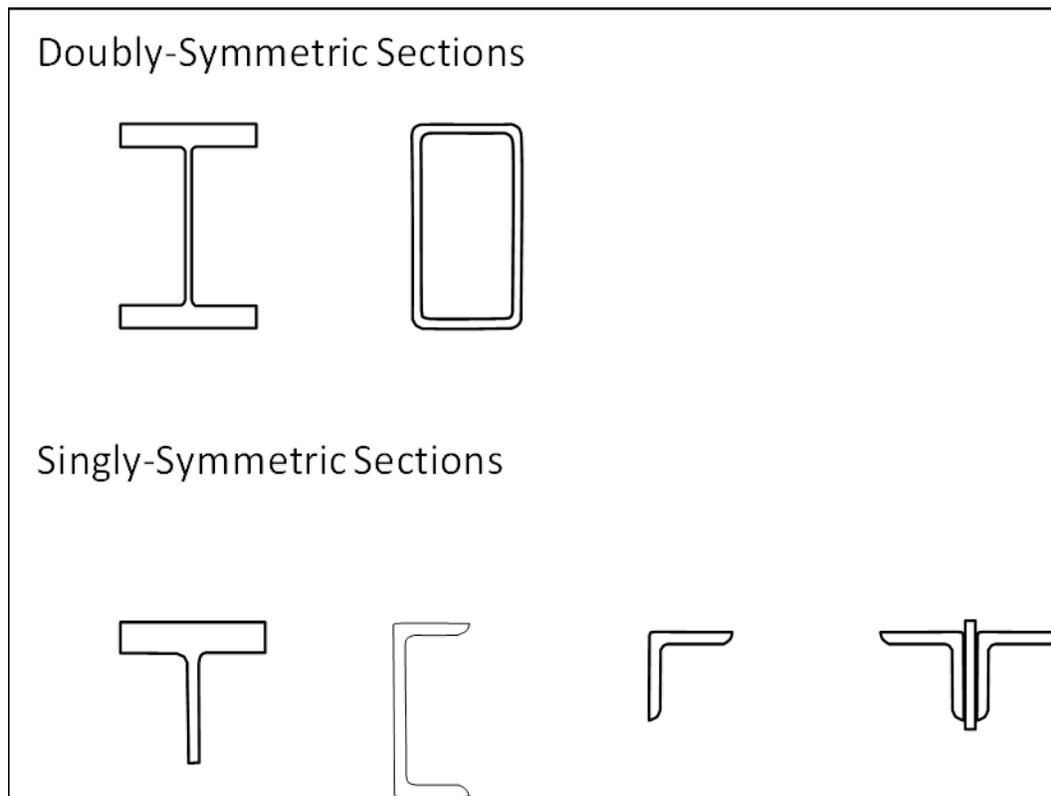


Figure 4-7 Typical Doubly and Singly-Symmetric Sections

The warping torsional resistance, on the other hand, is often referred to as non-uniform torsion since the stiffness is associated with the bending deformation in the plane of the individual plates that comprise the section. The warping stiffness of a section is related to the member's resistance to warping deformation.

Two I-girders subjected to a torque at the ends are shown in Figure 4-8 to illustrate warping deformation and also warping stiffness. The applied torque is indicated using the right-hand rule convention (direct thumb of right hand in the direction of double arrow heads and fingers curl in direction of applied torque). Figure 4-8 shows that warping deformation consists of a twist of the flanges relative to each other about a vertical axis through the web. Warping deformation distorts the cross-section such that it no longer is a plane section because the two flanges have distorted relative to one another. Twist about the longitudinal axis of the member in Figure 4-8A is prevented at one end; however the warping deformations are not restrained. Since the section is free to warp along the entire length, the flanges remain straight as they twist relative to each other and the member only possesses Saint-Venant torsional stiffness.

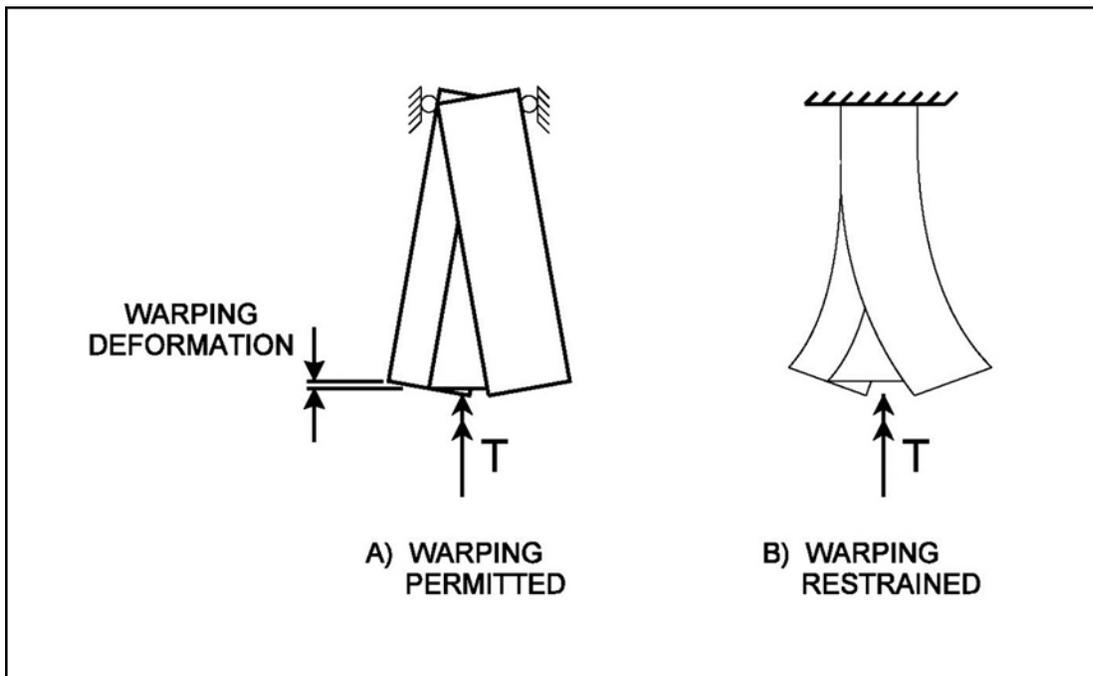


Figure 4-8 Warping Stiffness and Bending Stiffness of the Plate Elements

The wide flange section in Figure 4-8B has both twist and warping deformation prevented at one end. With warping restrained at just one location along the length, the member cannot twist without bending the flanges. Since the flanges must bend if the member twists, the section therefore has warping stiffness. The warping torsion produces longitudinal stresses in the flanges of the member.

Many members do not have a physical restraint preventing warping as shown in Figure 4-8B; however, the member still has warping stiffness if twist is prevented at a minimum of two points along the longitudinal axis. The twist restraint end conditions prevent the section from rotating about the longitudinal axis, but otherwise do not specifically restrain warping deformation of the section. Since the bending stiffness is very sensitive to the unsupported length, the warping stiffness is highly variable with the magnitude of the unbraced length. Larger bending lengths of the plate elements from the member twist lead to lower torsional warping resistance compared to shorter bending lengths.

4.3.2.1 Elastic Torsional Buckling about the Shear Center

Based upon the torsional stiffness components outlined above, the torsional buckling resistance potentially includes both Saint-Venant and warping components. For doubly-symmetric sections, the elastic torsional buckling capacity is given as follows:

$$\text{Where: } P_T = \left[\frac{\pi^2 E C_w}{(K_z L_z)^2} + GJ \right] \frac{A_g}{I_x + I_y} \quad \text{Equation 4-6}$$

- E = the modulus of elasticity (ksi)
- G = the shear modulus (ksi)
- J = the torsional constant (in⁴)
- C_w = the warping constant (in⁶)
- K_z = the effective length factor for torsion
- L_z = the spacing between locations restrained from twist (in)
- A_g = the area of the gross cross-section (in²)
- I_x = the moment of inertia about the x axis (in⁴)
- I_y = the moment of inertia about the y axis (in⁴)

The first term in the brackets is related to the warping stiffness while the second term is related to the Saint-Venant stiffness. The effective length factor in Equation 4-6 reflects the beneficial effects of warping restraint on the stiffness of the section. A pinned end in torsion has twist restrained; however, the section is free to warp. A fixed end in torsion is one that has twist restrained and full warping restraint. Therefore, if twist is restrained at both ends of the unbraced length, the effective length factor for torsion is as follows:

- Section free to warp at both ends of the unbraced length: K_z = 1.0
- Section free to warp at one end, warping restrained at the other: K_z = 0.7
- Section fixed from warping at both ends: K_z = 0.5
- Effective length factors similar to the sway mode can be found if twist is only restrained at one end, in which case K_z ≥ 1.0

The shear modulus in Equation 4-6 is given by the expression $G = \frac{E}{2(1 + \mu)}$, where μ is Poisson's Ratio (0.3 for metals). Therefore, since the elastic modulus for steel is 29,000 ksi, the value of G = 11,200 ksi. For open cross-sections, the torsional constant is given by the following expression:

$$J = \frac{1}{3} \sum_i b_i t_i^3 \quad \text{Equation 4-7}$$

where b_i and t_i are the respective width and thickness of the plate elements that make up the cross-section of the girder. The torsional buckling behavior of closed cross-sections is discussed at the end of this subsection. For doubly-symmetric I-girders, the warping constant is equal to:

$$C_w = \frac{I_y h_o^2}{4}$$

Equation 4-8

Where, h_o is the distance between flange centroids which is equal to the total girder depth minus flange thickness, which equals $d - t_f$.

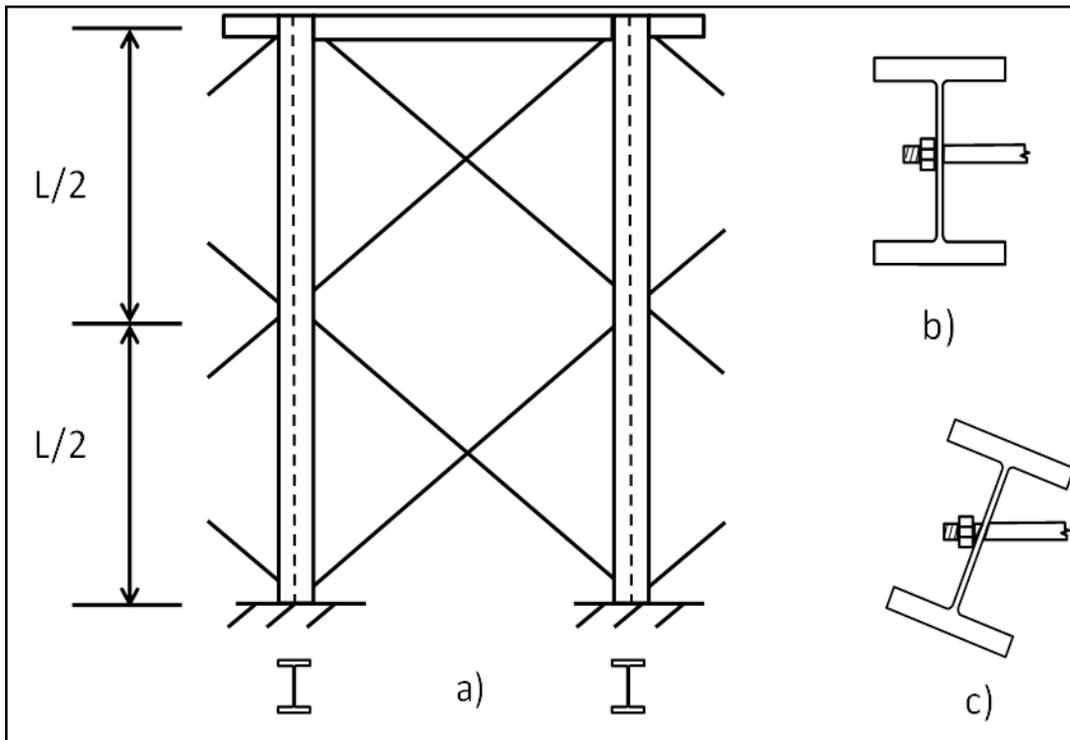


Figure 4-9 Ineffective Torsional Brace

For I-girders, weak axis flexural buckling will always control over torsional buckling when the unbraced length is the same for the two modes. When the unbraced length for torsional buckling is larger than for weak axis flexure, then both modes need to be evaluated. Figure 4-9 shows the case where the unbraced length for flexural buckling and torsional buckling are different. In many applications, rods are used to provide bracing as shown in Figure 4-9a. The detail that is commonly used to connect the rod to the wide flange section consists of a hole in the web as indicated in Figure 4-9b. Although the large axial stiffness of the rod provides good control of the lateral movement of the section, the low flexural stiffness of the rod makes such a brace ineffective at controlling twist as depicted in Figure 4-9c. As a result, with twist restrained at the top and bottom of the column, the unbraced length for torsional

buckling would be the full column length, L , while the unbraced length for weak axis flexure is $L/2$. As a result, both weak axis flexural buckling and torsional buckling need to be considered.

The solution given in Equation 4-6 is for sections with substantial warping stiffness. Cross-sections that are composed of plates that intersect at a single point do not have significant warping stiffness. Although the AISC Manual (2012) provides values of C_w for WT-sections, the torsional stiffness of the section primarily is the result of Saint-Venant stiffness. The contribution of the warping term in Equation 4-6 for WT sections is often less than 1% of the total and is therefore not worth including in the calculation. Sections with insignificant warping stiffness include angles, cruciforms, WT, and built-up double angle sections. The elastic torsional buckling capacity for these sections consists of only the Saint-Venant term:

$$P_T = \frac{GJ A_g}{I_x + I_y}$$

Equation 4-9

Although closed cross-sections do have warping stiffness, the contribution is a relatively small percentage of the total stiffness due to the extremely large Saint-Venant stiffness of the closed shape and therefore, the warping stiffness is often conservatively neglected. The torsional constant of the closed shape is much larger than the value for a comparable open section. The torsional constant for a rectangular tube is given by the following expression:

$$J = \frac{4A_o^2}{\sum_i b_i/t_i}$$

Equation 4-10

Where A_o is the enclosed area of the cross-section of the closed shape, and the variables b_i and t_i in the summation are the respective width and thickness of the i^{th} plate that make up the cross-section. For example, considering a rectangular cross-section made up of four plates, the denominator in Equation 4-10 is calculated by simply summing the width-to-thickness ratios of the four plate elements. In a circular tube section (pipe or HSS section), the torsional constant can be found in Equation 4-10 by replacing the summation in the denominator with $(2\pi r/t)$, where r is the radius of the pipe measured to the mid thickness of the wall and t is the wall thickness. A_o in Equation 4-10 is typically defined by the area enclosed from the mid-thickness of the plates that make up the cross-section. Because the torsional constant for closed cross-sections is so large, the resulting torsional buckling capacity predicted by Equation 4-10 is also very large. If the load is converted to a stress, the elastic buckling stress is of the order of $G = 11,200$ ksi, which obviously means the section would yield. As a result, torsional buckling of closed cross-sections is not a concern and the mode does not need to be evaluated. Flexural buckling will always control for columns made from closed shapes.

4.3.2.2 Elastic Torsional Buckling Cases with Twist about a Restrained Axis

The bracing detail that is shown in Figure 4-9 results in a case where the section can twist about mid height and the torsional unbraced length is larger than the weak axis flexure unbraced length. In some instances, the lateral bracing may frame into the cross-section at a location offset from the shear center. Two cases are shown in Figure 4-10 in which the lateral bracing may be offset on either the weak or strong axis. The bracing in this case only stops lateral translation and does not restrain twist. The column section can fail in these cases by torsional buckling about a restrained axis.

Timoshenko and Gere (1970) provided the following two expressions for columns with the respective cases of the lateral bracing offset on the strong or weak axis:

$$P_T = \left[\frac{P_E \left(\frac{h_o^2}{4} + a^2 \right) + GJ}{a^2 + r_x^2 + r_y^2} \right]$$

Equation 4-11

$$P_T = \left[\frac{P_E \left(\frac{h_o^2}{4} + \frac{I_x}{I_y} b^2 \right) + GJ}{b^2 + r_x^2 + r_y^2} \right]$$

Equation 4-12

Where, P_E is the Euler buckling load given by Equation 4-1 with the unbraced length equal to the total spacing between locations where twist is restrained, and a or b are the distance from the shear center to the location where the lateral brace frames into the section as indicated in Figure 4-10. The other terms are as specified in the last section. As expected, if a or b is equal to zero (lateral bracing at the shear center), Equation 4-11 and Equation 4-12 reduce to Equation 4-6. However, if the bracing is offset from the shear center, the expressions can be used to evaluate the torsional buckling resistance. If the lateral bracing is offset along the weak axis, the torsional buckling capacity will be smaller than the corresponding capacity if the lateral bracing was positioned at the shear center. Depending on the section properties, if the bracing is offset along the strong axis, the torsional buckling capacity can increase or decrease relative to shear center bracing. Further discussion and comparisons of the expressions with Equation 4-6 are provided in Helwig and Yura (1999). Common cases in which the bracing is offset from the shear center are found in trusses. The chords can be oriented as shown in Figure 4-10. If the deck is located above the top chord, the lateral bracing may be offset and Equation 4-11 or Equation 4-12 may be applicable.

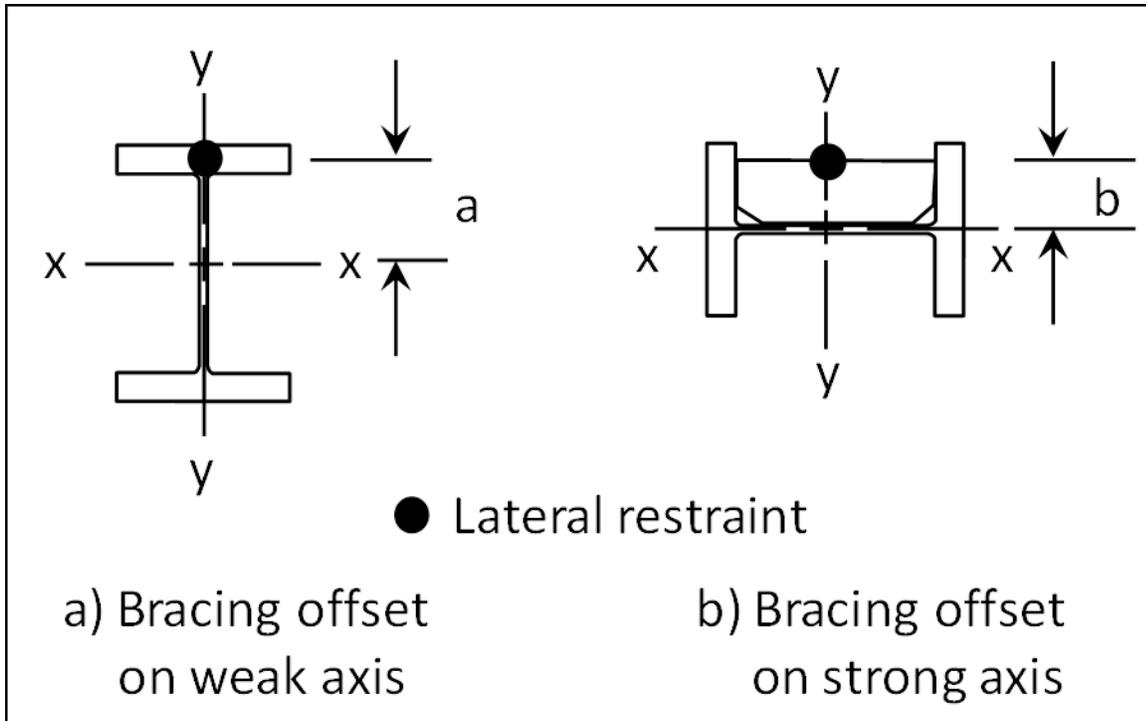


Figure 4-10 Lateral Bracing Offset from Shear Center

4.3.3 Elastic Flexural Torsional Buckling

Flexural torsional buckling has a mode shape in which the cross-section translates and twists. Many sections may be susceptible to either flexural buckling about one of the principle axes or flexural torsional buckling about the other principle axis. Although expressions can be presented for these cases (and are given at the end of this subsection), better insight into the buckling behavior can be obtained by first reviewing the general buckling expression that is valid for any shape section. The general solution is given as follows:

$$(P_{cr} - P_{Ex})(P_{cr} - P_{Ey})(P_{cr} - P_T) - P_{cr}^2 (P_{cr} - P_{Ey}) \left(\frac{x_o}{r_o} \right)^2 - P_{cr}^2 (P_{cr} - P_{Ex}) \left(\frac{y_o}{r_o} \right)^2 = 0 \quad \text{Equation 4-13}$$

Where:

- P_{cr} = the critical buckling load (kips)
- P_{Ex} = the flexural buckling capacity given by Equation 4-2 using I_x (kips)
- P_{Ey} = the flexural buckling capacity given by Equation 4-2 using I_y (kips)
- P_T = the torsional buckling capacity given by Equation 4-6 (kips)

The values of x_o and y_o are demonstrated in Figure 4-11 and represent the distance between the shear center and the geometric centroid measured about either the weak

axis or the strong axis. The term r_0 represents the polar radius of gyration and is given by the expression:

$$r_0^2 = x_0^2 + y_0^2 + \frac{I_x + I_y}{A_g}$$

Equation 4-14

Where:

A_g = the gross area of the cross-section.

Although Equation 4-13 appears relatively complex, the expression is applicable to any shape cross-section and also provides valuable insight into the behavior of various types of cross-section with regards to the potential modes that may control column buckling capacity. For example, in a doubly-symmetric section, the center of gravity and the shear center coincide. Therefore, x_0 and y_0 are equal to zero and Equation 4-13 reduces to the following:

$$(P_{cr} - P_{Ex})(P_{cr} - P_{Ey})(P_{cr} - P_T) = 0$$

Equation 4-15

Equation 4-15 has three potential roots. The critical buckling load may be either flexural buckling about the x or y axis, or torsional buckling about the shear center. In a singly-symmetric section, either x_0 or y_0 is equal to zero. For example, in the WT section shown in Figure 4-11, x_0 is equal to zero and Equation 4-13 reduces to the following expression:

$$(P_{cr} - P_{Ex})(P_{cr} - P_{Ey})(P_{cr} - P_T) - P_{cr}^2 (P_{cr} - P_{Ex}) \left(\frac{y_0}{r_0} \right)^2 = 0$$

Equation 4-16

One root to the equation is $P_{cr} = P_{Ex}$, which represents flexural buckling about the x-x axis. The other root is given by the remaining quadratic expression and represents flexural-torsional buckling in which the section translates about the y-y axis and twists about the shear center. Although the flexural torsional mode can be found directly using Equation 4-13 many design specifications, such as AASHTO, provide a separate equation for the flexural torsional mode of singly symmetric shapes:

$$P_{cr} = \left(\frac{P_{Ey} + P_T}{2H} \right) \left[1 - \sqrt{1 - \frac{4P_{Ey} P_T H}{(P_{Ey} + P_T)^2}} \right]$$

Equation 4-17

Where:

$$H = 1 - \frac{y_0^2}{r_0^2}, \text{ and } r_0^2 = y_0^2 + \frac{I_x + I_y}{A_g}$$

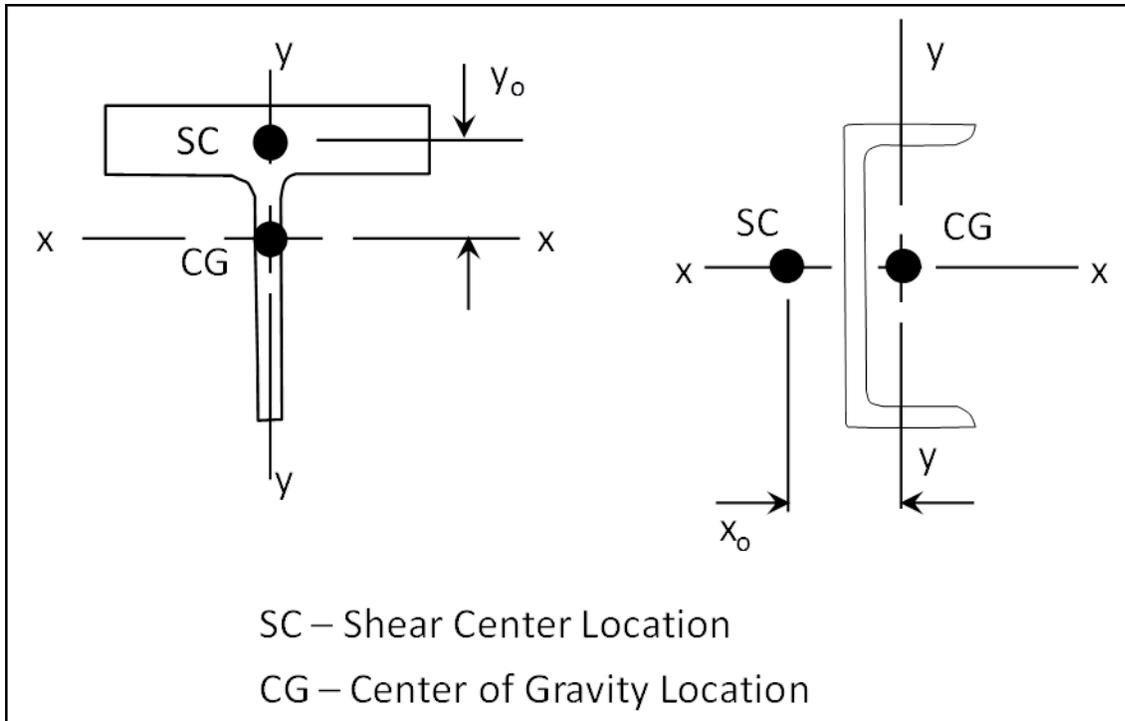


Figure 4-11 Definitions of y_o and x_o

Equation 4-17 is simply the quadratic solution to Equation 4-16 after the $(P_{cr}-P_{Ex})$ is factored out. The solution can be used to calculate the flexural torsional mode when the y-axis is the axis of symmetry. If a designer is considering a section such as a channel, as shown in Figure 4-11, x_o is equal to zero. Therefore, when the section is symmetric about the x-axis, P_{Ey} is replaced with P_{Ex} and y_o is replaced with x_o in Equation 4-17.

4.3.4 Inelastic Effects for Torsional and Flexural Torsional Buckling

The torsional and flexural torsional buckling expressions given up to this point represent the elastic torsional buckling capacities. The St. Venant term is not as severely affected by inelasticity compared to the warping term. For singly-symmetric sections without significant warping stiffness (WT-sections or double angles), the AISC specification uses the elastic torsional buckling capacity, P_T , in Equation 4-17 and uses the inelastic buckling capacity given by Equation 4-4 and Equation 4-5 for P_{Ey} .

For all other sections AISC simply calculates the elastic critical torsional or flexural torsional buckling capacity, and uses that value for P_E in Equation 4-4 and Equation 4-5. Such an approach reduces both the warping and St. Venant torsional components to account for inelasticity, which is conservative. The AASHTO Specification uses the same conservative approach and uses the elastic critical torsional or flexural-torsional buckling load for P_E in Equation 4-4 and Equation 4-5 for all cross-sections.

SECTION 4. PLATE BUCKLING AND LOCAL BUCKLING

The resulting configuration of the I-shape in Figure 4-12 maximizes the moment of inertia of the section about the X-X axis which improves the resistance to bending or global buckling about that axis. The numerical breakdown of the terms that contribute to the moments of inertia in Figure 4-12 show the terms that tend to dominate the calculation of I_x and I_y . For example, in the evaluation of I_x , the moments of inertia of the flange plates about their centroidal axes contribute very little compared to the Ad^2 term. In a similar fashion, the web contributes very little towards the moment of inertia about the Y-axis. In design, the terms that have small contributions to the moments of inertia can often be neglected with little impact on the accuracy of the calculation.

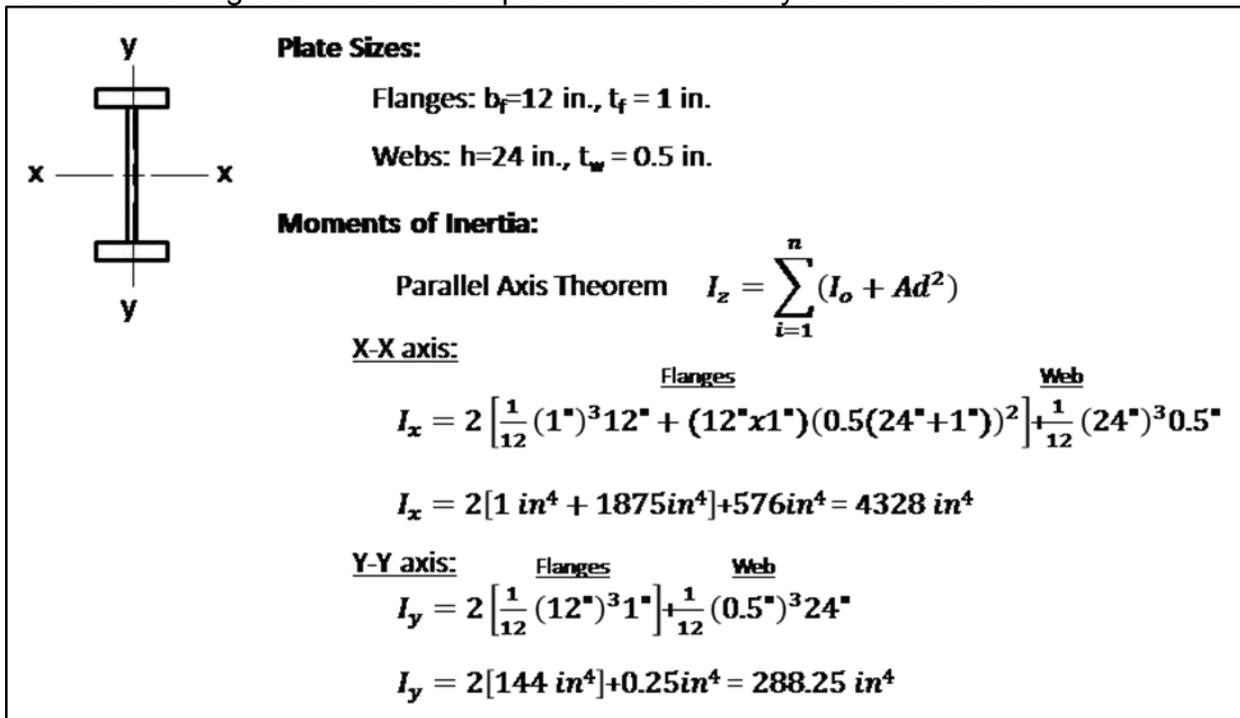


Figure 4-12 Moment of Inertia Sample Calculations

However, in the process of design, engineers often must make decisions on proportions of the member to improve the efficiency of the section. For example, if the designer desires to increase the moment of inertia about the Y-axis, the calculations demonstrate that increasing the flange width will increase I_y much more substantially than increasing the thickness. If the flange area is maintained at 12 in^2 , but a flange size of $24 \text{ in.} \times 0.5 \text{ in.}$ is used, I_y increases by a factor of 4 to 1152 in^4 . The large increase is due to the cubed exponent on the flange width. Therefore, a wider plate with a smaller thickness results in a much more significant increase in the moment of inertia compared to a thicker plate with a smaller width. From a flexural buckling perspective that is heavily dependent on the cross-section moment of inertia, this observation is valid. However, in addition to the global buckling modes discussed earlier in this chapter, another limit state that may control the capacity of the member is local buckling of one of the

elements of the cross-section. For the example given above, the cross-section with the 24 in. wide flanges that are 0.5 in. thick will be much more susceptible to local buckling of the flange plates compared to the section with the 1 in. thick flanges. However, before local buckling is discussed, an overview of plate buckling is first outlined in the following subsection.

4.4.1 Plate Buckling

There are a number of fundamental differences in the stability behavior of plates compared to some of the global buckling modes discussed earlier in this chapter. One of the primary differences between plate buckling and global buckling is associated with the post-buckling behavior. In global buckling modes, the ultimate strength of the members is typically very close to the global buckling capacity. However, in plate buckling problems, there can be substantial post buckling strength in which the load carrying capacity of the cross-section can be substantially higher than predicted from the plate buckling solution. A common example of this increased strength is found in the webs of plate girders. As outlined later in this section, the shear capacity of the webs can be much higher than predicted by shear buckling formulations.

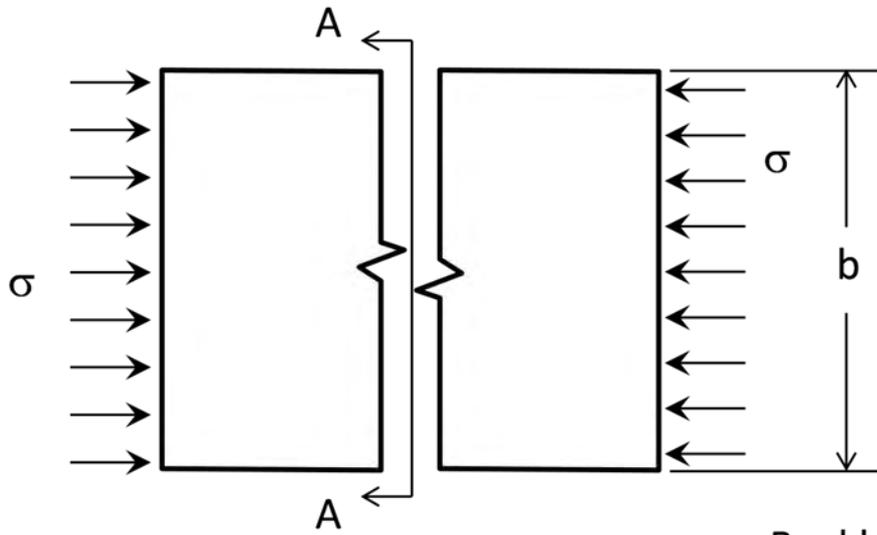
A discussion of plate buckling can be found in several texts on stability (Bleich, Timoshenko and Gere, SSRC Guide). The elastic plate buckling capacity is commonly expressed as follows:

$$\sigma_{cr} = k \frac{\pi^2 E}{12(1-\mu^2) \left(\frac{b}{t}\right)^2}$$

Equation 4-18

Where, σ_{cr} is the elastic plate buckling stress (ksi), k is the plate buckling coefficient, E is the modulus of elasticity of the plate material, μ is Poisson's ratio of the material, while b and t are the respective width and thickness of the plate. Poisson's ratio for metals is 0.3. The plate buckling coefficient, k , accounts for variations in the support conditions and distribution of stress in the plate.

The plate buckling coefficients can be divided into three primary categories: axial compression, bending, and shear. The values shown in Figure 4-13 show the plate buckling coefficients for a long plate subjected to axial compression. The support conditions have a significant impact on the plate buckling coefficient. Although many plates may not match exactly the idealized boundary conditions that are depicted in Figure 4-13, the values shown can be used to bracket the critical buckling stress.



Case	Edge Support	k	Buckled Shape Along Section A-A
1	Both Edges SS	4.00	
2	One Edge SS, the other fixed	5.42	
3	Both Edges Fixed	6.97	
4	One Edge SS, Other Free	0.42	
5	One Fixed, Other Free	1.28	

SS – Simply Supported

Figure 4-13 Plate Buckling Coefficients for Axial Compression in a Long Plate

An example of the ability to bracket the plate buckling coefficient would be the web plate of an I-shaped member such as the section depicted in Figure 4-12. For the case of pure compression, the web will likely receive some restraint from the two flange plates. The flange plates have sufficient lateral stiffness so that the edges of the plate can be assumed as supported against out-of-plane translation. Therefore, the idealized support conditions from Figure 4-13 will be somewhere between Cases 1 and 3, which correspond to the respective cases of simply supported conditions and fully fixed conditions. The simply-supported conditions will provide a conservative estimate of the

plate buckling coefficient. The actual amount of restraint that the flanges provide is a function of the stiffness of the flange plates compared to the web plate. The stiffness of the flange plates are a function of the magnitude of the axial compressive stress compared to the flange buckling stress. If the flange plates are providing restraint to the web plate, then the web can provide no rotational restraint to the flanges, and therefore, the plate buckling coefficient for the flange would be Case 4 with one edge free and the other edge pinned. The width of the plate in that case would be half the flange width ($b_f/2$).

The plate buckling coefficients in Figure 4-13 are for the case of a “long” plate. The number of waves that the plate buckles in along the length (perpendicular to the dimension “b” in the plate of the plate) can be sensitive to the total plate length. If the plate length is defined as “a”, many stability texts (Timoshenko and Gere, Bleich, SSRC Guide) provide graphs of the corresponding plate buckling coefficient, k, as a function of the aspect ratio of the plate, a/b. Variations in the plate aspect ratio, a/b, results in a series of curves with lower bound solutions that match the tabulated plate buckling coefficients given in Figure 4-13. For plates subjected to combined bending and axial compression, Equation 4-18 is applicable with the appropriate plate buckling coefficient. Due to the wide variability of potential stress distributions there is a wide range of potential plate buckling coefficients; however tabulated values for specific cases can be found in sources such as the SSRC Guide (2010).

SECTION 5. GIRDER BUCKLING

Flexural bending of girders results in a state of stress that includes both tension and compression on the cross-section. Figure 4-14 shows the typical elastic stress distributions that may develop in doubly- and singly-symmetric I-shapes from gravity loading. The maximum stresses occur at the extreme fibers of the cross-section. Elastic bending of a member about an axis of symmetry leads to a stress distribution such as that depicted in Figure 4-14(A) in which the neutral axis is located at the middle of the section and the magnitude of the maximum tensile and compressive stresses are equal. For bending about the unsymmetric axis for a singly-symmetric I-shape, the stress distribution in Figure 4-14(B) shows that the neutral axis will be closer to the larger flange and the maximum elastic bending stress occurs at the extreme fiber in the section with the smaller flange. The location of the neutral axis is dictated by horizontal equilibrium of the cross-section with recognition that the compression and tensile resultants must be equal. The bending stress, σ at any location on the cross-section a distance, y, from the neutral axis is given by the expression:

$$\sigma = \frac{M(y)}{I}$$

Equation 4-19

Where:

M = the bending moment at the particular cross-section (k-in)

I = the moment of inertia of the section about the axis of bending (in^4)

The magnitude of the maximum bending stress is determined by inserting the maximum distance, c , from the neutral axis to the extreme fibers.

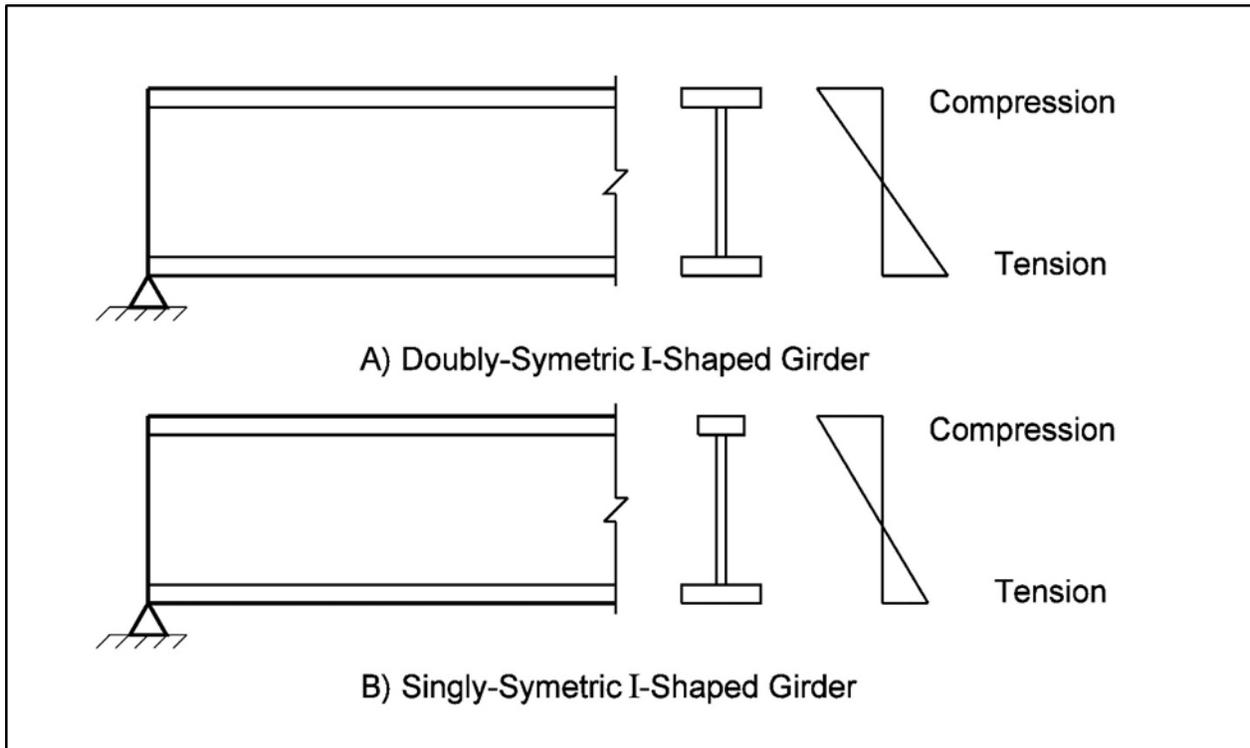


Figure 4-14 Bending Stresses in Doubly and Singly Symmetric Girders

4.5.1 Uniform Moment Loading

Because both tensile and compressive stresses result from bending, the buckling modes for girders include both lateral translation and twisting of the cross-section. Therefore, the mode of buckling is often referred to as lateral-torsional buckling (LTB). The buckled shape of a simply supported girder with compression in the top flange is depicted in Figure 4-15. The expression for the elastic critical buckling moment, M_{cr} , of a doubly-symmetric shape that was derived by Timoshenko (Timoshenko and Gere, 1961) is given in the following equation:

$$M_{cr} = \frac{\pi}{L_b} \sqrt{EI_y GJ + \frac{\pi^2 E^2 I_y C_w}{L_b^2}}$$

Equation 4-20

Most of the terms in Equation 4-20 were defined in the section on torsional buckling earlier. L_b is the unbraced length and is equal to the spacing between cross-frames or the panel points of a lateral truss on the compression flange. The first term under the radical is related to the St. Venant torsional stiffness while the second term is related to the warping torsional stiffness.

The AASHTO Specification Appendix A6 uses the following equation for evaluating the lateral-torsional buckling capacity:

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

Equation 4-21

$$r_t = \frac{b_{fc}}{\sqrt{12 \left(1 + \frac{1}{3} \frac{D_c t_w}{b_{fc} t_{fc}}\right)}}$$

Equation 4-22

Where, F_{cr} is the critical lateral-torsional buckling stress, equal to M_{cr}/S_{xc} , S_{xc} is the elastic section modulus about the major axis of the section to the compression flange, b_{fc} is the width of the compression flange, t_{fc} is the thickness of the compression flange, D_c is the depth of web in compression, t_w is the thickness of the web, and r_t is the effective radius of gyration of the section for lateral-torsional buckling.

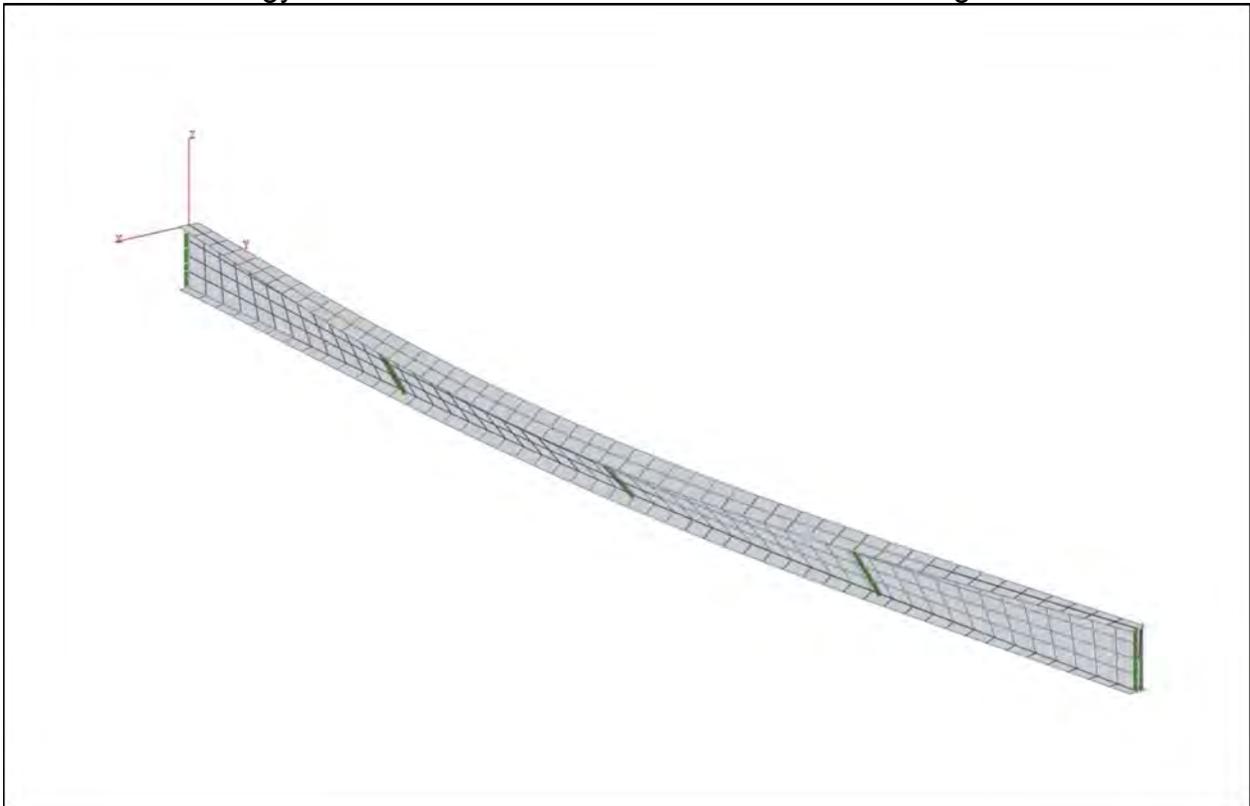


Figure 4-15 Buckled Shape of I-Girder

For doubly-symmetric I-sections, Equation 4-21 essentially produces the same solution as Equation 4-20; however the AASHTO expression is also applicable for singly-symmetric I-sections with different flange sizes.

4.5.2 Moment Gradient Loading

The buckling expressions shown in the last section were developed for the case of uniform moment loading. The effects of moment gradient are accounted for with a moment modification factor, C_b , which is applied to the expressions. The AISC specification recommends the following expression for C_b :

$$C_b = \frac{12.5M_{\max}}{2.5M_{\max} + 3M_A + 4M_B + 3M_C} \quad \text{Equation 4-23}$$

Where M_{\max} is the maximum moment in the girder segment defined by L_b , M_B is the moment at the middle of the unbraced length, while M_A and M_C are the values of the moment at the quarter points of the unbraced length. The absolute value is used for all values of the moment in Equation 4-23. Past editions of the AASHTO Specification included Equation 4-23; however the most current edition of the AASHTO LRFD specification uses the following expression:

$$C_b = 1.75 - 1.05 \frac{f_1}{f_2} + 0.3 \left(\frac{f_1}{f_2} \right)^2 \leq 2.3 \quad \text{Equation 4-24}$$

The values of f_1 and f_2 represent stresses at the ends of the unbraced length and involve a number of different cases outlined in the AASHTO specification depending on the distribution of stress. The above expression is similar in form to a long-standing formulation that was expressed in terms of moment, but was only applicable to straight line moment diagrams. The current AASHTO specification includes variable definitions of the stress to approximate the impact of non-linear moment diagrams to improve the range of applications for which the expression can be used. The AASHTO commentary frequently refers to Equation 4-21 as the “more accurate” expression.

4.5.3 Effect of Load Position on Cross-section

The C_b expressions outlined above are applicable to cases in which the applied loads are applied at midheight of the section. In situations where the loads are not applied at midheight, the buckling capacity can be substantially different than predicted by the C_b expressions. Figure 4-16 shows three different cases where the load is applied at the top flange, midheight, and the bottom flange. For example, when the load is applied at the top flange as depicted in Figure 4-16(A), the buckling capacity will be less than for cases with the load applied at midheight. The reason for the reduction in the buckling capacity is because the applied load leads to an overturning torque that tends to further destabilize the member compared to midheight loading. In a similar fashion, when the

load is applied at the bottom flange the load produces a stabilizing torque that tends to straighten the girder and results in an increase in the buckling capacity compared to midheight loading. The effects of load position are more significant for members without intermediate (between the supports) bracing. Intermediate bracing tends to reduce the impact of load position. In cases with intermediate bracing, the impact of load position can generally be neglected. For cases without intermediate bracing, solutions accounting for the impact of load position are presented in the literature (SSRC Guide, Helwig et al.).

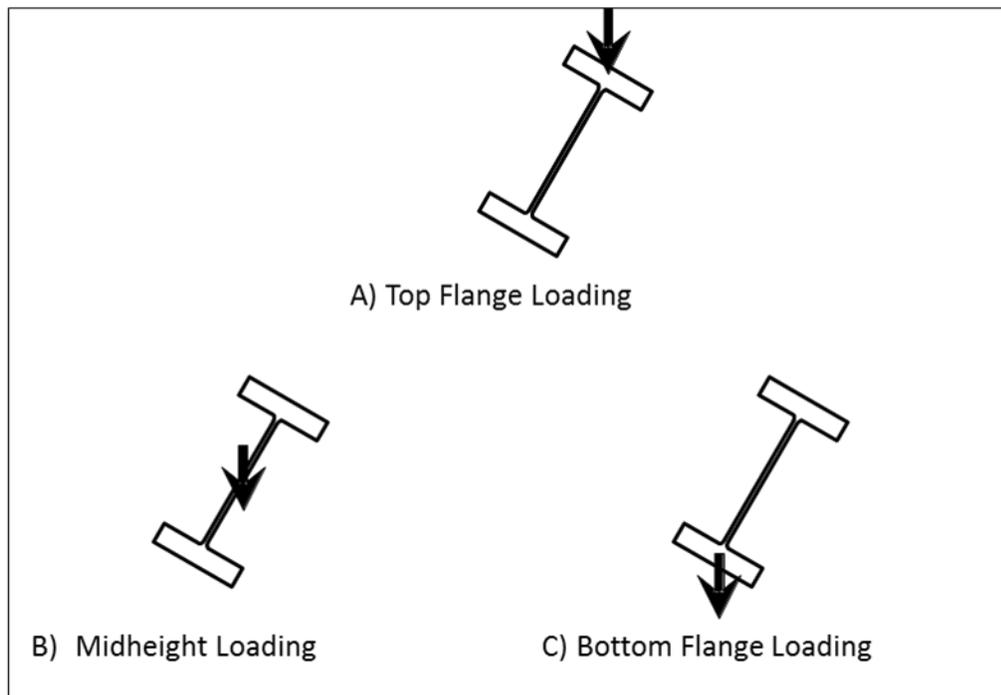


Figure 4-16 Effect of Load Position on Cross-section

Effective bracing of girders can be obtained by either preventing twist of the section or by preventing lateral movement of the compression flange. The most common form of girder bracing is cross-frames or plate diaphragms that control the twist of the girder cross-section. Therefore, L_b in the buckling expressions above are typically taken as the spacing between cross-frames or diaphragms. Bracing for girders is discussed in more detail in Chapter 5.

SECTION 6. SUMMARY

This chapter has provided an overview of the stability fundamentals necessary to understand the many factors that can affect the behavior of bridge structures during construction. A discussion of the various local and global buckling modes that are possible for column and beam systems was provided. Understanding the many factors that affect these different buckling modes can provide engineers and erectors insight into potential problems that might be encountered in the field as well as possible remedies for these problems. The subsequent chapters build from the fundamental material that is presented in this chapter.

CHAPTER 5

STABILITY IN BRIDGE ERECTION

SECTION 1. INTRODUCTION

There are a number of critical stages that need to be considered in the design of steel and concrete bridge systems. The widely accepted role of the bridge designer is to provide a safe and economical system in the completed structure, while the safety of the bridge during erection and construction is generally the responsibility of the contractor. Although the members of the bridge industry accept these roles, the distribution of responsibilities can result in some difficult challenges towards ensuring a safe environment during construction. In most situations, girder proportions are based upon the behavior of the finished bridge when the girders are composite with the concrete bridge deck. In addition to substantially improving the flexural strength of the girders, the cured concrete deck also provides continuous lateral and torsional restraint to the girders. This continuous restraint greatly improves the stability resistance of the girders. Therefore, the critical stages for structural stability generally occur during construction before the concrete deck has cured.

Assessing the stability during construction can be particularly complex due to the limited presence of bracing during early erection stages. Contractors also face challenging problems that often limit the position of cranes or falsework, such as shore towers, due to traffic congestion or geographical limitations of the job site. Most bridge owners require the contractor to present erection and construction schemes along with supporting calculations that show the scheme to be safe. However, verifying the safety of the construction scheme requires proper modeling of the structural system during the critical construction stages. From a stability perspective, there are generally three different stage classifications that present complicated scenarios:

1. the stability of individual girder segments during lifting with a crane,
2. the evaluation of the stability of the partially erected structural system, and
3. the stability and behavior of the structural system during placement of the concrete bridge deck.

Much of the behavior that is mentioned in these three points is addressed through the proper analysis of the structure, which is covered in Chapter 6. However, the stability and lateral load resistance of the structure is also highly dependent on the proper design and detailing of the bracing, which must be completed before a proper erection scheme can be investigated.

This chapter addresses the global stability of erected or partially erected superstructures. An overview is provided of some of the changing requirements that develop as girder erection and deck concrete placement progresses. In addition, sources of stability that are provided by previously completed structural elements and temporary structures are discussed.

SECTION 2. LATERAL RESISTANCE OF PARTIALLY ERECTED SUPERSTRUCTURES

5.2.1 General Considerations

Throughout the construction process, fully- or partially-erected bridge superstructures are subjected to lateral force effects from a variety of potential sources. These force effects may be due to externally applied loads, such as wind, or may be caused by internal force effects from girder curvature or buckling-induced forces caused from stability effects. This chapter focuses on the externally applied loads as well as stability-induced forces and potential sources of restraint.

A suitable load path must be provided to resist the lateral loads that are applied to the structure at various stages during construction. A force transfer mechanism must be provided that will transfer the applied loads to the foundations, which may consist of either permanent or temporary elements. Permanent foundations will generally be the piers or abutments of the structure, while temporary foundations are found in crane supports, rigging or temporary shoring towers.

Buckling-induced forces are generally the result of second-order effects which are related to the applied forces and deformations in the structure. In many cases, the lateral loads on the structure will result in deformations that intensify the second-order effects. The buckling resistance of the structure is enhanced by the use of bracing to reduce the unsupported length of main members. In many situations, the bracing that is provided may serve the dual purpose of providing stability as well as acting as a component of the load path for the resistance of lateral forces on the structure. The bracing can be attached to portions of the existing structure already completed, a temporary structure, or an adjacent structure. Some examples of these sources of support are as follows:

- Existing Structure: abutments, piers, or an adjacent girder
- Temporary Structure: falsework, a deadman block, a crane, or a shoring tower
- Adjacent Structure: existing bridge or structure adjacent to construction

Various bracing types and schemes are discussed in this chapter. While the primary functions of bracing in concrete bridges are usually to prevent roll of the girders and to provide load distribution, bracing in steel bridges often provides additional functions. Some of the fundamental characteristics that the bracing in steel bridges must possess include the following:

- The bracing must have adequate strength to resist the stability-induced forces. From this perspective, the brace should not have strength issues that may be associated with material inelasticity or a potential instability of the brace itself that may preclude the brace from functioning properly.
- A clear load path must be provided between the primary member and the brace. The connection to the brace is often a critical component to the system both from

a strength and stiffness perspective. As is noted later in the chapter, a bracing system is only as stiff as the most flexible component. Therefore, providing a stiff brace with an overly-flexible connection will usually result in an ineffective brace.

- Because the braces may resist both stability-induced forces and also serve as a member of a lateral load path, the bracing must be designed to resist the combination of both the externally applied loads and the stability-induced forces.

5.2.2 Lateral Resistance Effects

Lateral stability is often attained by providing structural elements to offer restraint to the primary members that would otherwise lack sufficient stability to resist the applied loads. In most situations, these elements are discrete bracing members, such as diaphragms, cross-frames, or blocking that span between adjacent girders. In addition, lateral resistance can also be obtained by alternative means such as falsework or other sources of temporary support. In some situations, restraint may come from sources that are not relied upon in conventional bridge design, such as stay-in-place forms, for the concrete bridge deck.

In steel girder systems, cross-frames or diaphragms are required at all points of support, including at temporary shoring towers, in order to transfer lateral and torsional forces from the superstructure to the support. Intermediate (between supports) cross-frames are also required to ensure adequate transfer of lateral and rotational forces to the system. A rational analysis should be performed to determine the intermediate cross-frame spacing during all stages of construction to ensure:

- Sufficient lateral support of the bottom flange is provided for deck overhang brackets;
- Sufficient transfer of lateral wind loads on the girders to the foundation;
- Stability of the bottom flange for loads producing compression in the bottom flange;
- Stability of the top flange for loads producing compression in the top flange, especially during the deck placement before the fresh concrete begins to act compositely with the girders;
- Control of flange lateral bending effects; and
- Adequate distribution of vertical dead and live loads (including construction loads) applied to the structure.

It should be noted that for deck replacement projects, the top flanges experience various permutations of bracing demand as sections of deck are subsequently removed. Each stage needs to be analyzed to ensure adequate stability and transference, as well as resistance to dead, live, wind and construction loads.

Cross-frames should be proportioned to be as deep as possible to efficiently transfer vertical and lateral forces to the flanges and to control girder rotation to the extent possible. In composite systems subjected to positive bending (compression in the top flange), the top flange is generally located near the neutral axis of the final composite section. In the positive moment region, the composite girders are essentially continuously braced by the concrete deck and therefore the top flange is not susceptible to significant instability or lateral demand from wind load or other lateral forces. As a result, construction dead and wind loads may be the most critical load cases that should be considered in the stability design of the member and the necessary bracing; however, these loads are typically not considered by the designer. The location and magnitudes of the forces are in part determined by the contractor's chosen erection method and sequence. In addition, when positioning bracing near the top flange, care should be taken so that it does not interfere with the placement of the deck formwork.

The cross-frame connections to the girders should be designed and detailed such that undesirable fatigue-sensitive details are avoided. If the situation necessitates fatigue-sensitive details, a detailed fatigue analysis should be performed for the final configuration of the bridge to ensure that this detail will not control the design.

Because many concrete girder systems do not have permanent cross-frames or diaphragms with a positive connection to the primary members, or because permanent cross-frames or diaphragms are installed after the primary members have been set, blocking or other temporary means of restraint are often required to provide a stable system that is capable of resisting lateral forces. The blocking provides a load path that is capable of distributing lateral loads from sources such as wind as well as resisting forces from the overhang construction.

Bracing to resist torsional forces imparted to the system as a result of deck overhangs is particularly important. This situation is problematic for both steel and concrete bridges. Deck overhangs are nearly always constructed using overhang brackets. The dead load of the wet concrete in the overhang, combined with the load imparted by the outer wheel of the screed machine, leads to a significant force in the overhang bracket that imparts a torque on the fascia girder.

If these forces are not properly accounted for in the design of a bracing system, a failure of an individual girder or a collection of girders is possible. In steel girders, the failure may manifest itself as either a local instability of the girder flange or web, lateral-torsional buckling of the fascia girder, or a global stability failure of multiple girders. In concrete girder systems, the failure mode is typically overturning (roll) of one or more of the concrete girders. In addition, rotational displacement due to overstressing and eccentric loading can occur for both steel and concrete girders, which can lead to shifting of the deck reinforcement and loss of cover. In such a case, the durability of the deck will likely be compromised, leading to poor long-term performance of the deck.

Stay-in-place (SIP) forms are increasingly used in bridge deck construction. In building construction, the commonly-used SIP forms are relied upon as lateral beam bracing.

However, SIP forms in the building industry are continuous over the top flanges of the girders and connected by welding the shear studs through the deck to the flange. Alternatively, the building connections may consist of puddle welds or self-tapping screws to achieve lateral load transfer. These connection techniques in the building industry provide a good force transfer between the SIP forms and the girders, thereby providing a good source of lateral bracing to the girders.

In the bridge industry, longer girder spans often lead to variations in flange thickness along the bridge length as well as differential camber between adjacent girders. As a result, the SIP forms require a connection that will permit the adjustment of the form elevation so as to achieve a uniform deck thickness along the bridge length. The connection that is typically used in bridges consists of a leveling angle that allows the contractor to adjust the form elevation and provide uniform deck thickness along the length and width of the bridge. Although the leveling angles are beneficial for solving many of the constructability issues with the deck, they are considered a flexible connection to the girder flange. As a result, the lateral bracing potential of the SIP form is greatly reduced and may be highly dependent on workmanship.

Due to such concerns, the AASHTO LRFD Bridge Design Specifications do not allow SIP forms to be considered to provide lateral flange stability. Direct attachment of SIP forms to flanges (Chen, Yura, Williamson, and Frank, 2005) of U-girders and use of improved details of leveling angles to provide lateral resistance (Egilmez, Helwig, Jetann, and Lowery, 2007) have, however, demonstrated that SIP forms can act as lateral bracing when properly connected to the flanges.

5.2.3 Lateral Bracing Classifications

Effective stability bracing can be achieved through a number of different means, however for the purposes of design, bracing systems are usually classified into four categories: (a) relative, (b) nodal (also called discrete), (c) lean-on, and (d) continuous. In many situations, a given bracing application may fit into multiple classifications; however the design methods that have been developed are based upon individual categories.

5.2.3.1 Relative Bracing

A relative brace system, such as diagonal bracing in column or beam systems, is attached at two locations along the length of the member that define the unbraced length, L_b . For the two columns in Figure 5-1(A) or the two girders in Figure 5-1(B), the diagonals and the horizontal struts combine to control the relative movement at the ends of the unbraced length. In this figure, the ends of the unbraced length are labeled A, B, and C. Considering the bracing panel with the ends labeled A and B, the brace controls the relative movement of point A with respect to point B and is therefore a relative brace.

Although there are two diagonals within each bracing panel, the diagonals are often designed as tension-only members since they often consist of slender members with a minimal buckling capacity. In this case the compression diagonal is neglected since its buckling capacity is often relatively small and the tension diagonal is designed to provide enough stiffness and strength to brace the structure.

Both the diagonal and the strut contribute to the strength and stiffness of the bracing system. For beams and trusses, a relative bracing system can consist of a top flange (chord) lateral truss system that may be provided to help resist lateral loads such as wind. Generally, if a perpendicular cut everywhere along the unbraced length passes through the brace, then the brace system is relative.

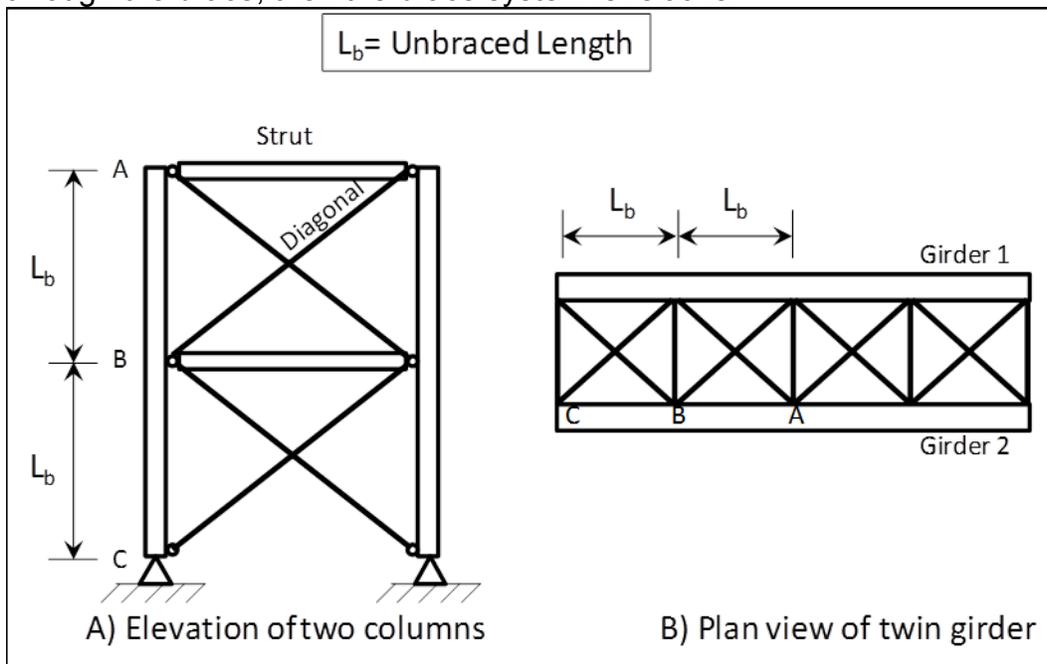


Figure 5-1 Relative Bracing System for Columns and Girder

5.2.3.2 Nodal (Discrete) Bracing

A nodal brace controls the movement only at the particular brace point, without direct interaction with adjacent braced points along the length of the member. Nodal braces can be designed to control either the lateral movement of the compression flange or to control twist of the cross-section as depicted in Figure 5.1. Figure 5-2(A) shows a cross-frame which is generally the most commonly used type of bracing that is used in steel bridges. The cross-frame ties the two girders together at a specific location along the girder length and is categorized as a torsional nodal brace since the brace controls twist of the girders at a single point. In staged construction, a lateral nodal brace might consist of a strut that is used to restrain the lateral movement of the compression flange of a girder as depicted in Figure 5-2(B). In the case depicted in Figure 5-2(B), the compression flange of the girder on the right is braced off of the two composite steel girders that have previously-placed concrete.

In this case, care needs to be taken so that the unsupported length of the bottom flange does not become too large so that the torsional flexibility leads to excessive lateral deformation. In addition, the ability for the non-composite girder on the right to deflect vertically relative to the existing composite girders during concrete placement should be taken into consideration. In situations where the laterally unrestrained length of the bottom flange becomes too large, both a top and bottom strut may be necessary, as is frequently used in lean-on bracing applications, which is discussed in the next subsection.

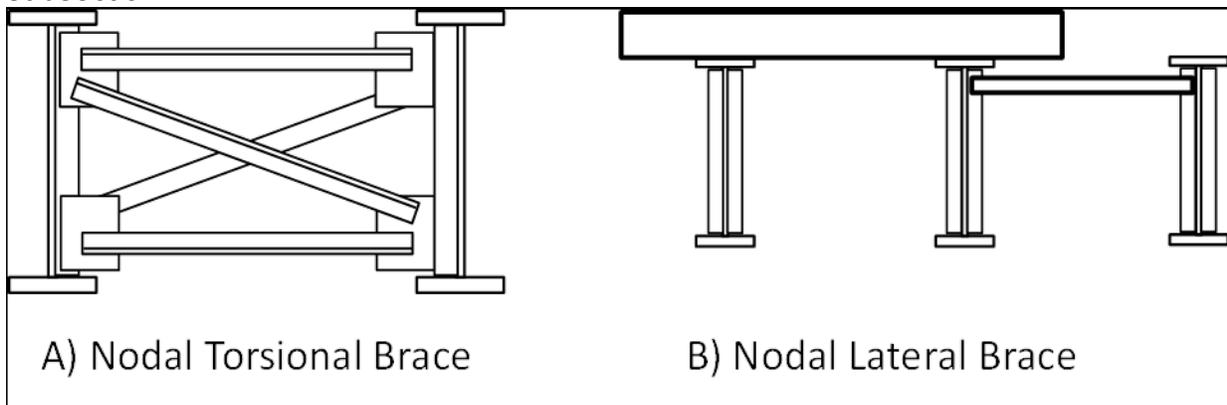


Figure 5-2 Nodal Bracing System for Bridge Girders

5.2.3.3 Lean-On Bracing

In some applications, the efficiency of the system and the behavior can improve by using lean-on techniques such as those depicted in Figure 5-3. The case shown previously in 5-2(B) is really an application of lean-on bracing; however the size of the strut necessary to brace the girders off of the hardened concrete slab would generally be based on nodal bracing concepts and the example was therefore shown in that section. In lean-on bracing cases with torsional bracing, the cross-frames are replaced by top and bottom struts that allow several girders to lean on a single cross-frame. Applications where such a bracing system is particularly useful are in bridges with large

support skews such as the one shown in Figure 5-4. Large support skews can make the cross-frames difficult to install and also may lead to large forces induced in the cross-frames from truck traffic on the bridge. The large live load induced forces increase the likelihood of fatigue cracking in the girders around the cross-frames. In lean-on bracing applications, cross-frames within a given bracing line are located as far away from the support as possible, such as the case labeled “A” in Figure 5-4 (Helwig, 2005). Top and bottom struts are used between the other girders to lean the girders on the bracing. In skewed girder applications, bracing lines (such as the point labeled as “B” in Figure 5-4) should be offset from the supports by about 4 to 5 feet to make the bracing easier to install and to also prevent large forces that would develop if the bracing line frames directly into the support.

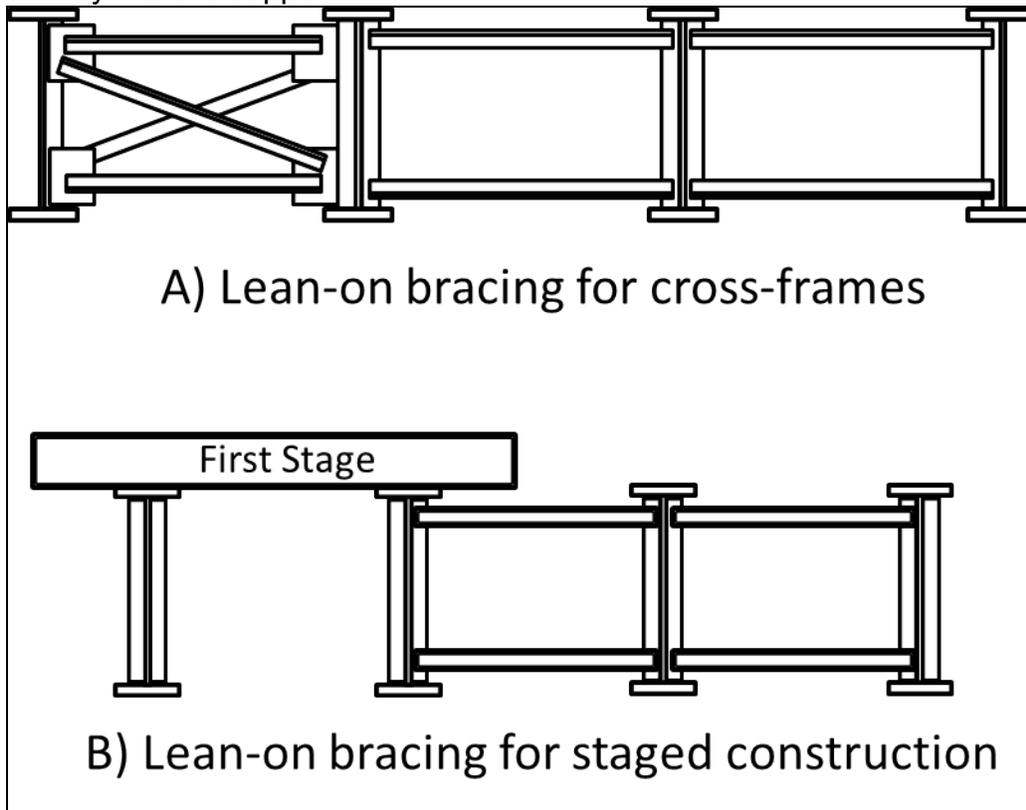


Figure 5-3 Lean-on Bracing for Bridge Girders

5.2.3.4 Continuous Bracing

The last category of bracing is continuous bracing, in which case the bracing is fastened all along the length of the member. As noted earlier, in the building industry the stay-in-place (SIP) forms are often relied upon for stability bracing, which is a form of continuous bracing. In the bridge industry, the connections between the forms and girders are too flexible and therefore, the forms are not permitted to be relied upon for bracing. Cases with continuous bracing during construction are not commonly found in bridge applications. The cured concrete slab provides continuous bracing to the steel girders in composite construction (with shear studs). The concrete slab actually

provides continuous lateral and torsional restraint to the girders. Due to the limited applications of continuous bracing in bridges, no further discussion is warranted.



Figure 5-4 Cross-frames in Skewed Application

SECTION 3. BRACING REQUIREMENTS

5.3.1 I-Girder Bracing

Both steel and concrete girders normally require bracing during the various stages of erection. Though the high torsional stiffness of standard precast concrete girders typically precludes lateral torsional buckling, bracing to assure roll stability is often required. Roll stability is addressed in Sections 7 and 8 of this chapter and Section 7 of Chapter 7.

For steel I-girder systems, effective bracing systems can be designed to provide torsional (β_T), lateral (β_L) or warping (β_W) restraints. The discussion presented in this chapter focuses on torsional bracing and lateral bracing since these types of bracing are the most commonly used stability bracing systems in bridges.

The most common forms of torsional bracing are cross-frames or diaphragms that prevent twist of the section at the brace location. If two adjacent beams are interconnected at a point by properly designed cross-frames or diaphragms, that location can be considered as torsionally braced in relation to the beam buckling strength. Though the pair of beams can move laterally at midspan, the two flanges must move laterally together and twisting of the flanges is prevented. Since twist is prevented at that point, it acts as a brace point.

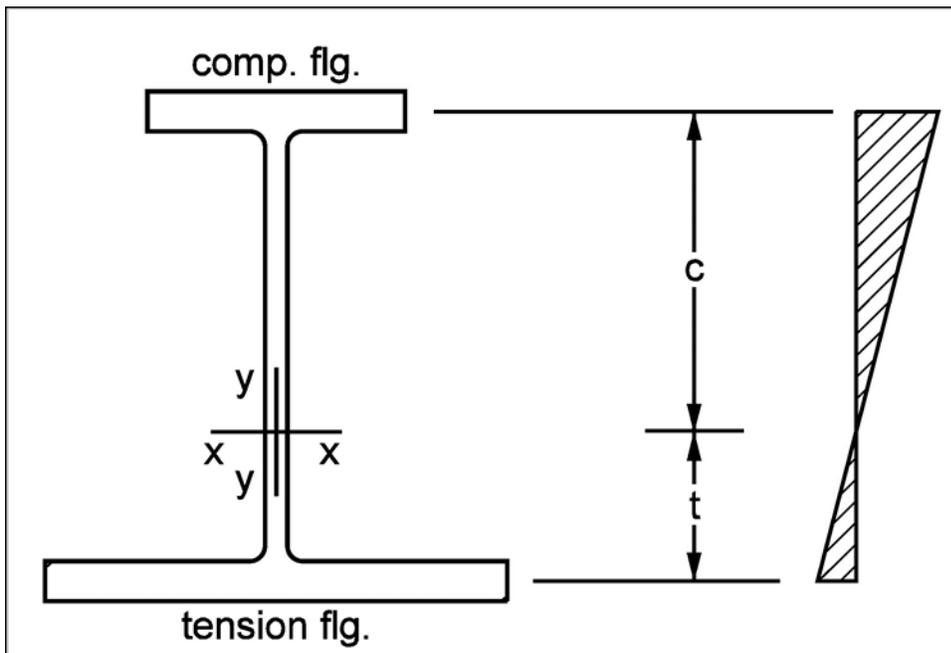


Figure 5-5 Bending Stresses in Singly Symmetric Section

The torsional bracing requirements for beam systems are provided in the AISC Specification for Structural Steel Buildings (2010) and the provisions are directly applicable to bridge systems. Because steel bridge girders may consist of singly-symmetric I-shaped sections, the provisions in the Commentary of the AISC Specification for Structural Steel Buildings (2010) are the most applicable. The required strength and stiffness of the cross-frames or diaphragms can be determined from the following expressions:

$$M_{br} \geq F_{br} h_b = \frac{0.005 L_b L M_u^2}{n E I_{eff} C_b^2 h_o} \quad \text{(Strength)} \quad \text{Equation 5-1}$$

$$\beta_T \geq \frac{2.4 L M_u^2}{\phi_{br} n E I_{eff} C_b^2} \quad \text{(Stiffness)} \quad \text{Equation 5-2}$$

Where:

- M_{br} = Design brace moment in torsional brace (kip-in)
- M_u = maximum factored girder moment within the span (kip-in)
- C_b = moment gradient magnification factor based upon girder moments between cross-frame locations (use $C_b = 1.0$ or AISC Chapter F to calculate alternate values)
- $\phi_{br} = 0.75$
- L = span length (in)
- L_b = unbraced length equal to the spacing between cross-frames (in)
- h_o = distance between flange centroids of the girder (in)
- h_b = vertical distance between cross-frame bracing chords or work points (see Figure 5-7) (in)
- n = number of braces in span
- $I_{eff} = I_{yc} + (t/c) I_{yt}$ (see Figure 5-5 for t and c definitions) (in^4)
- I_{yc} = out-of-plane moment of inertia of the compression flange about the weak axis of girder (in^4)
- I_{yt} = out-of-plane moment of inertia of the tension flange, about the weak axis of girder (in^4)
- β_T = torsional bracing system stiffness (kip-in/rad)
- F_{br} = factored chord forces in cross-frame (kip) (See Figure 5-7)

Equation 5-2 provides the stiffness requirements for the torsional bracing system, which is a function of the stiffness of the following components:

- β_b = attached brace (cross-frame) stiffness (See Figures 5-6 and 5-7)
- β_{sec} = web distortional stiffness
- β_g = in-plane girder system stiffness

Each of the above three components are subsequently discussed in the remainder of this section. Bracing systems generally follow the classic equation for springs in series. Therefore, the system stiffness, β_T , is governed by the following expression:

$$\frac{1}{\beta_T} = \frac{1}{\beta_b} + \frac{1}{\beta_{sec}} + \frac{1}{\beta_g}$$

Equation 5-3

By considering the mathematical formulation of Eq. 5.3, the system stiffness, β_T , is less than the smallest value of β_b , β_{sec} , or β_g . Therefore, it is important to properly consider the stiffness of each of the system components.

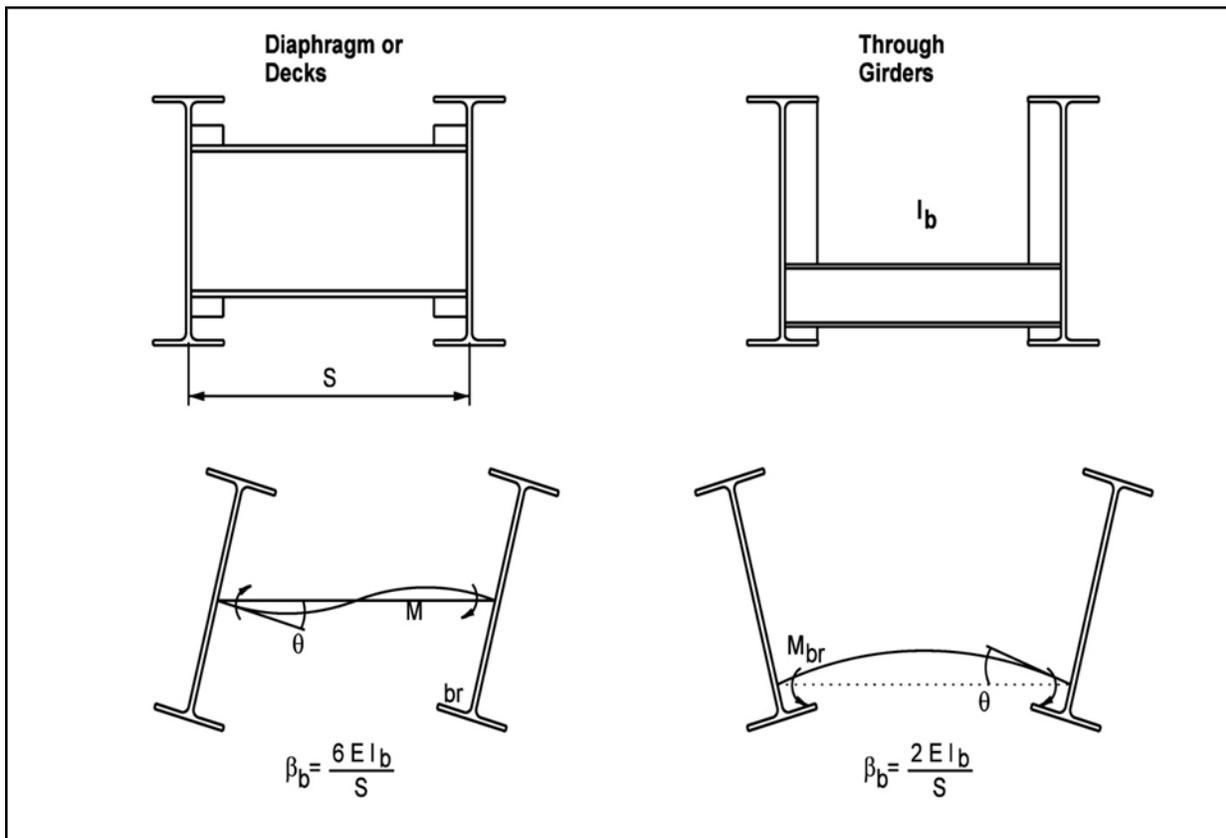


Figure 5-6 Torsional Bracing Stiffness

The brace stiffness, β_b , for common bridge girder bracing systems is shown in Figure 5-6 and Figure 5-7 (from Yura, et al. 2005). The torsional bracing stiffness of the diaphragms in Figure 5-6 depends upon the ability of the diaphragm to force the girder flanges to displace in the same direction. In locations where the girder top flange is in compression, locating the diaphragm above mid-height of the girder will usually cause the compression flanges to displace laterally in the same direction. If the diaphragm is located below midheight, such as cases with a through-girder system, the two flanges will displace in opposite directions resulting in a lower stiffness.

In order to achieve the torsional bracing stiffness shown in Figure 5-7, the connection between the brace and girder must be capable of developing the bracing moment, M_{br} . In most situations, the brace moment is relatively small and can be obtained with simple connections. For example, although a wide flange or channel shape may be used for the diaphragm, it is usually not necessary to connect the flanges of the diaphragm to develop M_{br} .

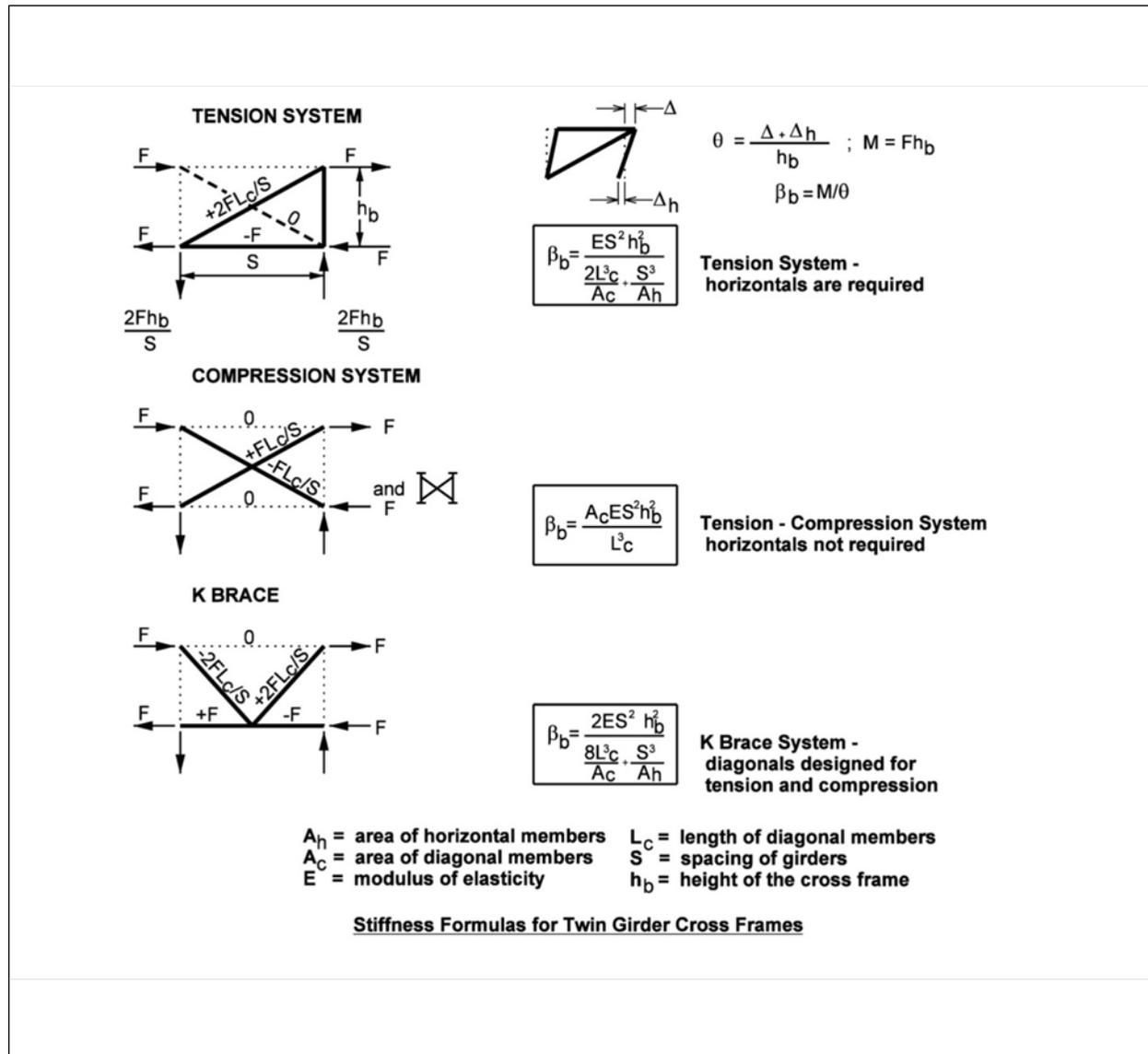


Figure 5-7 Stiffness Formulas for Twin Girder Cross-frames (assumes no connection eccentricity)

Figure 5-7 provides the stiffness of various cross-frame configurations. The stiffnesses shown in Figure 5-7 were derived from an elastic truss analysis. If the truss members in an x-bracing system are designed as tension-only members, then a horizontal member is required. Provided the two diagonals have adequate compression resistance, a compression system can be used with or without the top struts since they are zero force

members. For k-bracing, a top horizontal member should be used, even though it carries no force, to ensure equal lateral top flange displacement.

Cross-frames are often composed of angles with an eccentric connection. Work by Wang (2013) has shown that the eccentricity in the angle connections results in a reduction in the stiffness of the cross-frame. To account for the reduction in the stiffness from the eccentric connections, the stiffness from the equations shown in Figure 5-7 should be reduced by 50% when members with eccentric connections (such as angles) are used. If a tension-only system is considered, there is no reduction necessary due to the conservatism by neglecting the compression diagonal.

The web distortional stiffness, β_{sec} , accounts for the flexibility introduced into the bracing due to deformation of the girder web. The following expression for β_{sec} takes into account the flexibility of both the web and a web stiffener:

$$\beta_{sec} = 3.3 \frac{E}{h_o} \left[\frac{(1.5h_o)t_w^3}{12} + \frac{t_s b_s^3}{12} \right]$$

Equation 5-4

Where the terms are as shown in Figure 5-8, the effective width of the web is taken as $1.5h_o$. For diaphragm connections configured as shown in Figure 5-9, β_{sec} is computed from the stiffness of the separate areas so that $1/\beta_{sec}$ can be computed as shown in Equation 5-5.

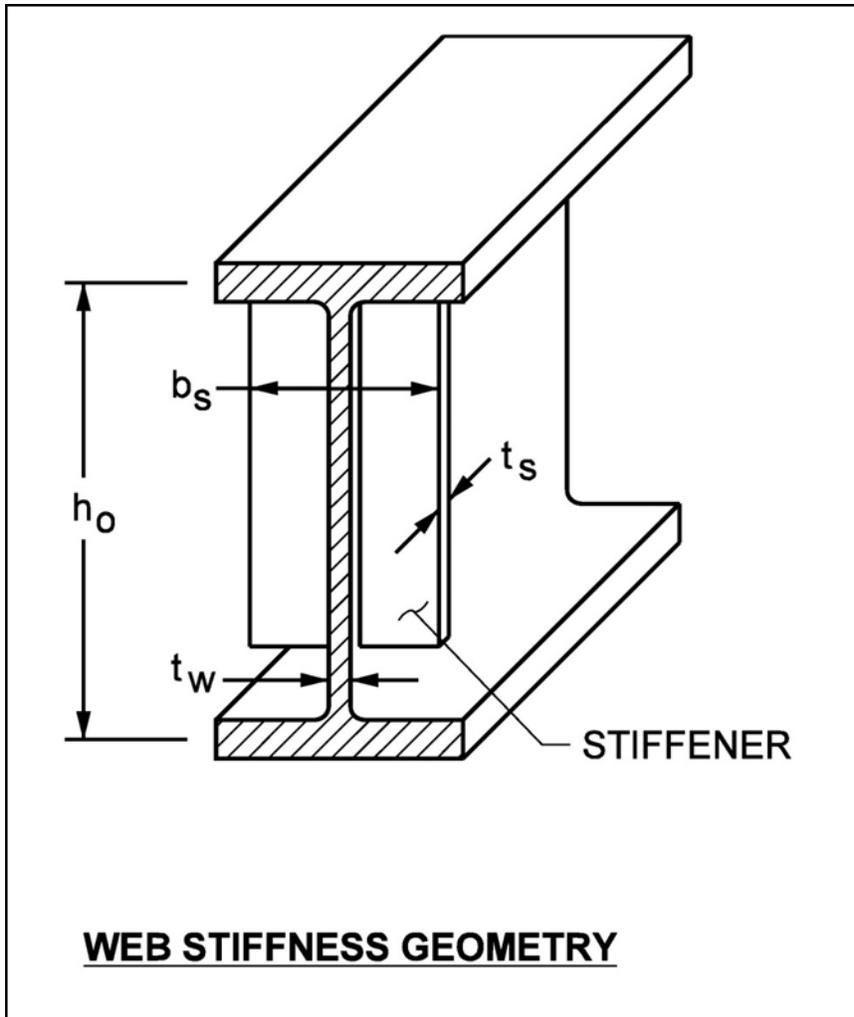


Figure 5-8 Web Stiffness Geometry

$$\frac{1}{\beta_{\text{sec}}} = \sum \left(\frac{1}{\beta_i} \right)$$

Equation 5-5

$$\beta_i = \frac{3.3E}{h_i} \left(\frac{h_o}{h_i} \right)^2 \left[\frac{(1.5h_i)t_w^3}{12} + \frac{t_s b_s^3}{12} \right]$$

Equation 5-6

Compared to Equation 5-4, Equations 5-5 and 5-6 are the more accurate formulations for accounting for the effects of cross-sectional distortion since the web can be divided up into regions above and below the brace. For plate or channel diaphragms or cross-frames, the region of the web within the depth of the brace can be considered rigid

since web distortion will be minimal within the depth of the brace. Therefore, for full depth braces such as diaphragm or cross-frames, the cross-sectional stiffness can be considered infinitely stiff.

The brace moment, M_{br} , at the ends of a cross-frame or diaphragm produces a vertical force couple that acts on the girders. The resulting force couple produces a differential displacement in adjacent girders that reduces the torsional stiffness of the cross-frame. For a brace only at midspan in a multi-girder system, the contribution of the in-plane girder flexibility to the brace system stiffness is (Yura and Philips, 1992):

$$\beta_g = \frac{24(n_g - 1)^2 S^2 EI_x}{n_g L^3}$$

Equation 5-7

Where n_g is the number of girders across the width of the bridge interconnected by the cross-frames.

The effect of girder stiffness decreases as the number of girders increases. For a pair of girders, the β_g factor is significant (Helwig, Frank and Yura, 1997), and the above equation can be used in cases of multiple braces along the span as well. If β_g dominates the torsional brace stiffness, β_b , then a system mode of buckling as discussed in Section 5-4 is possible.

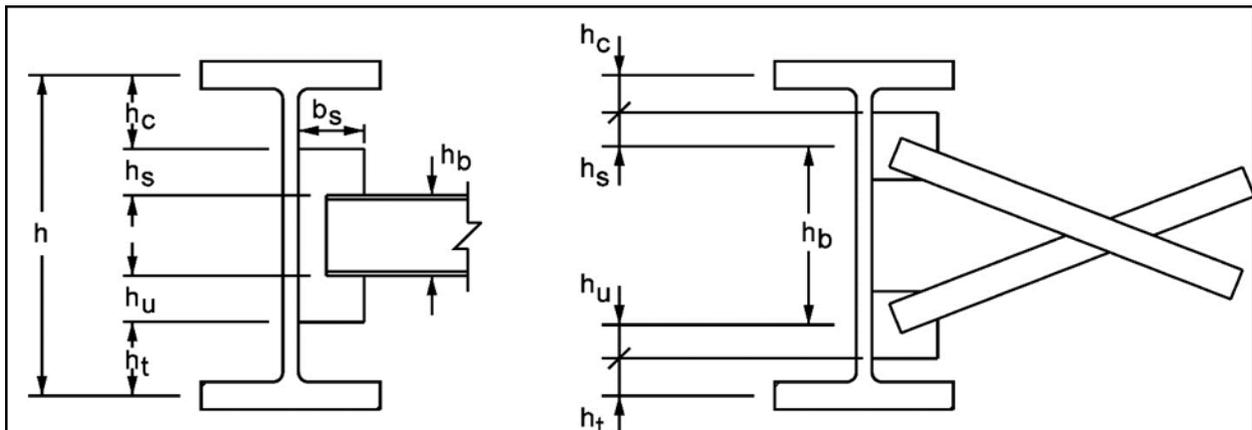


Figure 5-9 Web Stiffness Geometry at Cross-frames

The diaphragm and cross-frame stiffnesses shown in Figures 5-6 and 5-7 assume rigid connections exist between the braces and the girders. For flexible connections such as clip angles bolted only through one leg or welded only along the toe will flex, reducing the system stiffness. For any connection with a given stiffness, β_{conn} , the impact on the

system stiffness can be accounted for by adding the term $2/\beta_{\text{conn}}$ (there is a connection at both ends of the brace) to the right hand side of Equation 5-3.

The brace force requirements are directly proportional to the magnitude of the initial out-of-straightness of the girders (Ziemian SSRC, 2010). The brace force requirements given above were developed for an assumed out-of-straightness of 0.002 L. If oversized holes are used in the bracing details, the brace force will increase if connection slip takes place. This can be accounted for in the design by increasing the magnitude of the brace force by the factor $(1 + \text{oversize}/L_b/500)$.

Besides cross-frames, another frequently used bracing system in steel bridges is a lateral truss that acts as a relative brace to control the movement of a braced point with respect to adjacent braced points. The AISC Specifications provide the following equations for the design of the relative beam bracing:

$$P_{br} = \frac{0.008M_u C_d}{h_o} \quad (\text{Strength}) \quad \text{Equation 5-8A}$$

$$\beta_{br} = \frac{1}{\phi_{br}} \left(\frac{4M_u C_d}{L_b h_o} \right) \quad (\text{Stiffness}) \quad \text{Equation 5-8B}$$

For a lateral brace connecting to a rigid structure (e.g., adjacent bridge), this nodal lateral bracing is governed by the following set of equations from the AISC Specification:

$$P_{br} = 0.02(M_u C_d / h_o) \quad (\text{Strength}) \quad \text{Equation 5-9A}$$

$$\beta_{br} = 10M_u C_d / \phi_{br} L_b h_o \quad (\text{Stiffness}) \quad \text{Equation 5-9B}$$

Where:

- P_{br} = The required brace strength (kip)
- β_{br} = The required brace stiffness (kip/in)
- h_o = distance between flange centroids (in)
- L_b = length between brace points (in)
- M_u = maximum factored moment (kip-in)
- C_d = 1.0 except as follows:
2.0 for the brace closest to inflection point for a beam subject to double curvature
- ϕ_{br} = 0.75

For design of the overall girder bracing system, the girder stability-induced bracing forces computed from the above equations should be added to the bracing forces due to wind, skew, curvature and any other induced effects. The total bracing force, due to the combined effects, is then used for member design.

5.3.2 Skew Effects

Skew angles, as shown in Figure 5-10, increase the interaction between the girders and bracing. When the cross-frames are oriented perpendicular to the girder longitudinal axis, the skew angle does not impact the stability-induced forces and the provisions previously outlined can be used for the bracing design. However, the bridge deflections in a bridge with skewed supports do vary across the width of the bridge creating differential deflections between the ends of the bracing. For larger skew angles, the differential deflections can result in significant forces induced into the cross-frame that are additive to the stability-induced forces. For skew angles larger than approximately 45 degrees, the forces from the differential deflection often exceed the stability-induced forces. The forces due to differential deflections should be added to the stability forces when designing the bracing. Forces in the bracing resulting from casting of the bridge deck can generally be satisfactorily predicted from a first-order computer model of the girders and bracing system.

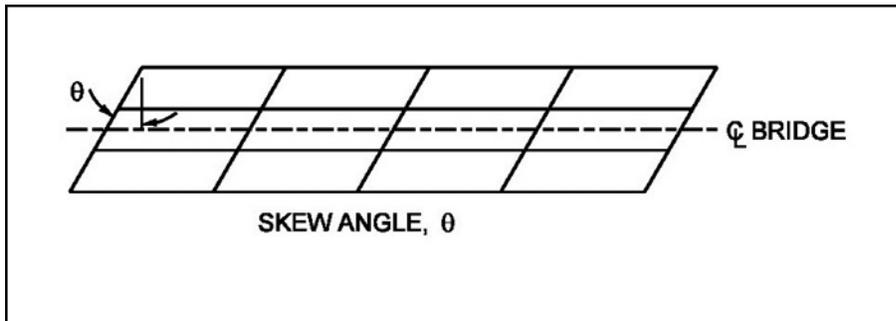


Figure 5-10 Brace Orientations for Bridges with Skewed Supports

Although AASHTO requires cross-frames to be oriented perpendicular to the girders for skew angles larger than 20 degrees, for skew angles less than 20 degrees, the intermediate braces can be oriented parallel to the skew angle. When the cross-frames are oriented parallel to the skew angle, the skew angle impacts both the stability stiffness and strength requirements for the bracing. Expressions for the required strength and stiffness are given as (Wang and Helwig, 2008):

$$\beta_{\text{bskew}} = \frac{\beta_b}{\text{Cos}^2\theta}$$

Equation 5-10

Where β_b is given in Equation 5-6 and 5.7, $\beta_{b\text{skew}}$ is then used in the previous equations in place of β_b . The value “s” in the stiffness equations should be the skewed bracing length as opposed to the girder spacing. The moment in the skewed bracing then becomes:

$$M_{\text{brskew}} = \frac{M_{\text{br}}}{\text{Cos}\theta}$$

Equation 5-11

SECTION 4. SYSTEM BUCKLING OF GIRDERS

The most common braces that are utilized in steel bridge systems are cross-frames or diaphragms that control twist of the adjacent girders that they connect. The unbraced length, L_b , that designers typically use in buckling solutions such as the expressions shown in Chapter 4, is the distance between the cross-frames along the length of the bridge. In most applications, the cross-frame locations do behave as braced points against girder buckling. However, the cross-frames may not serve as a brace point for systems with a relatively large length-to-width ratio in which case the girder systems are susceptible to a system mode of buckling that is relatively insensitive to the spacing between the cross-frames.

Twin girder systems or cases during construction when only a few girders have been erected can produce systems with a large length-to-width ratio. There have been a number of applications during construction in which the girders have been close to buckling in the system mode. For example, the two-girder widening shown in Figure 5-11 had relatively close cross-frames, however the girder experienced significant twisting during placement of the concrete deck as evidenced by the 10 in. lateral deformation of the bottom flange relative to the plumb line. The load on the twin girder system was balanced and did not have an eccentricity. The behavior exhibited in the bridge widening was very nearly a buckling failure of the entire girder system as described below.

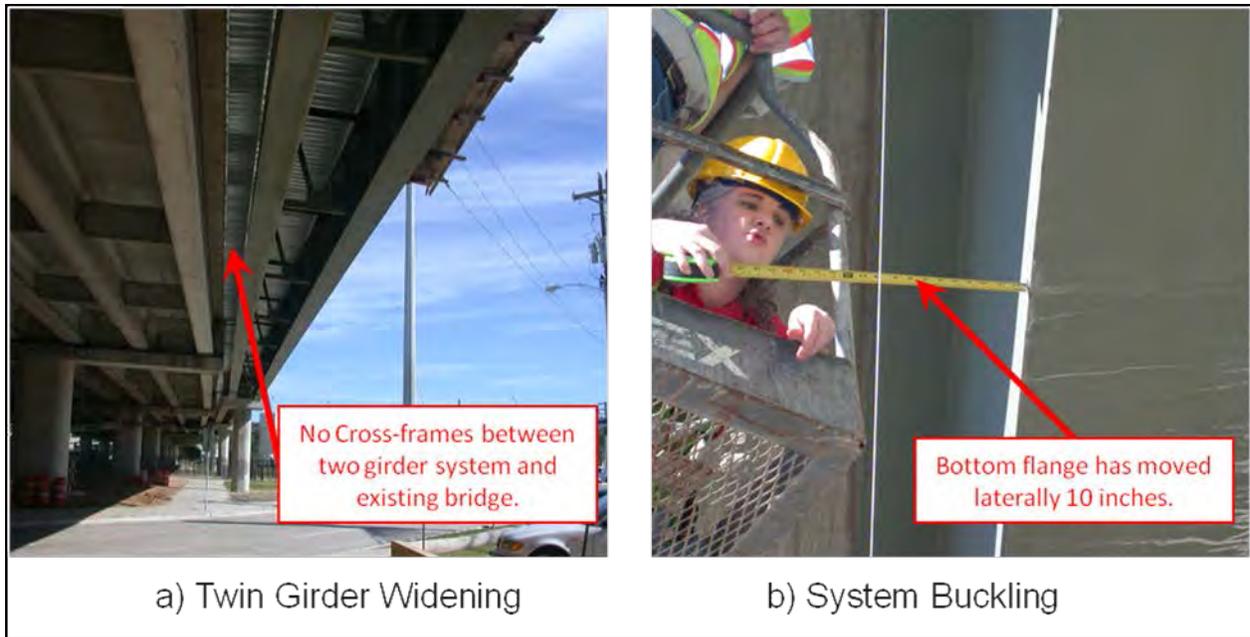


Figure 5-11 System Buckling of a Twin Girder Widening

The buckled shape of the compression flange that is typically envisioned in a properly braced girder system is depicted in Figure 5-12(a), which shows a plan view of a twin girder system. By reducing the spacing between the braces, the engineer can reduce the size of L_b and thereby improve the buckling capacity of the girders that results from the lateral-torsional buckling expressions. However in girder systems with a relatively large length to width ratio, the controlling mode is the buckled shape depicted in Figure 5-12(b). In the system buckling mode, the girder system behaves as a unit and the resulting resistance is not significantly affected by the spacing or size of the braces.

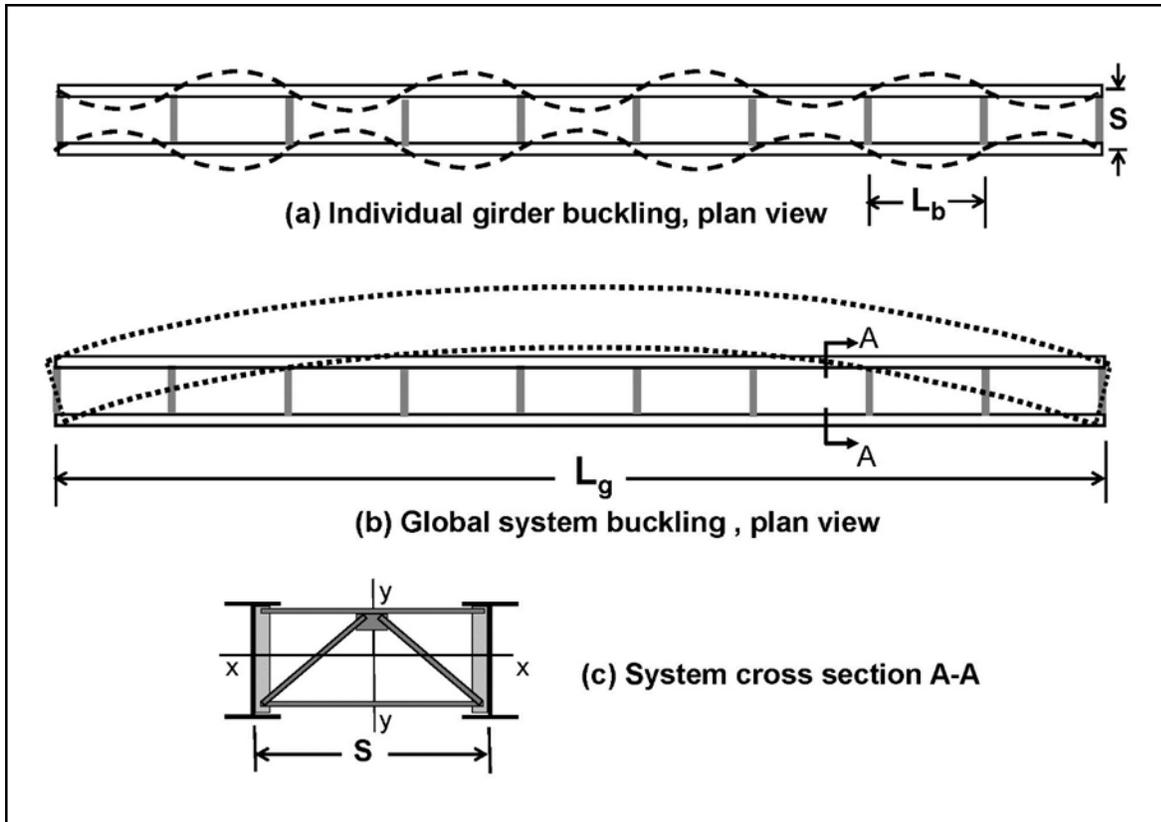


Figure 5-12 System Buckling Mode

Yura et al. (2005) presented the following solution for doubly-symmetric girders that can be used to evaluate the buckling capacity of girders in the system buckling mode:

$$M_{gs} = \frac{\pi^2 SE}{L_g^2} \sqrt{I_y I_x}$$

Equation 5-12

Where:

- M_{gs} = nominal buckling resistance of the girder system (kip-in)
- S = the girder spacing (in)
- L_g = the total length of the girder (in)
- E = the modulus of elasticity of the steel girder (ksi)
- I_y = the moment of inertia of a single girder about the weak axis (in^4)
- I_x = the moment of inertia of a single girder about the strong axis (in^4)

The expression can be divided by the number of girders, n , to estimate the capacity of one of the girders for comparison with the girder design moment. A resistance factor, ϕ , of 0.9 should be used to compare capacity with the factored design moment, M_u .

For singly-symmetric girders, I_y , in Equation 5-12 can be replaced with I_{eff} (Yura et al. 2005):

$$I_{eff} = I_{yc} + \frac{t}{c} I_{yt}$$

Equation 5-13

Where:

I_{yc} and I_{yt} = the respective moments of inertia of the compression and tension flanges about an axis through the web

c and t = the respective distances from the centroidal axis to the compression and tension flanges as shown previously in Figure 5-5.

For a doubly-symmetric section, I_{eff} given by Equation 5-13 reduces to I_y since $c = t$.

Equation 5-12 is a closed form solution that can be used to evaluate the system buckling capacity of twin girder systems. For a three girder system, replace I_{yc} in Equation 5-13 with $3/2 I_{yc}$, and define S in Equation 5-12 as the distance between the two outside girders (twice the spacing between adjacent girders). For a four girder system, replace I_{yc} in Equation 5-13 with $2I_{yc}$ and define S in Equation 5-12 as the distance between the two outside girders (or $3S$, where S is the girder spacing).

Equation 5-11 shows that for a given girder span (L_g), the system buckling mode can be improved by either increasing the stiffness of the individual girders or by increasing the girder spacing. An alternative method of improving the buckling capacity is presented in Yura et al (2005) and consists of adding a few panels of a top and bottom flange lateral truss near the ends of the girders.

Lateral load effects, when combined with the system global buckling resistance calculated in Equation 5-12, may be computed using Equation A6.1.1-1 from the

$$\frac{\phi M_{gs}}{n}$$

AASHTO Specifications, where the $\frac{\phi M_{gs}}{n}$ term replaces the $\phi_f M_{nc}$ term on the right hand side of the equation.

$$M_{gs} = \frac{\pi^2 SE}{L^2} \sqrt{I_{eff} I_x}$$

The Yura equation: $M_{gs} = \frac{\pi^2 SE}{L^2} \sqrt{I_{eff} I_x}$ has recently been incorporated into AASHTO LRFD as Eq. 6.10.3.4.2-1 for checking system buckling during deck pour. The approach presented above is applicable for intermediate steel checks prior to the deck pour

($M_u \leq \phi M_{gs} / n$), but the AASHTO approach omits the ϕ factor and limits the total sum of the factored positive girder moments to 50% M_{gs} during the deck pour (Strength VI load combination). Should the sum of the moments exceed 50%, the design can add flange level lateral bracing, revise the girder spans/sizes to increase system stiffness, or evaluate the amplified girder second-order displacements and verify that they are within owner tolerances. Note that amplification can also occur under steel-only dead load as the buckling limit is approached, but the recommended system buckling ϕ factor and Strength I/III load factors should provide an adequate level of safety for most narrow systems subject to buckling in the steel-only condition.

SECTION 5. BRACING TO ADJACENT STRUCTURES

Adjacent structures, adjacent portions of the existing structure, or adjacent portions of the newly-constructed structure, can be used as brace points. In these situations, guy wires are commonly used for bracing elements. However, when guy wires are long, one needs to account for the elastic deformation of the guy wires. The flexibility of the long cable needs to be considered in evaluating the stiffness requirements. Excessive elastic elongation may lead to a reduction of the anticipated bracing restraint as well as unanticipated deflection of the member being braced. This is illustrated in Figure 5-13. For guys attached to deadmen, the deadmen need to be designed for pullout of the anchorage, as well as global stability (sliding and overturning). In cases where the girder may deflect from additional loading added after the brace is installed (such as concrete placement), a turnbuckle should be included in the guy wire so that the slack can be removed from the brace due to the girder deflection. In continuous girder applications, the brace may develop additional tension when concrete is placed in an adjacent span and the use of a turnbuckle can also be used to release the added tension.

In addition, when external bracing is provided, the bracing force can lead to forces in the member being braced for which the designer did not account. For this reason, bracing points need to be evaluated to ensure that local and global stability is satisfied, and that the resulting forces do not result in overstressing the braced member. Additional supports, tie-downs and stiffening elements are frequently required in these cases.

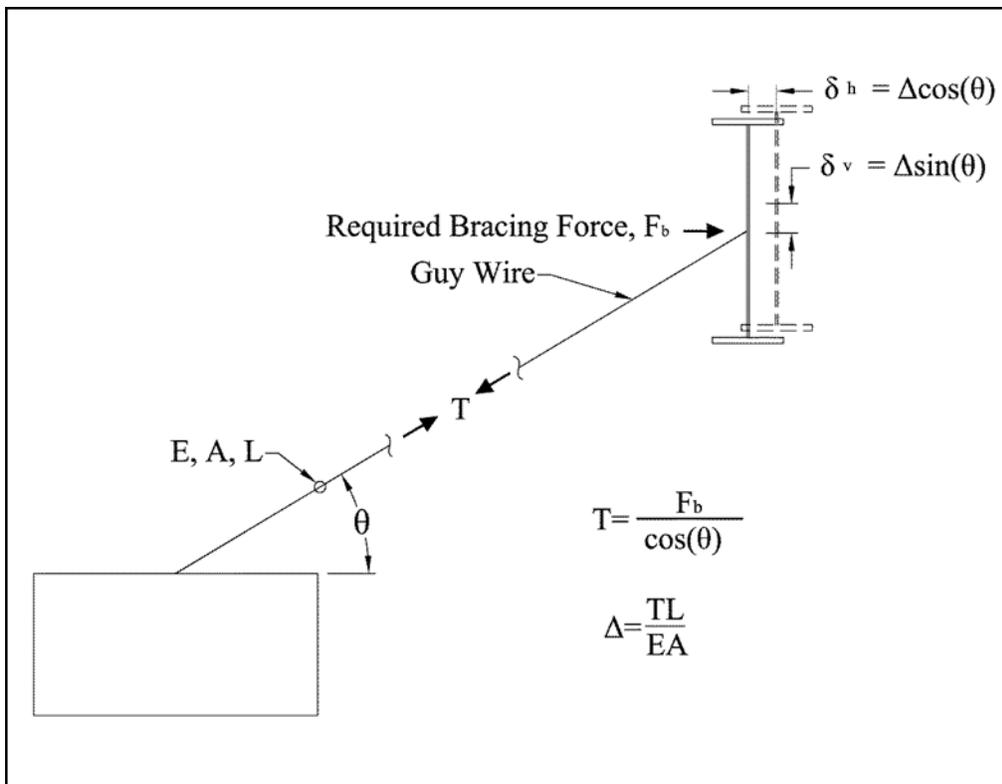


Figure 5-13 Elastic Deformation of Guy Wire Bracing

SECTION 6. Uplift During Erection And Deck Pour

5.6.1 Uplift Considerations

Uplift can occur at supports due to a variety of different scenarios. One of the most common causes for uplift is due to unbalanced loading during a deck pour sequence in continuous girder construction. Uplift can also occur at the corners of skewed bridges. Uplift can also occur at supports for partially erected spans due to the primary member dead weight alone, particularly in horizontally curved girders. Uplift considerations are much more common for steel bridges because the self-weight of a steel primary member is generally significantly less than that of a concrete primary member. Therefore, the discussion in this section will be focused on steel girder bridges, but the reader should bear in mind that potential uplift should also be investigated for concrete girder bridges.

For multiple-span bridges, a deck pour sequence must be developed that minimizes construction stresses in the primary member as well as the deck. In continuous span girders, contractors often like to have the option for a continuous concrete casting sequence from one end of the bridge to the other. Although a continuous casting sequence is desirable for ease of construction, such a scenario creates one of the most likely circumstances for uplift at a support. Uplift can also occur even when a segmented casting sequence is used. Typically, the positive moment sections are poured first, followed by the negative moment sections. This procedure is utilized to minimize undesirable cracking in the top of the deck in the negative moment areas.

For two-span bridges a portion of one of the spans (from the end support to the approximate point of contraflexure) will be poured first, followed by a similar portion of the other span, followed by the negative moment area. As depicted in Figure 5-14 and Figure 5-15, when the first span is poured, net uplift may occur at the opposite end support.

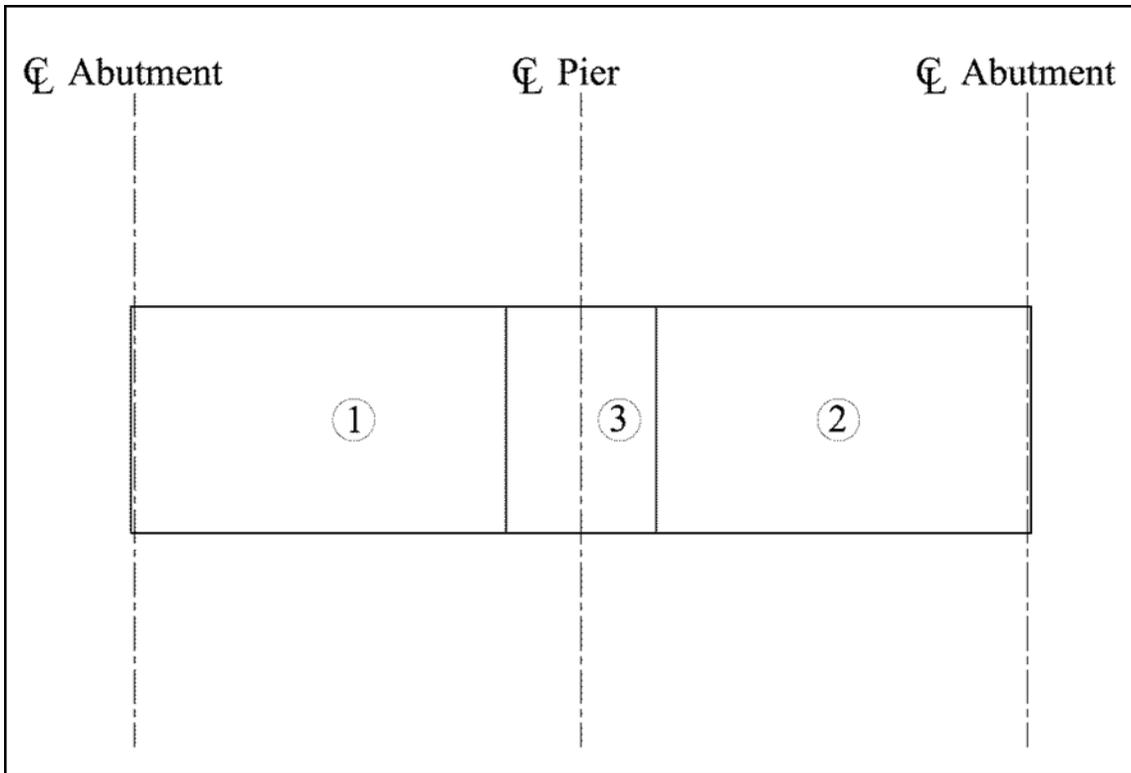


Figure 5-14 Typical Pour Sequence for Two Span Bridge

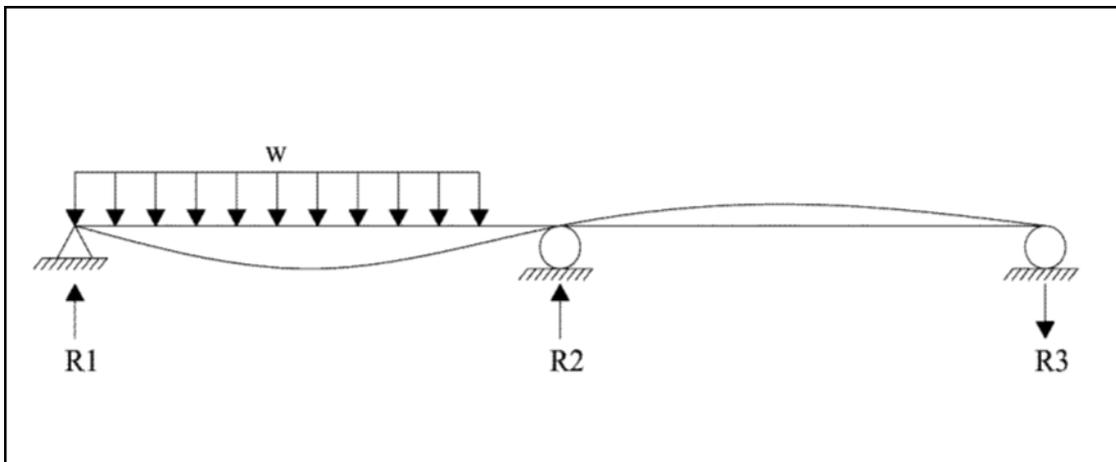


Figure 5-15 Uplift Effect on Two Span Deck Pour

This is because the necessary downward reaction (R_3) to the dead load of the wet concrete in the first span being poured will generally be greater than the upward reaction caused by the self-weight of the steel girder alone (See Figure 5-15), therefore leading to uplift at the support. Tie-downs will almost always be required in this case. The effect of uplift can be reduced by adjusting the length of the first stage of the concrete pour. If the span lengths are different, the best solution is generally to pour the shorter span first.

For three-span bridges, a portion of one of the end spans is generally poured first. This is because if the center span were poured first, uplift is likely to occur at both end supports. By pouring one of the end spans first, uplift may occur at the opposite interior support. However, due to the increased continuity of a three-span bridge compared to a two-span bridge, uplift occurs less frequently for a three-span bridge than for a two-span bridge, unless one of the outside spans is relatively short. Once again, the potential for uplift, as well as its magnitude, can be reduced by pouring the shorter end span first. After the shorter end span is poured, the other end span is typically poured, followed by the center span, and finally, the negative moment areas (see Figure 5-16).

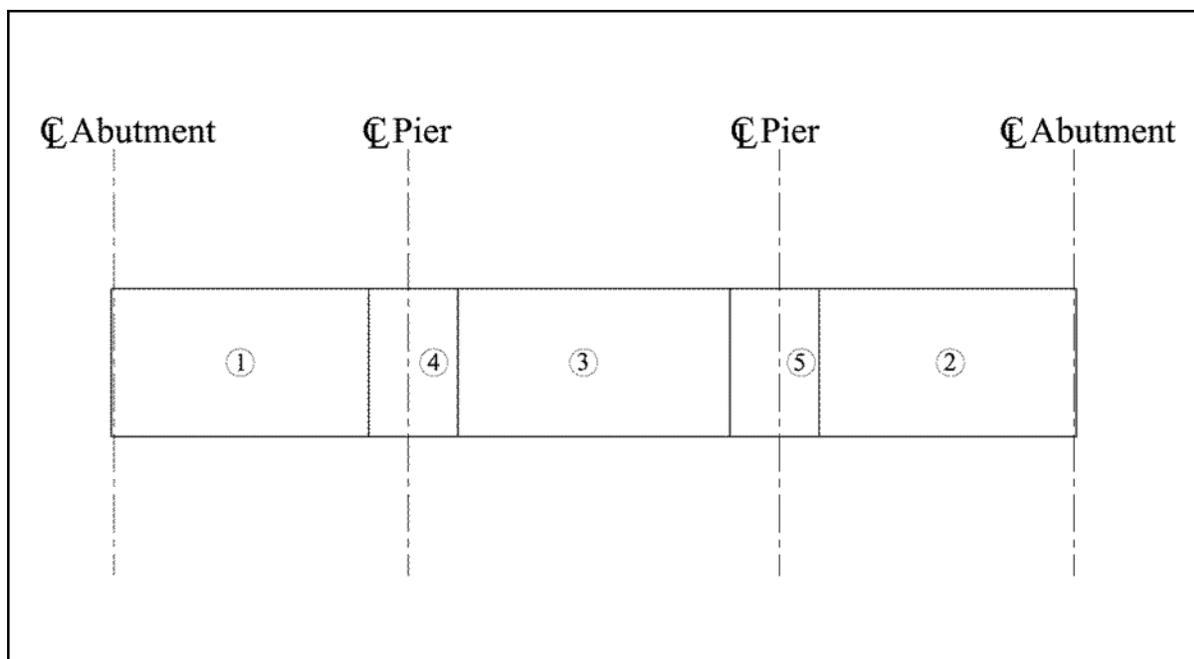


Figure 5-16 Typical Pour Sequence for Three Span Bridge

Bridges that consist of four or more spans should also be checked for uplift that could occur during deck pouring, but uplift occurs for these types of bridges less frequently than for two- or three-span bridges.

It should be noted that the foregoing is typically used as a starting point when developing pour sequences; however, other considerations may govern, such as equipment placement, access, construction staging and others that may require modifying the pour sequence to suit the specific condition encountered.

5.6.2 Additional Deck Pour Considerations

When developing the deck pour sequence, one should be aware that the engineer typically designs the bridge elements for the final condition of the fully poured deck, and that additional stresses imparted due to the contractor's choice of deck pour sequence are often not considered by the original designer. Even if a suggested pour sequence is

indicated in the design plans, it is important that this pour sequence is analyzed to account for the contractors' means and methods.

Consider the two-span bridge shown in Figure 5-15. The designer would typically check the member stresses assuming the fully cast deck is in place, which would generally be composite with the steel in the positive moment regions and may or may not be considered to be composite with the reinforcing bars in the negative moment regions. However, when the first span of the deck is poured, the stresses in the member in that span will exceed the design dead load stresses because the counteracting stresses from the concrete in the opposite span are not in place. Further, since the concrete in the first span will often have cured and become composite, when the second span is poured the reduction in stress in the member in the first span will be less than if the entirety of the deck were to have been poured at one time.

Girder camber should also be calculated based on the chosen deck pour sequence. These can sometimes be markedly different than the final condition cambers that are typically computed by the designer, particularly for two-span bridges. For single-span and multiple (greater than two) span bridges, cambers computed based on a deck pour sequence are usually close to those computed for the final condition, except for unusual (unbalanced) span configurations and very long spans.

SECTION 7. ROLL STABILITY OF CURVED SPANS

The geometry of horizontally curved girders generally leads to a de-stabilizing or overturning moment that needs to be considered in the erection plan. The overturning moment is the result of the offset of the center of gravity with respect to the line of support of the girder as shown in Figure 5-17. As shown in the figure, the axis of rotation is defined by a line connecting the end points of support of a curved girder segment. The center of gravity can be approximated by assuming that the girder is a line segment (i.e. has zero width).

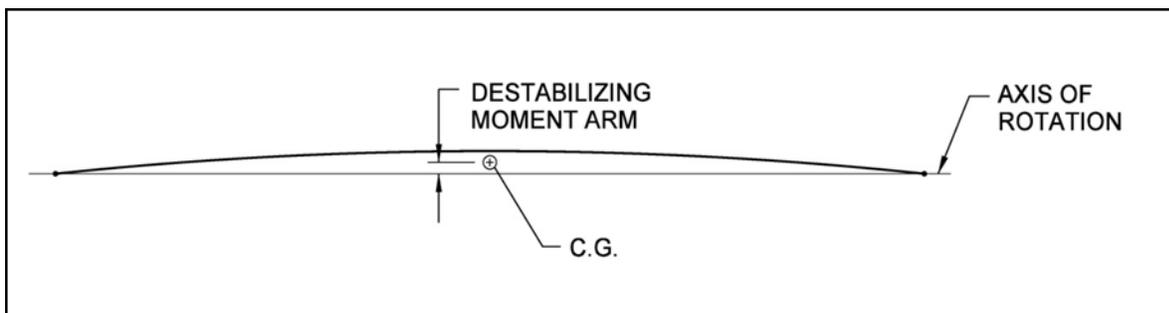


Figure 5-17 Center of Gravity and Destabilizing Moment Arm of Single Curved Girder

This approximation leads to the center of gravity being defined as:

$$\bar{x} = R \sin \alpha / \alpha \quad \text{Equation 5-14}$$

Where:

- \bar{x} = distance from the center of curvature to the center of gravity of the curved girder segment (ft)
- R = radius of curvature of the curved girder segment (ft)
- α = one-half of the degree of curvature of the curved girder segment (rad)

The distance from the center of curvature to the axis of rotation is:

$$X = R \cos \alpha \quad \text{Equation 5-15}$$

Therefore, the overturning moment arm is:

$$\bar{x} - X \quad \text{Equation 5-16}$$

As an example, for a curved girder segment with a radius (R) of 1000 ft. and a length (L) of 200 ft., the overturning moment arm can be computed as follows:

$$\begin{aligned} \alpha &= L / 2R = 200 \text{ ft} / (2 * 1000 \text{ ft}) = 0.1 \text{ rad} \\ \bar{x} &= R \sin \alpha / \alpha = 1000 \text{ ft} * \sin(0.1 \text{ rad}) / 0.1 \text{ rad} = 998.33 \text{ ft} \\ X &= R \cos \alpha = 1000 \text{ ft} * \cos(0.1 \text{ rad}) = 995.00 \text{ ft} \\ \bar{x} - X &= 998.33 \text{ ft} - 995.00 \text{ ft} = 3.33 \text{ ft} \end{aligned}$$

In cases where overturning is a concern, intermediate points of support, such as shoring towers or holding cranes can be added to reduce the likelihood for such problems. However, in any case, erecting a single curved girder will require significant bracing to ensure that the segment in question does not overturn.

The potential for overturning is dramatically reduced, and in most cases, eliminated, by erecting curved girder segments in pairs, as depicted in Figure 5-18. In this case, the axis of rotation moves out to the line connecting the end points of support of the outermost curved girder segment. The effect of a pair of girders can be demonstrated with the same geometry as in the above example. For a girder spacing of 10 ft. and properly designed cross-frame bracing between the girders, the calculation of overturning moment is as follows:

For the inside girder (Girder 1):

$$\alpha_1 = L_1 / 2R_1 = 200 \text{ ft} / 2(2 * 1000 \text{ ft}) = 0.1 \text{ rad as above, and}$$

$$\bar{x}_1 = R_1 \sin \alpha / \alpha = 1000 \text{ ft} * \sin(0.1 \text{ rad}) / 0.1 \text{ rad} = 998.33 \text{ ft as above.}$$

For the outside girder (Girder 2):

$R_2 = 1010 \text{ ft}$ and $L_2 = 202 \text{ ft}$, which leads to:

$$\alpha_2 = \frac{L_2}{2R_2} = 202 \text{ ft} / (2 * 1010 \text{ ft}) = 0.1 \text{ rad}$$

$$\bar{x}_2 = \frac{R_2 \sin \alpha_2}{\alpha_2} = 1010 \text{ ft} * \sin(0.1 \text{ rad}) / 0.1 \text{ rad} = 1008.32 \text{ ft}$$

The center of gravity of the pair of girders is:

$$\bar{x} = \frac{(\bar{x}_1 + \bar{x}_2)}{2} = 1003.33 \text{ ft}$$

The distance from the center of curvature to the axis of rotation is:

$$X = R_2 \cos \alpha_2 = 1010 \text{ ft} * \cos(0.1 \text{ rad}) = 1004.95 \text{ ft}$$

The resulting moment arm is then:

$$\bar{x} - X = 1003.33 \text{ ft} - 1004.95 \text{ ft} = -1.63 \text{ ft}$$

Since the moment arm is negative, there is no overturning moment and the pair of curved girder segments will be globally stable for roll over.

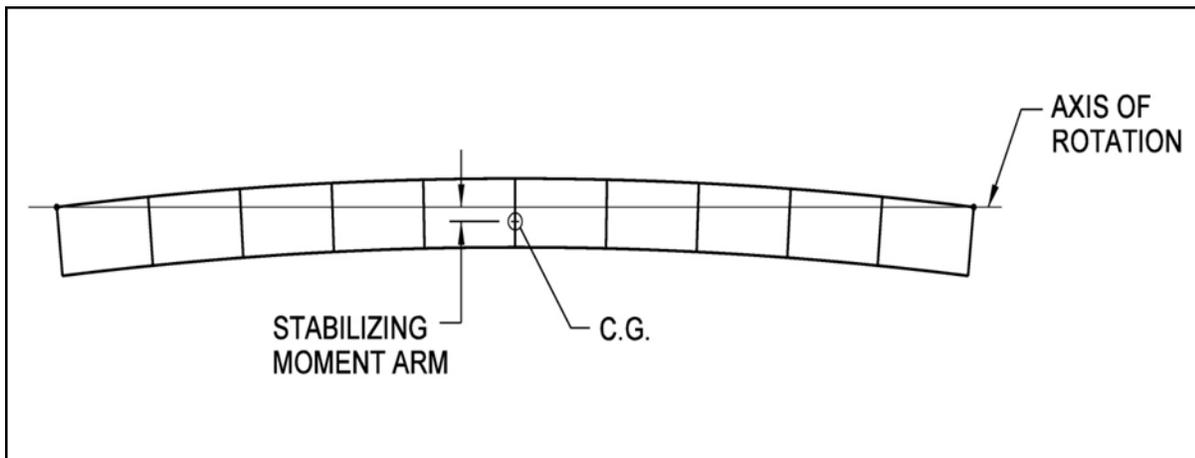


Figure 5-18 Center of Gravity and Stabilizing Moment Arm of a Pair of Curved Girders

SECTION 8. **ROLL STABILITY OF FASCIA GIRDERS**

The torsional loading from the deck overhang can potentially cause problems in both concrete and steel girder bridges during construction. The main issue with concrete girder bridges is that the overhang load can generate excessive torsional rotation in the fascia girder. For steel girder bridges, the torque from the overhang can lead to both global and local stability issues. Most global stability issues with the overhangs occur in bridge widening projects. The widening is often isolated from the original construction to permit vertical deflections during deck casting. Therefore, the widening often consists of a two- or three-girder system with a large length-to-width ratio.

From a lateral-torsional buckling perspective, the girders are susceptible to the system buckling mode outlined in Section 5.4 that is relatively insensitive to the spacing between intermediate cross-frames. The low resistance to lateral torsional buckling, coupled with the torque from the overhang brackets, has led to systems that may have been dangerously close to failure. In addition to the global stability issues, potential problems are related to the local stability of the girder webs. In many instances, the overhang brackets exert large concentrated forces on the webs of the steel girders. The forces from the overhang bracket can distort the web, thereby leading to local instabilities or large web imperfections that can get locked into the girders once the deck cures.

Many bridge owners specify limits on overhang width in order to keep the torsional load on the fascia girder reasonably balanced. Article 6.10.3.4 of the AASHTO LRFD Bridge Specifications provides a method of computing the flange lateral bending stress in steel girders due to overhang brackets.

The torsional loads can be resisted by the use of ties, usually reinforcing bars placed above the top flange, and timber compression struts located at the bottom flange. The computer design tool TAEG (Torsional Analysis for Exterior Girder), developed by K-TRAN, is a public domain program that can be used to evaluate these systems including girder and cross-frame forces.

Yang et. al. (2010) have developed a recommended design procedure for overhangs on concrete I-girder bridges for both flexible and stiff connections. For flexible connections, the design procedure is based on the eccentric loading causing rigid body rotation of the I-girder. The limiting rotation in their procedure is the rotation that causes lift-off of the girder from the elastomeric bearing pad over $\frac{1}{4}$ of the pad width. The bearing pad stiffness after lift-off actually changes depending upon the axial load, but this is complex to determine and the refinement may not be warranted. The rotation is further restricted by limiting the angle of rotation to 0.5 degrees based on serviceability; Figure 5-19 shows a free-body diagram of the fascia girder/overhang model; for notation see Equation 5-17.

The girder rotation causes the deck reinforcing steel mat to also rotate, which can reduce the concrete cover over the reinforcing. The Florida DOT, for example, limits girder rotation so that the maximum reduction in cover is less than 1/4-inch. Bracing may be required to control this rotation.

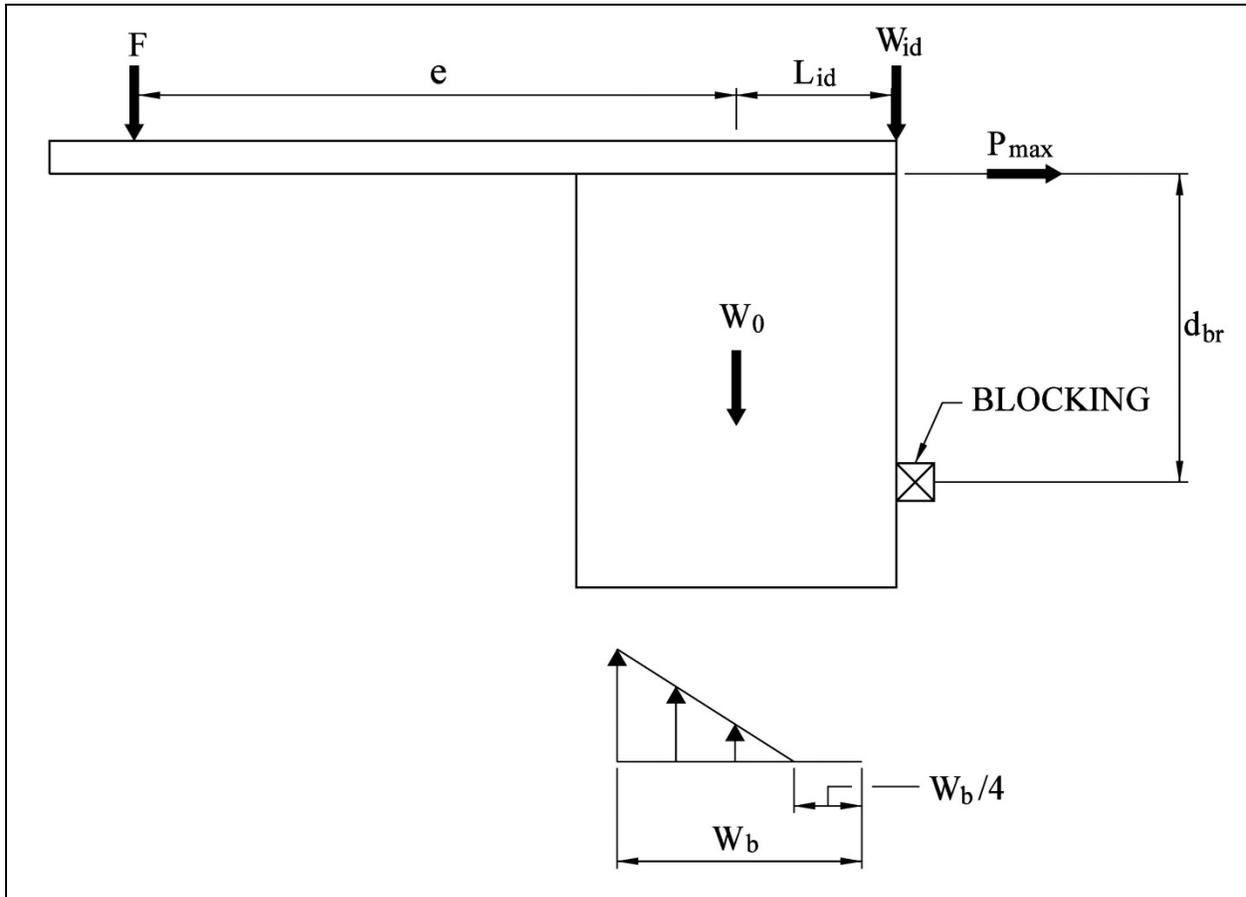


Figure 5-19 Free-body Diagram of Concrete Beam with Eccentric Overhang Loading at Lift-off

The summarized procedure is as follows:

Step 1 – Calculate the effective eccentric force and its eccentricity. Forces to be considered include the weight of the fresh concrete on the overhang, the weight of the work bridge and workers, the weight of the finishing equipment, and the weight of the formwork. The analysis is based upon simple statics.

Step 2 – Calculate the quarter point lift-off force and check it against the effective eccentric force.

$$F_{QPL} = \frac{4W_{id}L_{id} + 4P_{max}d_{br} + w_b(W_o + W_{id})}{4e - w_b}$$

Equation 5-17

Where:

- F_{QPL} = quarter point lift-off force (kips)
- W_{id} = weight of half of slab between fascia girder and 1st interior girder (kips)
- L_{id} = eccentricity of W_{id} (in)
- P_{max} = yield capacity of bracing (kip)
- d_{br} = distance from top of beam to blocking (in)
- w_b = width of elastomeric bearing (in)
- W_o = weight of beam + haunch (kips)
- e = eccentricity of overhang force (in)

Step 3 – Check beam rotations to ensure angles of rotation (θ_1 and θ_2) are less than 0.5 degrees.

$$\theta_1 = \frac{12}{k_b w_b^3} (F e - W_{id} L_{id} - P_{max} d_{br}) (180 / \pi)$$

Equation 5-18

$$\theta_2 = \frac{8}{9 k_b} \frac{(F + W_o + W_{id})^3}{\left[(-2e + w_b) F + w_b (W_o + W_{id}) + 2L_{id} W_{id} + 2d_{br} P_{max} \right]^2} (180 / \pi)$$

Equation 5-19

Where:

- F = eccentric force (kips)
- θ_1 = angle of rotation at lift-off (deg)
- θ_2 = angle of rotation after lift-off (deg)
- k_b = combined stiffness of two fascia girder elastomeric bearings (k/in per in width of bearing)

For stiff connections, the design procedure is based on rupture of the reinforcing bar. The summarized procedure is as follows:

Step 1 – Calculate the effective eccentric force and its eccentricity. Forces to be considered include the wet weight of the overhang concrete, the weight of the work bridge and workers, the weight of the finishing equipment, and the weight of the formwork. This is done using simple statics.

Step 2 – Check for rupture of reinforcing bar by ensuring that the beam rotation at rupture, θ_{BrY} , is less than the beam rotation for a given overhang width, θ .

$$\theta_{BrY} = \frac{k_{st} + k_{wd}}{k_{st} k_{wd} d_{br}} P_{max} (180 / \pi)$$

Equation 5-20

$$\theta = \frac{(F_e - W_{id} L_{id})}{\left(\frac{k_b W_b^3}{12} + \frac{k_{st} k_{wd} d_{br}^2}{k_{st} + k_{wd}} \right)} (180 / \pi)$$

Equation 5-21

Where:

- θ_{BrY} = angle of rotation at reinforcing bar rupture (rad)
- θ = angle of rotation for a given overhang (rad)
- k_{st} = combined stiffness of top bracing bars (k/in)
- k_{wd} = combined stiffness of wood blocking (k/in)

SECTION 9. ADDITIONAL CONSIDERATIONS FOR STEEL BOX GIRDERS

Erecting steel box (also known as tub) girders present some issues that are not addressed elsewhere in this Manual. This section describes, in general terms, some of these issues and how they should be addressed.

5.9.1 Differential Deflections

For multi-girder I-girder type bridges, cross-frames or diaphragms are almost always provided by the designer to provide bracing against buckling during throughout the construction process. In addition to stability bracing, the cross-frames also control construction-induced differential deflections between the girders.

For steel box girders, however, the design typically calls for top flange lateral bracing, which serves to provide torsional stiffness to the box section by developing a quasi-closed section. This lateral bracing also serves as brace points for the top flanges during the wet concrete stage. Due to the large torsional stiffness of the girders, both in the quasi-closed state and in the finished bridge, intermediate external cross-frames or diaphragms are usually not necessary.

The absence of permanent external cross-frames could present issues with respect to differential deflection between adjacent girders. Depending on the contractor's chosen pour sequence and methodology, along with any potential changes to the staging that may have occurred during construction, temporary external cross-frames, if not specified on the design plans, may be required.

5.9.2 Top Flange Lateral Bracing

As stated above, top flange lateral bracing is typically provided in steel box girders to develop a quasi-closed section in order to provide torsional stiffness. The top flange lateral bracing controls differential longitudinal displacements between the two webs; thus, this bracing must be designed to resist the warping stresses (both torsional and normal) induced in the box girder. The need for top flange lateral bracing is especially critical for curved steel box girders, since these braces are primary members that actively participate in the distribution of force due to the curvature.

WT-sections are often used for the top flange lateral truss in tub girders. Since the warping stresses are greatest near the ends of the girders, the strength requirements for the design of the top flange lateral bracing are greatest at these locations. Often (particularly in the case of curved girders), top flange lateral bracing is provided throughout the span, although the section and/or spacing may be reduced near the center of the span.

The bridge designer would have designed the top flange lateral bracing for the final erected condition (in addition to the deck pour stages shown on the plans). Therefore, the erection engineer should evaluate the potential need for more robust top flange lateral bracing for the various temporary conditions that reflect the contractor's chosen means and methods, including, but not limited to the contractor's chosen deck pour sequence. If the erection engineer adds top flange lateral bracing, he should avoid the use of fatigue-sensitive connection details that may result from this change. These details are usually avoided by providing bolted connections to the primary member in all tension or reversal stress zones.

5.9.3 Bottom Flange Stiffeners

Bottom flanges for steel box girders are inherently wide. For simply-supported box girders, the bottom flange will be in tension during all stages analyzed by the designer. Because the design of a tension flange is based purely on the required cross-sectional area of the flange to resist the design stresses (or moments), the design typically results in a thin bottom flange. Moreover, due to the bottom flange being designed for a tension-only condition, the designer typically will not provide bottom flange stiffeners.

For continuous steel box girders, the bottom flanges at the intermediate support locations are designed to resist the design compression forces; therefore, the bottom flange thickness in these locations is more robust. This required thickness sometimes results in a bottom flange with a width-to-thickness (b/t) ratio that satisfies the AASHTO threshold for not requiring a bottom flange stiffener, even though the flange may experience significant compressive force.

During construction, however, the contractor's chosen means and methods often will result in compressive forces in the bottom flanges of steel box girders well in excess of

those envisioned by the designer. Intermediate shoring and the contractor's chosen pick points are two of the primary reasons for this. The contractor's engineer should therefore be aware of the potential need for bottom flange stiffeners for erection.

Bottom flange stiffeners are most often T-sections that are longitudinally welded to the bottom flange at the center of the flange. When provided for erection purposes, these stiffeners are usually left in place. If this is the case, the allowable stress associated with the fatigue category of the weld should be checked against the design live load stress range to ensure that all of the long-term fatigue requirements are satisfied.

SECTION 10. SUMMARY

This chapter has provided an overview of a number of stability issues that need to be included when evaluating the behavior of a bridge during the erection and deck construction stages. The chapter included a discussion of a number of potential stability modes that need to be evaluated including system buckling modes or girder rolling modes that can happen during construction. A number of other issues were also discussed such as problems associated with uplift at supports and other considerations that may be affected by the deck pouring sequence. An overview of the available design expressions for stability bracing were provided as well as an overview of analytic modeling decisions that the erection engineer should consider when deciding on how to model the bridge. A number of the factors discussed in the chapter provide a good segue in the next chapter which focuses on the methods of stability analysis.

CHAPTER 6

ANALYSIS FOR STABILITY

SECTION 1. INTRODUCTION

Chapter 6 extends the stability fundamentals presented in Chapter 4 to look at the behavior and analysis of individual members, partially erected systems of members, and the behavior during concrete deck placement. Although the primary focus of the examples in the chapter are on steel girders, the basic principles that are covered also apply to concrete systems. Advanced stability concepts such as non-linear analysis and 2nd order effects are presented. Effects of computer modeling parameters and software limitations on the accuracy of predicted behavior are considered, as well as techniques for verification of computer models. Representative outputs with buckled shapes are included to show the effects of properly designed bracing.

This chapter provides an overview of computational modeling techniques for evaluating the stability of bridge systems during the various construction stages. The primary emphasis is on steel girder systems, however many of the techniques are also applicable to concrete girder systems. Following this introductory section, important considerations for proper modeling of the girders and boundary conditions are discussed. The types of analyses are then covered including geometric linear, eigenvalue buckling, and geometric non-linear analyses. Because the stresses during construction are typically well below the yield strength of the material, the emphasis in this chapter is on elastic materials.

SECTION 2. COMPUTATIONAL MODELING

Computational abilities have dramatically improved in recent years. The speed and data storage capabilities of personal computers have made it possible for engineers to quickly carry out sophisticated analyses on complex systems. By nature, stability problems often require iterative analyses that, in the 1980's and 1990's, may have required several days on super computers to solve large-sized problems. With modern computers, these analyses may be completed in a relatively short period of time on a desktop or laptop computer. In addition to improvements in the computational capabilities, the sophistication of the software has also improved dramatically. Engineers have a variety of software choices available and they must have a clear understanding of the behavior that is to be captured in the analysis when selecting a suitable software package.

Many of the commercial software packages that are available specifically for bridge systems make use of two-dimensional grillage models in which the girders are modeled as line elements. Beyond the grillage models, engineers also have available a variety of

full 3D finite element programs that make it possible to develop detailed models of the components on the entire bridge system. The different software packages come with varying levels of complexity and analytical abilities. This section outlines the basic capabilities of some of the different classifications of software packages. Throughout the chapter, results from some specific software packages (MASTAN2, BASP, UT LIFT, and UT Bridge) are referred to or shown. It should be noted that the analysis discussed in this chapter can be carried out with a wide array of commercially available software. The software packages that are referred to in this chapter are available in the public domain and are free for download at the following sites: MASTAN2: <http://www.mastan2.com/>; BASP, UT LIFT, and UT Bridge: <http://fsel.engr.utexas.edu/software/index.cfm>. The goal of the discussions in this chapter is not to endorse any specific brand of software, but to instead provide an overview so that engineers can understand the necessary analytical capabilities to predict the type of behavior that is desired. The material presented in this chapter should help the engineer in selecting a suitable software package to satisfy the needs for their particular application as well as potentially understanding the abilities and proper use of the software packages that they may already be using.

The discussions on the modeling techniques often refer to “nodes” and “elements”, which are important characteristics of computer models that are used to define the structural system. As shown in Figure 6-1, nodes represent a point in space that is used to define the geometry of the basic model. The nodes in a structural model may possess translational and rotational degrees of freedom (DOF) that define the structural displacements. Depending on the types of elements that are used, each node in the model will usually have between two to six degrees of freedom (depends on the number of translational or rotational degrees of freedom in 2D or 3D space). The size of the model is related to the total number of DOFs.

The elements represent the structural member with the boundaries or geometries of the individual elements defined by the nodes. The two models shown in Figure 6-1 could be used to represent the same I-shaped cross-section. As shown in Figure 6-1(a), the connectivity of the line element is defined by the two nodes at the ends of the element. The cross-section in Figure 6-1(b) is composed of four-node shell elements and directly models the I-shape of the member.

The two models depicted in Figure 6-1 demonstrate the potential differences in the relative size of models. Only three nodes and two elements are required to define two sequential segments of the beam in Figure 6-1(a), whereas 27 nodes and 16 elements were used to define two sequential segments of the I-shaped beam in Figure 6-1(b). In a 3D model with six DOF per node, the two element representation of the I-shaped girder contributes 18 DOF to the model, while the two segments of the shell element model contribute 162 DOF to the model.

In general, due to concerns about element aspect ratios, shell element models will require many more element divisions along the length of the girders compared to line element models. Therefore, in addition to possessing many more nodes through the

cross-section, shell element models will generally have more cross-sectional divisions along the length resulting in much larger models (more DOFs) compared to line element models. Thus, the size of the shell element model obviously is much larger than the corresponding line element model; however, such a shell element model also potentially has the ability to capture much more complex behavior than the more simple line element model. The following two subsections provide a simple overview of some basic models that may be used to represent a given structural system.

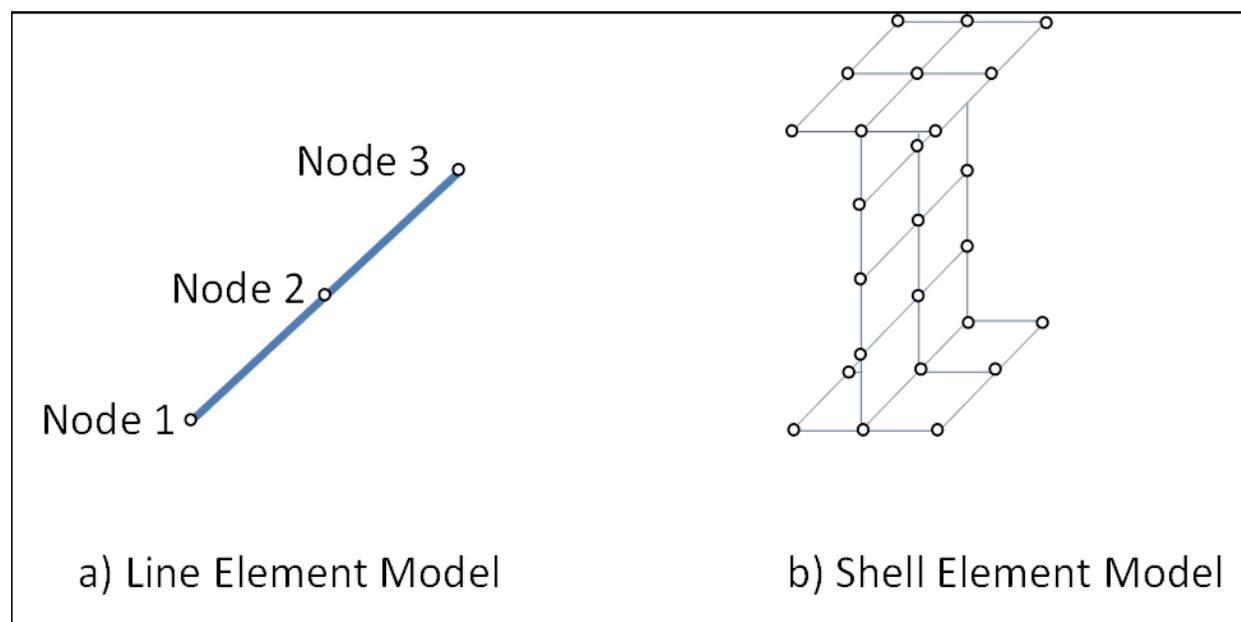


Figure 6-1 Line Element Versus Shell Element

6.2.1 Grillage Models

Most of the software packages that are commonly used in bridge design fit into the category of 2D grillage models. The software can be used to analyze either straight or horizontally curved girder systems. The term “grillage model” may also be referred to as a “grid model” in AASHTO and other documents. The girders are modeled as line elements and the concrete deck may be modeled by shell elements. Typical input for a grillage model consists of defining the nodal geometry, the cross-sectional properties of the girders, and the slab geometry. Based upon the desired model, the user can position nodes at specific locations so as to be able to define supports, concentrated forces, cross-frame locations, and transitions or changes in the shape of the cross-section. The nodes are positioned so that the analysis provides deformations and force/stress results. In many situations the software may allow the engineer to subdivide the elements after they are defined so as to add supplementary nodes in the structure to provide additional output. In some cases, the software may also internally subdivide the elements to provide a better representation of the structural system. Engineers should have an understanding of whether the software package internally subdivides elements to provide a more robust solution so that they can decide how many nodes

they should specify to achieve a sufficient level of accuracy. Methods of checking modeling accuracy are discussed later in the chapter.

Grillage models are attractive from a design perspective due to the ease of defining the model as well as the solution efficiency with respect to computational effort. The user generally has to simply specify the coordinates of the desired nodes as well as the girder properties along the length of the bridge. Although some grillage model software may allow the engineer to specify plate sizes for the flanges and webs and have section properties computed by the software, other programs require the engineer to input section properties such as areas, moments of inertia, and torsional properties of the girders.

The attractive feature of grillage models is the ease by which the model can be defined. The models generally have sufficient accuracy to provide good estimates of the design forces and deformations in the finished bridge. Although grillage models can be used to predict some behavior during construction stages, the programs are often not sufficiently accurate representations of the system to capture stability issues with most bridges during construction. One limitation of the line element representation of the girders is that such a model does not represent the local flexibility of the cross-section. In cases where braces, such as cross-frames or plate diaphragms, are not full depth, the effects of cross-sectional distortion play a very important role in the effectiveness of the bracing. In other situations the locations of supports or applied loads on the cross-section can play an important role in the overall behavior and line element models do not adequately capture these effects.

Another drawback to line element models is the ability to observe the displaced shape of the model. Because the girder cross-section is represented by a line, the displaced shape does not tend to show twist of the cross-section and as a result the user may not have a clear indication of the structural displacements. For example, consider the buckled shape of the beam in Figure 6-2(a). The beam has simple supports and is subjected to uniform moment loading. The buckling mode for the beam is lateral-torsional buckling; however the line element model does not show twist of the section. Some grillage or frame element programs have graphical interfaces that will extrapolate the line element model to show up graphically with depth and can therefore show twist of the cross-section. For programs that do not have this feature, the user can create “flags” at a few locations along the length to demonstrate twist of the cross-section. For example, the same model in Figure 6-2(a) was modified to include a “flag” at select locations along the length as shown in Figure 6-2(b). The flags were made by defining nodes and connecting them with elements. The element properties for these flags are not too important since the flag essentially acts as a deformation indicator and does not restrain the model. If material density is specified, it is important to not give the flags an area that will lead to a large self-weight contribution.

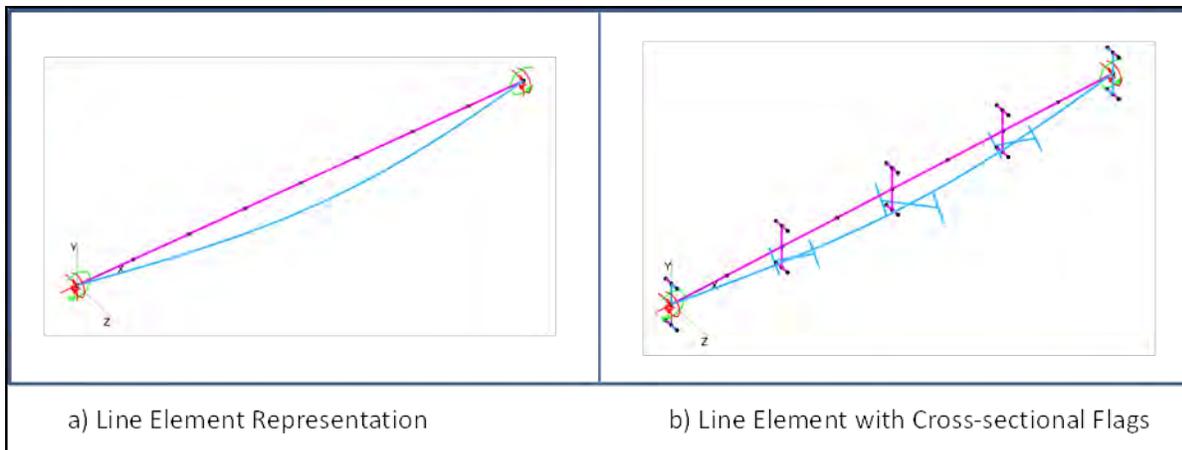


Figure 6-2 Indicating Torsional Deformations with Line Elements

The flags have the same rotation as the girder nodes to which they are connected and provide an indication of the rotational deformation along the length. In this case, “flanges” and the “web” were included in the flags; however, simply putting a vertical element at the locations provides the visual indication of the twist. Since the nodes on the flags are not restrained in any way, the solution to the problem does not change compared to the model in Figure 6-1(a). The effects of load position on the cross-section were discussed in Chapter 4. Gravity loads applied at the top flange are generally more critical than loads applied at the centroid or at the bottom flange. In a line element solution, a reasonable approximation of the effects of load position can be obtained by defining an element similar to the “flags” in Figure 6-2(b). For example, if concentrated forces were applied to the nodes at the simulated top flange location in Figure 6-2(b), the model will provide a reasonable approximation of the effects of load position in the buckling behavior. However, in this case, the properties of the flag elements may be important since they are loaded elements, and the user should consider the effects of the cross-sectional stiffness on the element properties. Considering the tributary area of the web/flange cross-section between the “flags”, the user can estimate an appropriate area and moment of inertia of the flag section.

6.2.2 Three Dimensional Models

There are a number of general purpose finite element programs that can be used to model structural systems. Most 3D finite element programs have an extensive library of element types that allow the user to model a wide variety of structural systems. The programs will usually include truss, beam, shell, or solid elements. The use of solid elements is not discussed in this document since these elements generally result in extremely large models and will not typically be required in bridge modeling. However, truss, beam, and shell elements are frequently used to model bridge systems and some of the basics of each element are covered.

As the name implies, truss elements are line elements that possess axial stiffness but not flexural stiffness. The elements are defined by two nodes at the ends that generally only possess translational DOF. Another type of line element is a beam element that

has both translational and rotational DOFs. Beam elements potentially have axial, bending, and torsional stiffness and can be used to model a variety of structural members; however as noted earlier, these elements are mainly able to model global member behavior and will not capture localized distortions of the cross-section. Shell elements as previously depicted in Figure 6-1 can be used to model a variety of structural members. General purpose programs may have a variety of different types of shell elements depending on the desired characteristics to be modeled. Some commonly-used shell elements may consist of four-noded or eight-noded elements that allow the user to model straight or curved plate elements.

An important feature of modeling with shell elements is to keep the aspect ratio (length/width of the individual shells) as close to unity as possible; however, good results can often be achieved with aspect ratios as high as three or more. The accuracy of the finite element model is often a function of the mesh density that is used. As is discussed later in the chapter, a mesh sensitivity analysis can be conducted to ensure that a sufficiently fine mesh density is utilized. The mesh density that is used may sometimes be dictated by the need to maintain a reasonable aspect ratio of the shell elements. Achieving an element aspect ratio close to unity will often not be possible for all of the elements in the cross-section. The engineer will often have to consider the number of elements necessary in the web depth versus the flange widths and balance the number of element divisions along the length to result in a satisfactory aspect ratio. As noted above, aspect ratios of two to three are not uncommon and still often yield good results.

To reduce the size of the model, a combination of shell and beam elements may be selected to model the members. A common practice for 3D models of girder systems is to use shell elements to model the webs and to use beam elements to model the flanges and web stiffeners. Utilizing beam elements for the flange can still capture both the St. Venant and warping stiffness of the beam, and can also reduce the size of the problem, since extra nodes for the flanges are not necessary.

3D modeling allows for the most accurate representation of the member cross-sections and allows the user to accurately model the locations of supports and load points on the cross-section, which can have a significant impact on the stability of the system. Because the web and stiffeners are represented in the model, the interaction of the bracing members and the girders can be more accurately captured compared to 2D models. An accurate representation of the interaction between the bracing and the girder cross-section is extremely important since cross-sectional distortion can render the bracing ineffective.

While three-dimensional models provide the most accurate rendering of the bridge elements, the drawback is the difficulty in creating the models. Although performance of computers and the capabilities of structural analysis software have dramatically improved over the past two decades, the bottleneck with regards to the ability to model and study structural behavior is in the efficient use of the software. The learning curve for users to become familiar with 3D modeling techniques is relatively steep. The length

of time necessary for an engineer to become familiar and efficient with 3D modeling software can take several months, and even then the time necessary to create the models for complex bridge systems is significant.

Efforts have been put forth to develop a user-friendly program that can be used to facilitate the creation of models to study the behavior of steel bridge systems during erection and construction (Stith et al., 2010). The resulting program is UT Bridge, which includes a preprocessor and a post processor that specifically target the modeling of steel I-girder bridge systems during erection and construction. Such an approach to model creation is limited by the necessary modeling assumptions; however, engineers can effectively use the software to quickly create 3D models to study complex bridge geometries and to consider a wide variety of erection or construction scenarios. Over time, modeling advances in commercial programs are likely to introduce products that the bridge industry can use to accurately and efficiently model a wider variety of systems.

Before important modeling scenarios and other specific problems can be discussed, it is important to cover some of the basic decisions that need to be made with regards to the types of analysis that are available to capture the structural behavior. Modern day programs have several options for different types of analyses that can be conducted, and engineers should have a proper understanding of the meaning and capabilities of these analyses. The following section provides a discussion of some of the different types of analyses that can be carried out.

SECTION 3. TYPES OF ANALYSES

In addition to the modeling options for the engineer, there are also a number of considerations with regard to the type of analysis to carry out when evaluating the safety of a structural system during construction. Although proper material models are often a major concern in evaluating the strength of a given structural system, the bridge elements will generally remain elastic throughout the construction stages. Therefore, linearly elastic materials will typically be utilized in most analyses. However, making decisions on the type of analysis can be a major decision and engineers should have a good understanding for the types of analyses they should be conducting to evaluate the structure. The analysis types that are discussed below include first order, eigenvalue buckling, and second order (nonlinear geometry) analyses. Regardless of the type of analysis that is utilized, in all cases the first step is to properly define the geometry of the model so that the structural representation and appropriate boundary (support) conditions are defined. How the load is applied to the structure may be somewhat dependent on the specific type of analysis that is to be carried out, as is discussed in the following sections.

6.3.1 First Order Analyses

The most common analysis that is generally carried out on bridge systems is a first-order analysis that assumes that the deformations in the structure will not significantly change the geometry. A first order analysis does not consider the stability of the system. With the exception of axial deformations in the structural members, equilibrium is taken on the undisplaced members. Such an analysis will typically provide good estimates of the reactions at supports or shore towers, as well as necessary holding crane lifting forces. In addition, from a girder design perspective, a first order analysis will usually provide good estimates of the moments and torsion induced in straight or horizontally curved girders. However, from a stability perspective, a first order analysis will not provide any indication of the stability-induced forces in the girders or the braces. Although a second-order analysis with non-linear geometry is generally necessary to capture deformations and forces related to girder stability, in many situations a second-order analysis may not be necessary as is discussed in more detail in Section 6-3.3.

The number of steps with which the load is applied to the system does not generally have any impact on the solution in a first order structural analysis. Therefore, the engineer can typically define the loads to be applied in a single load step.

6.3.2 Eigenvalue Buckling Analysis

Many programs have the ability to evaluate the global and potentially the local stability using an eigenvalue buckling analyses. The analysis option in the software may be referred to as a “critical load analysis” or a “buckling analysis”, both of which generally indicate that the program can solve for the buckling modes and the corresponding buckling loads. The program documentation likely includes some theory related to generalized eigenvalue problems, or if engineers desire additional information there are a variety of textbooks such as McGuire et al. (2000) that include a discussion on the matrix formulation. In order to properly utilize the results of an eigenvalue buckling analysis, the engineer needs to understand the significance of the eigenvalue and the corresponding eigenvector (mode shape).

The eigenvalue solution will give the user an indication of the elastic buckling capacity of the section and therefore does not consider the effects of material inelasticity. Since the stresses are typically well within the elastic buckling range during construction, the eigenvalue solutions provide a good indicator of the buckling capacity. However, the eigenvalue solutions are primarily applicable for problems in which prebuckling deformations are small. Cases where prebuckling deformations are not small are discussed toward the end of this subsection.

After the model is properly defined and the appropriate boundary conditions have been defined, the loads can be applied to the structure. However, the meaning of the resulting eigenvalue is directly dependent on the magnitude of the applied loads. The loads that are applied to the structure represent a “reference” load that the program

uses to compute the geometric stiffness matrix for the structure based upon a linear elastic structural analysis. Once the linear elastic stiffness matrix and the geometric stiffness matrix have been formed, the program solves the eigenvalue problem that yields the requested number of eigenvalues and corresponding eigenvectors. Many programs may allow the user to request multiple modes (eigenvectors); however, the mode that is usually of interest is the first mode since that will have the lowest eigenvalue. The eigenvalue, λ , represents a factor that is applied to the reference load to determine the critical buckling load using the following expression:

$$P_{cr} = \lambda P_{ref}$$

Equation 6-22

Where, P_{cr} is the corresponding buckling load, λ is the eigenvalue, and P_{ref} is the magnitude of the applied force. For example, consider the column with the cross-section shown in Figure 6-3. The column has pinned ends with a length of 30 ft

The column was modeled in MASTAN2 and the results are summarized in Figure 6-4. Three different cases were used for the applied “reference” load. In Case 1, a unit axial load of 1 kip was applied at the top of the column and resulting eigenvalue, λ , was 368.1. Referring back to Equation 6-1, since a reference load of 1 kip was used, the buckling load is directly equal to the eigenvalue. If instead a reference load of 10 kips is used, as is shown for Case 2, the eigenvalue must be multiplied by 10 to obtain the critical buckling load. In Case 3, the applied reference load was 368.1 kips and as expected, the resulting eigenvalue is 1.0.

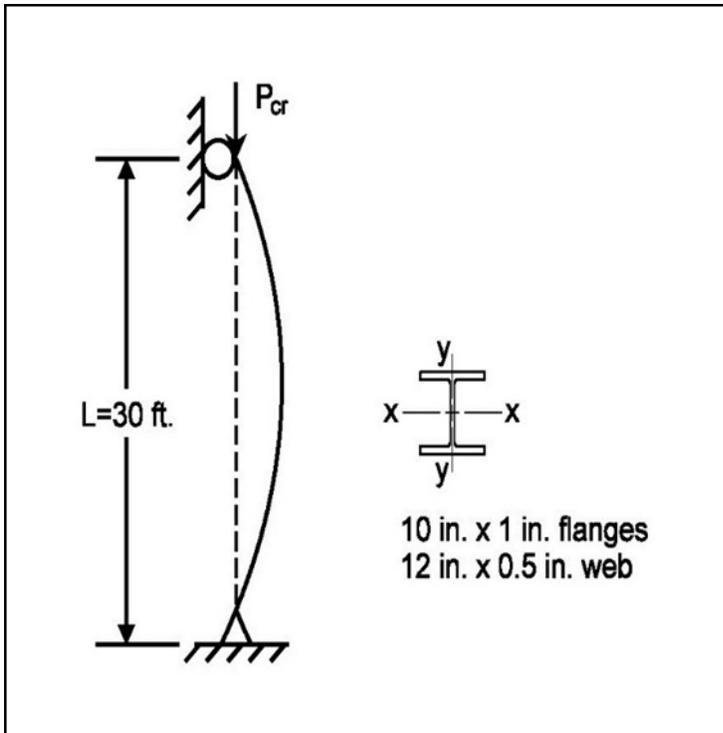


Figure 6-3 Euler Column with Length of 30 ft

The moment of inertia about the y-axis of the column cross-section shown in Figure 6-3 is 167.7 in^4 . Since the column has the boundary conditions of a classic Euler column, Equation 6-2 can be used to calculate the buckling capacity:

$$P_c = \frac{\pi^2 EI}{L^2} = \frac{\pi^2 (29000 \text{ ksi}) 167.7 \text{ in}^4}{(360 \text{ in})^2} = 368.1 \text{ kips}$$

Equation 6-23

As expected, the eigenvalue buckling analysis exactly predicts the critical load for this classic column buckling case.

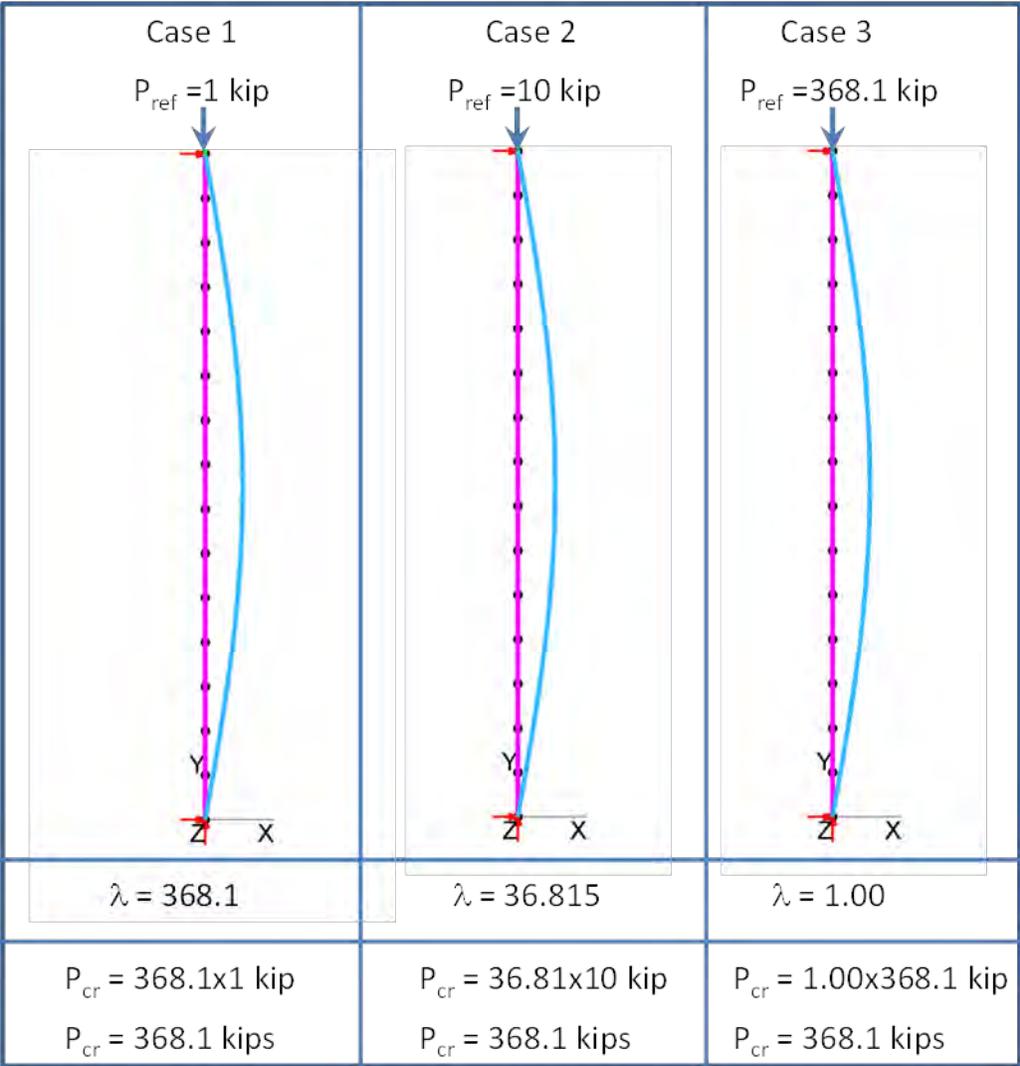


Figure 6-4 Determining Buckling Capacity based upon Eigenvalue Buckling Solution and Magnitude of Reference Load

Eigenvalue analyses can also be utilized in the cases of beam buckling problems. For example, consider the beam cross-section shown in Figure 6-5. The beam was modeled in MASTAN2 with a span of 50 ft (600 in.) and subjected to uniform moment loading. The corresponding buckled shape of the beam is shown in Figure 6-6. The beam elements in grillage or frame programs can often capture the torsional warping stiffness of the beams; however, the user may have to specify the types of torsional connections at the ends of the individual beam elements. For example, in MASTAN2 the connections at the ends of the beam elements can be “free”, “fixed”, or “continuous”. The “free” and “fixed” options either make the section warping free or warping fixed. In most situations, the user will want the section to be “continuous” which links the warping stiffness of the adjacent elements but does not imply a warping fixed condition.

To help show the buckled shape in Figure 6-6, the flagged model discussed earlier in the chapter was used. Because a concentrated moment of 1k-in was applied at the two ends, the eigenvalue is equal to the critical buckling load and gave a buckling moment of 7578 k-in.

The same beam was also modeled in the finite element program BASP (Akay et al. 1977), which is an acronym that stands for Buckling Analysis of Stiffened Plates. BASP is an eigenvalue buckling program that can capture both local and global buckling of stiffened plate elements. The program is a 2D finite element program that can capture plate buckling modes. Since the web of a girder is essentially a plate stiffened by the flanges, the program can be used to model global buckling of columns and beams. The program can be used to model doubly- and singly-symmetric sections; however, the sections must be symmetric about the web plate. Therefore although a doubly- or singly-symmetric I-section can be modeled, a channel section cannot be modeled since the flanges are not symmetric about the web. BASP uses four-node shell elements to model the webs and uses beam elements to model the flanges and stiffeners.

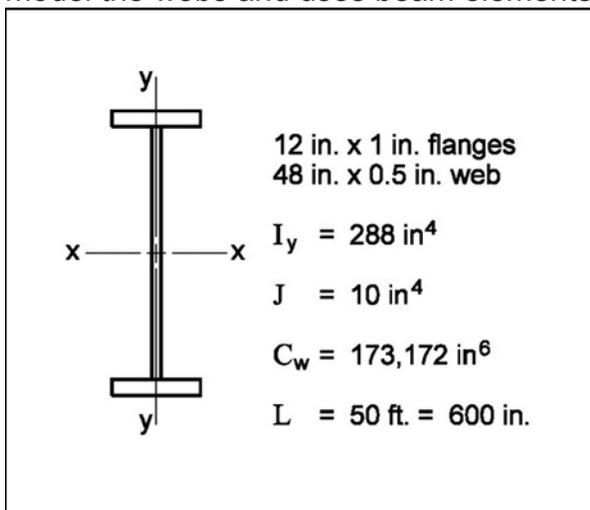


Figure 6-5 Girder Properties for Beam Buckling Analysis

The beam section depicted in Figure 6-5 was modeled in BASP to determine the buckling mode and the corresponding buckled shape is shown in Figure 6-7. The web was meshed with four elements through the depth, and 50 elements along the length, which gives an aspect ratio of unity for the web elements (i.e. $48 \text{ in}/4 = 12 \text{ in.}$ and $600 \text{ in}/50 = 12 \text{ in.}$). Because the flanges are modeled with beam elements, the aspect ratio is not an issue. The BASP model demonstrates some new aspects for discussion for modeling considerations when dealing with finite element models. BASP allows the user to apply concentrated forces at a node in either the x or y direction (the z direction is out of the plane of the plate). The program does not have a feature for applying concentrated moments. Therefore, a force couple must be used to model a concentrated moment. For the model shown in Figure 6-7, the couple consisted of two forces of magnitude “1/d” applied in opposite directions at the flanges, where d is the depth of the beam. Therefore, the magnitude of the applied concentrated moment was

(1/d) x d = 1 k-in. The predicted buckling capacity in BASP was 7193 k-in which results in a difference of approximately 5% from the capacity predicted by MASTAN. Part of the difference is likely due to local effects that occur around the locations of the concentrated forces; however, in general, the two programs have reasonable agreement despite very different model characteristics.

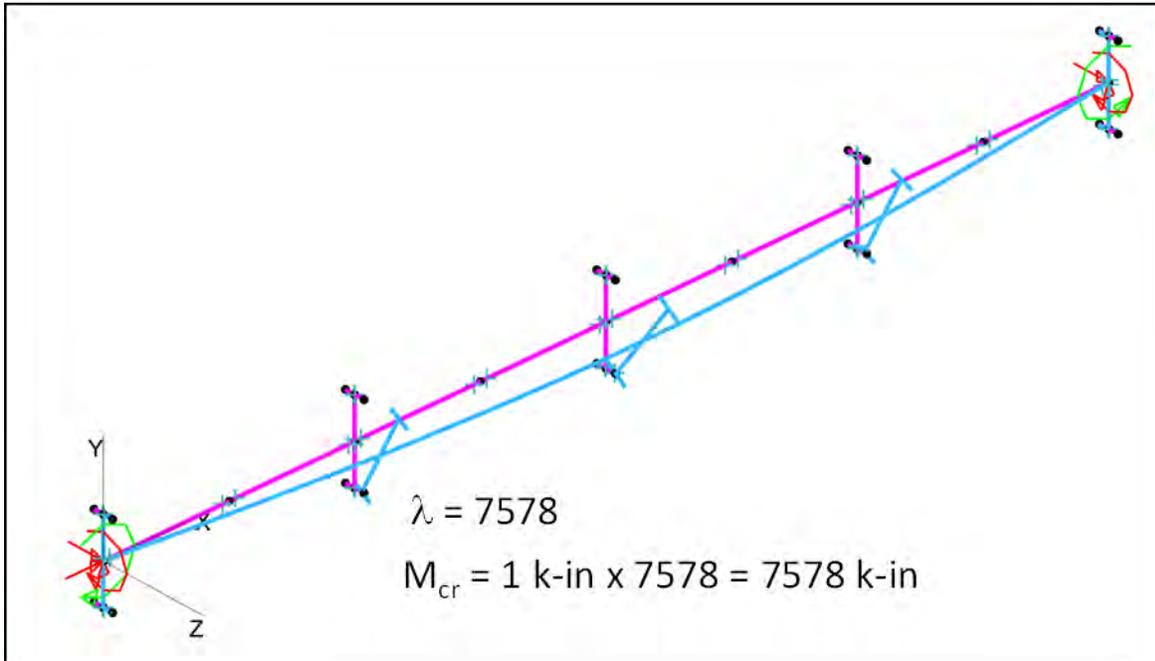


Figure 6-6 Beam Modeled in MASTAN2 Subjected to Uniform Moment Loading

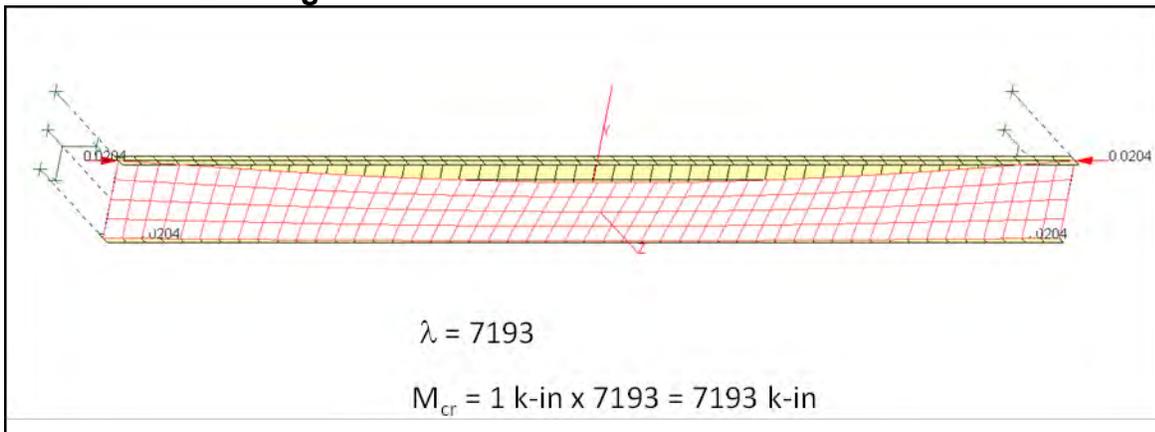


Figure 6-7 Beam Subjected to Uniform Moment Modeled in BASP

The elastic lateral-torsional buckling expression given in Equation 6-3 can be used to predict the buckling capacity of the beam modeled with MASTAN2 and BASP.

$$M_{cr} = \frac{\pi}{L_b} \sqrt{EI_y GJ + \frac{\pi^2 E^2 I_y C_w}{L_b^2}}$$

Equation 6-24

Inserting the section properties from Figure 6-5 into the above expression gives an estimate of the buckling moment of 7547 k-in, which has good agreement with the eigenvalue predictions from the two different computer models. The comparisons of the eigenvalue buckling solutions and the elastic buckling expressions for the column and beam problems demonstrate that the eigenvalue buckling solutions are analogous to the solutions that are predicted from the elastic buckling equations. While the two problems considered were the idealized cases for which theoretical solutions are readily available, eigenvalue buckling solutions can be used to model more general problems for which the theoretical solutions must rely on approximate modifiers such as K-factors for column buckling and C_b factors for beams with moment gradient. Provided the problems are properly modeled, the eigenvalue buckling programs can provide more accurate solutions for the specific problems.

The load cases that were used in the respective column and beam buckling problem consisted of a pure axial load and the case of uniform moment. The loading that is used in actual structures is more complicated and therefore the selection of a “reference load” to use in the eigenvalue problem is not always clear. For the case of gravity loads from self-weight, most engineers simply put the self-weight on the structure and conduct the eigenvalue buckling analysis. In this case, the eigenvalue represents a multiplier on the self-weight that will result in buckling. Some engineers think of this as a factor of safety; however this is not necessarily a correct interpretation since there are nonlinear effects associated with the stability problem. The degree of non-linearity is a function of how close the applied load is to the buckling load.

Questions are often raised with regard to how large the eigenvalue should be relative to the full service loading applied to the structure to have a safe condition. An eigenvalue of unity or less for cases where the full service load is applied to the structure clearly demonstrate cases in which the structure will likely have stability issues. Cases where the eigenvalue is substantially larger than unity, such as three or four, present cases where stability is not likely an issue. However, an exact limit that should be targeted with regards to the magnitude of the eigenvalue is highly dependent on the buckling mode. The slenderness of the element that is being considered should be taken into consideration since the flexibility of the element is more likely to lead to larger second order effects. A target “load factor” during construction might be of the order of 1.4~1.5 and therefore the eigenvalue based upon service loads should at least be in this range. If an extra 25%-30% safety margin is placed to avoid flexibility issues with slender elements a target eigenvalue of 1.75 may be appropriate for some problems. If an engineer is not comfortable with the buckling capacity predicted from an eigenvalue analysis, bracing should be added, or a more detailed analysis considering non-linear geometry, such as discussed in the next subsection, should be considered.

Engineers should pay careful attention to the buckled shape to see if it coincides with their expectations. In some cases the engineer may be expecting a global buckling mode and instead a local instability may control the buckled shape. Although there is typically a significant difference in the buckling capacity for different modes for global

buckling, local instabilities often have numerous potential mode shapes clustered around a small range of loads.

Engineers should have a good understanding of when an eigenvalue solution may not provide good insight into the buckling behavior. A primary assumption in an eigenvalue analysis is that pre-buckling deformations do not significantly affect the geometry of the structure. The pre-buckling deformations that adversely affect the accuracy of the eigenvalue solution are generally in the same direction that is being predicted in the analysis. For example, in the two problems considered in this section (the column and the beam buckling problems), the pre-buckling deformations were not in the same direction as the buckling mode that was being predicted. In the case of the column, the applied load causes axial shortening but does not cause any flexural deformations in the direction of buckling. Therefore, the magnitude of the reference load has no impact on the predicted buckling capacity. The same can be said for the beam buckling problem. Although a larger reference load will lead to larger bending about the strong axis of the beam, the buckling mode primarily involves twisting about the longitudinal axis and bending about the weak axis. As a result, the magnitude of the reference load has no significant impact on the buckling load. If the reference load is doubled, the eigenvalue will be halved.

The type of problem that is not well-suited for an eigenvalue buckling analysis is depicted in Figure 6-8. The shallow frame may be susceptible to in-plane buckling and the prebuckling deformations, Δ , are in the same direction as the potential buckling direction.

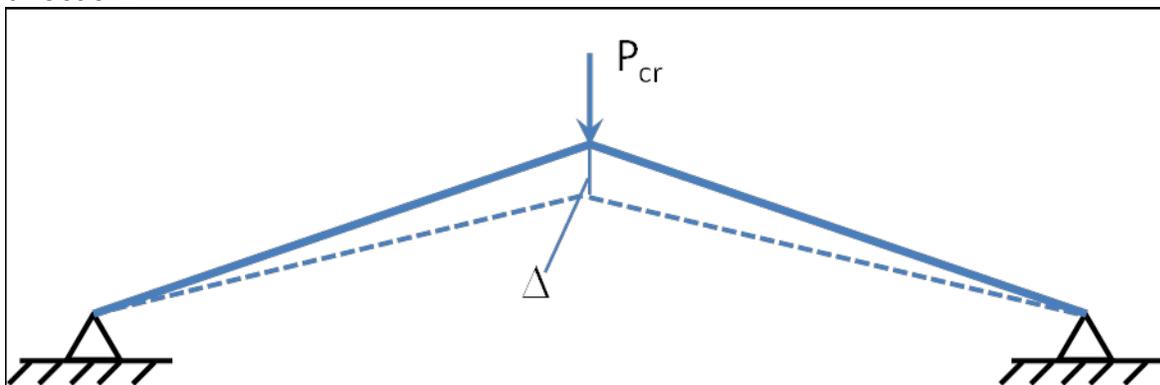


Figure 6-8 Problem not Well Suited for Eigenvalue Buckling Analysis

The linear eigenvalue problem is based on a linear elastic analysis; however, the analysis still does reflect the effects of axial shortening. As a result, the buckling capacity in this case will generally be sensitive to the magnitude of the reference load. For problems such as this, a second-order analysis should be conducted to evaluate the stability of the system.

A common problem in the bridge industry in which the eigenvalue will often not provide a definitive solution is in horizontally curved girders. Studies (Stith, et.al. 2009 and Stith

et. al., 2010) have shown that the eigenvalue solution is not generally sensitive to the degree of curvature in the bridge. For example, if the effective girder span is maintained but the radius of curvature of the girder is increased, the eigenvalue does not significantly change. However, it is relatively clear that for a given load the torsional deformations will increase as the radius of curvature of the girder is reduced. The larger girder twist can lead to second order effects that are not predicted by the eigenvalue analysis.

6.3.3 Second-Order Analysis

Many software packages have the ability to carry out a second-order analysis that considers the impact of deformations on the structural behavior. A second-order analysis is also sometimes referred to as a large displacement analysis or an analysis with nonlinear geometry. Second-order effects from an analysis perspective can generally be referred to as amplifications in member forces and displacements as a result of the changes in geometry of the structure. For example, the perfectly straight column subjected to pure axial load in Figure 6-9(a) only experiences axial stress until the buckling load is reached at which point the member must bend. If a more realistic column is considered with an imperfection, Δ_0 , as shown in Figure 6-9(b), the column will begin to bend immediately when load is applied due to the $P\Delta$ moments that develop in the column. Whereas first-order deformations can be solved for directly since the analysis is conducted on the original geometry of the structure, second-order deformations result in non-linear behavior that requires an iterative solution. Second-order effects develop in structures as a result of several factors, including combined bending and axial force, the impact of imperfections, or due to geometrical effects in the structural elements. Although the most accurate approach to predicting the behavior is to carry out a second-order analysis that includes the impact of deformations on the structural response, in many cases the impact of neglecting second-order effects is insignificant and, therefore, such an analysis may not be warranted. This section provides a discussion of some of the factors that should be taken into consideration when conducting a second-order analysis. Some discussion is also provided on when a second-order analysis may be required, as well as cases that may not necessitate the more detailed analysis.

The magnitude and shape of the initial imperfection has a significant impact on the behavior of the main members as well as the requirements for bracing. The shapes of the imperfections that should be modeled are dependent on the type of buckling that is critical. The critical shape for column members that are susceptible to flexural buckling generally consist of pure lateral sweeps, while members susceptible to either torsional or flexural-torsional modes (such as beams) will potentially have lateral sweep and twist. The impact of imperfections and the critical shapes and magnitudes are covered in more detail later in the chapter.

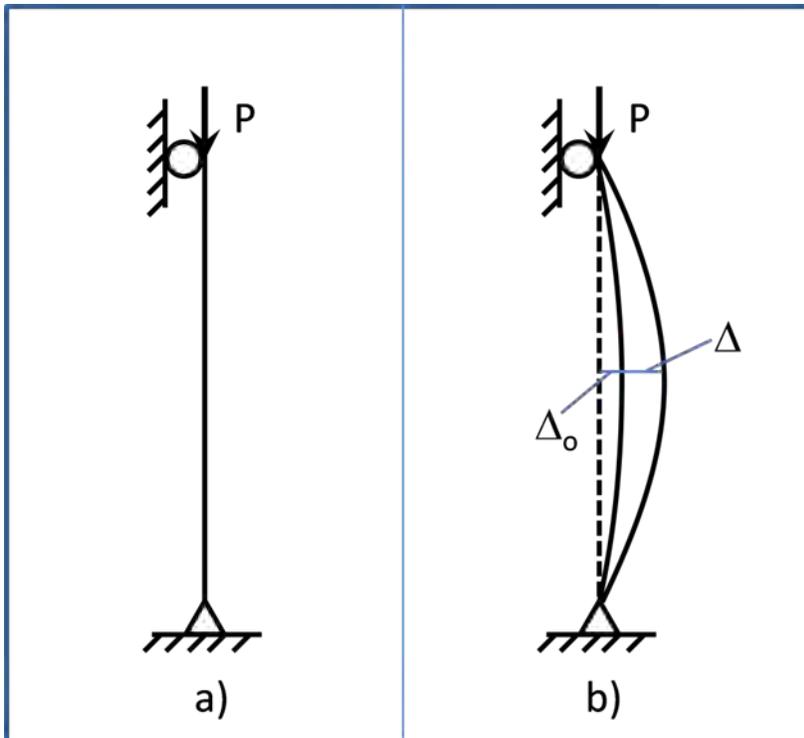


Figure 6-9 Second Order Effects as a Result of Imperfections

The degree of nonlinearity in a second-order analysis is sensitive to the magnitudes of the applied loads relative to the buckling load as well as the imperfections in the structure. For example, Figure 6-10 shows the load versus deformation curve for the column section shown previously in Figure 6-3 with a lateral sweep imperfection. The maximum sweep imperfection at midheight had a magnitude of $L/500$. The vertical axis was normalized by the eigenvalue buckling load ($P_{cr} = 368.1k$) that was covered earlier in the last subsection. The buckling load that was determined from the eigenvalue analysis is essentially an upper bound on the load carrying capacity since the column experiences very large lateral displacements as the buckling load is approached. At load levels of 50-60% of P_{cr} , the degree of non-linearity is not that significant; however, as the buckling load is approached, the load-deformation curve is relatively flat and the column experiences very large deformations for small increases in the applied load. This type of behavior is important for engineers and construction personnel to understand. The concept of buckling is often perceived to be a “sudden” event. In reality, the instabilities that occur due to global buckling modes are often a gradual softening in the structure. As the structure nears the buckling capacity, the stiffness decays and the structure may experience large deformations due to the increased flexibility. Therefore, engineers should be aware of potential problems when field personnel complain of fit-up problems or flexibility in the structure. Although in many situations the problem may just be a fabrication or erection issue, potential stability problems should not be ruled out without proper consideration and/or analysis.

The manner in which the load is applied in a second-order analysis can significantly impact the accuracy of the solution. Whereas in a linear elastic analysis, the load can be

applied in a single load step, the load in a second-order analysis should typically be applied in several steps, the size of which depend on the relative magnitude of the total load compared to the buckling load. For this reason, it is usually prudent to first conduct an eigenvalue buckling analysis before carrying out a second-order analysis. The knowledge of the load level compared to the critical buckling load will likely be an important consideration in the load steps that are selected for the analysis. For example, in a large displacement analysis where the full load may be close to the critical buckling load, the user can generally begin the first few load steps with larger percentages of the buckling load, but should gradually taper the steps as the buckling load is approached. This practice will often result in more accurate predictions of the deformations, as well as quicker convergence in the solution. Some of the software packages may offer different solution approaches with regard to how the load is applied. For example, the software may offer an automatic load stepping option where the program establishes the sizes of the load steps and if convergence is not reached within a certain number of iterations, the load increment is reduced until either: a) convergence is achieved, b) the time or load steps allotted to the analysis is exceeded, or c) the solution diverges.

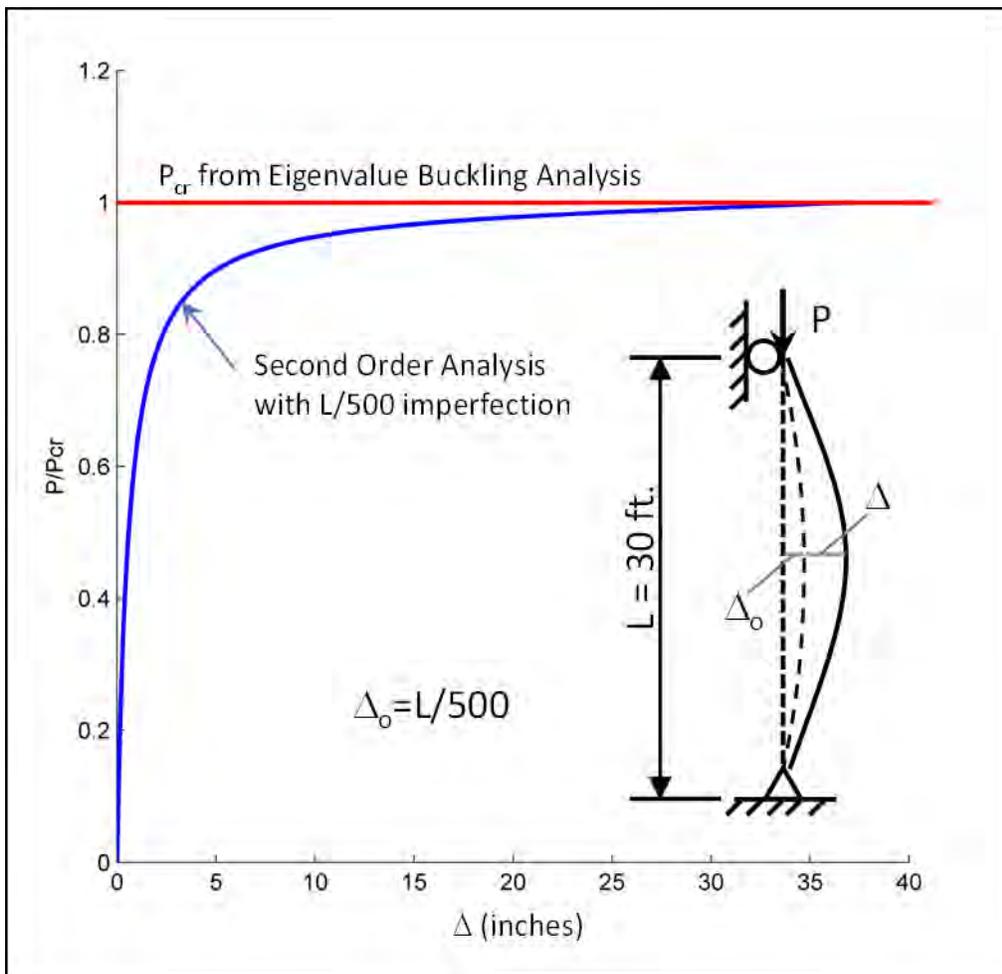


Figure 6-10 Second Order Analysis on Column with Initial Imperfection

Other important aspects in the accuracy of the second order analysis are the convergence tolerances, which some programs allow the user to specify. The large displacement analysis is based upon an iterative solution process that will continue until either 1) convergence is achieved, 2) the load step may be reduced (automatic load stepping) until convergence is achieved, 3) the solution diverges, or 4) the time or number of iterations allotted to the analysis is exceeded. Some programs will not allow the user to specify convergence tolerances; however, others allow user-specified limits such as force and displacement convergence limits. Care should be taken in selecting the convergence limits since too large of a tolerance can result in the accumulation of errors in the solution that can lead to poor estimates of the structural behavior. On the contrary, specifying too tight of a tolerance convergence can substantially increase the length of time for the analysis with very little impact on the accuracy of the solution. Methods of determining appropriate tolerance limits are discussed in the following subsection that focuses on model verification.

This chapter has focused on three analysis methods that may be used in the analysis of bridge systems:

4. first-order analysis,
5. eigenvalue buckling analysis, and
6. second-order analysis.

Of the three different analysis methods, the most accurate is the second-order analysis that reflects the impact of deformations on the overall behavior. However, the improved accuracy also requires potentially more complexity that may not be justified. For example, it can be stated that the most accurate analysis technique is to use a second-order analysis with non-linear material capabilities. Although the proper material model can be developed with sufficient user effort, these efforts are a waste of time if the structure is then found to remain in the elastic range. Similarly, the decision to carry out a second-order analysis should be based upon expectations or concerns of the impact of deformations on the structural performance.

In many instances, a second-order analysis may not be required. The decisions on whether to conduct a second-order analysis should be based upon considerations of the potential structural behavior and experience can help dictate these decisions. Including the impact of imperfections on straight girder systems is mandatory in order for a large displacement analysis to provide meaningful results. Other factors that will affect the large displacement analysis are effects of support skew as well as the impact of horizontal curvature on the behavior. Parametric studies conducted as part of a TxDOT sponsored research study on curved girders (Stith et. al., 2009) found that the impact of second order effects in horizontally curved girders were significantly affected by the lateral and torsional stiffness of the girder system. The study considered the geometry of the proportioning of the girders and recommended a ratio of the flange width/girder depth greater than approximately 0.25 to minimize the impact of second-order effects. The minimum value of the ratio of the flange width/girder depth permitted in AASHTO

(2012) is $1/6$, which results in relatively slender girder sections that may experience significant second order effects. The TxDOT study also considered the impact of holding cranes versus shore towers. Because holding cranes support the girders from the top flange, the cranes result in a restoring force that tends to reduce the second order effects. Therefore a first-order analysis will often actually be conservative compared to a second-order analysis, when considering the impact of the holding cranes. Second-order effects for shore towers will tend to destabilize the girders unless twist of the girder is also restrained at the shore tower.

When considering bracing requirements for stability, a large displacement analysis is required on the imperfect system in order determine the necessary stability forces.

This chapter has focused on some of the different analysis options that are available to the engineer and also highlighted some of the decisions that need to be made in developing the model. The modeling decisions that are made can be critical to ensure sufficient accuracy in the analysis. Before a new software package is used to model a complex structural system, the user should always begin with a relatively simple problem for which the behavior is well understood. For example, the problems that have been considered thus far in this chapter were an Euler column and a simply supported beam subjected to uniform moment. These basic problems were selected for simplicity in discussion and also because the solutions for the problems were readily available for comparative purposes in discussing the accuracy of the computer models. Simple problems and experimentation should always be the starting point when beginning with a new software package. For example, the simply supported beams shown in Figure 6-11 represent good problems that an engineer can use to make sure that they understand how to input basic support conditions, load types, and cross-sectional properties. The user has a good indication of the expected reactions and deformations for the problem and therefore can ensure that they are properly specifying the necessary input for the problem. Additional problems to those depicted in Figure 6-11 are available from a variety of sources such as basic structural analysis books or other sources. The users may want to also consider fixed ended beams or other idealized support or load conditions in the analysis to ensure that they are properly inputting the appropriate boundary conditions or applied loading.

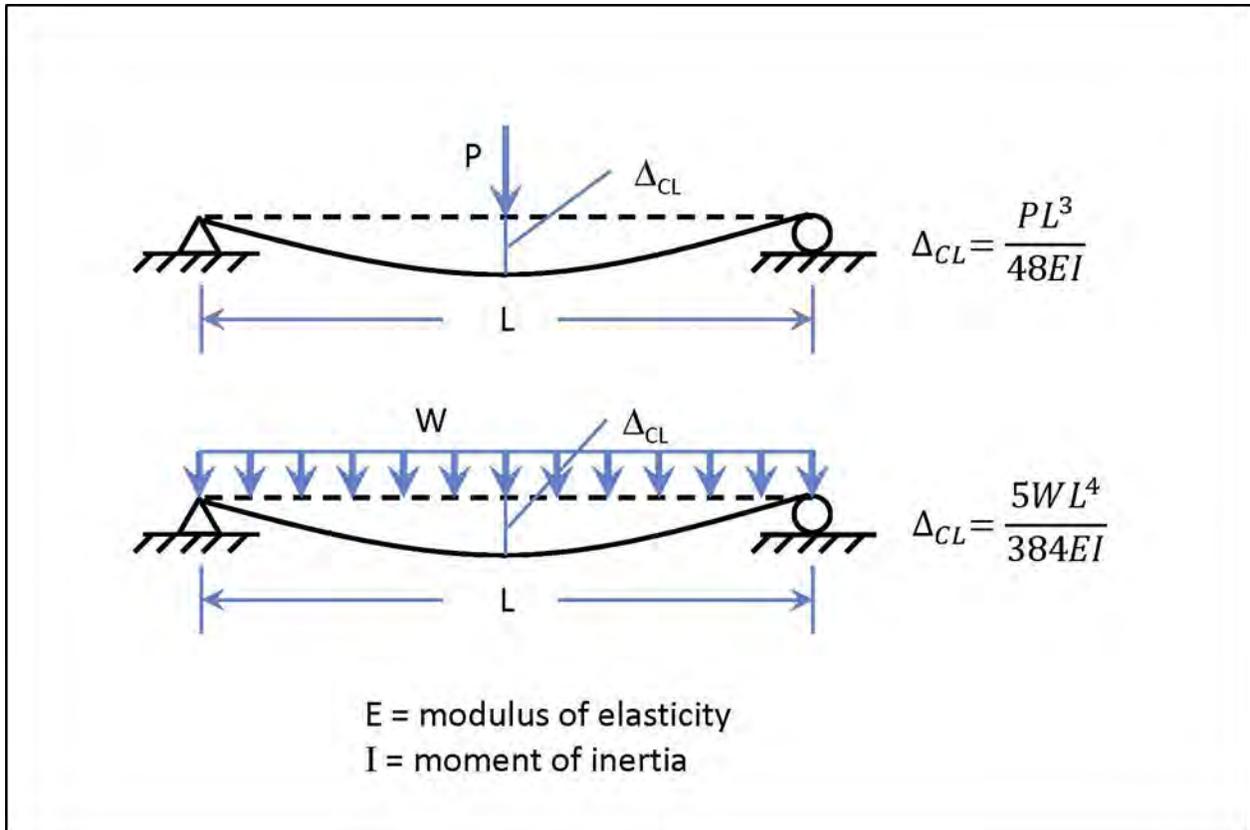


Figure 6-11 Simple Problems for Use in Becoming Familiar with Software Capabilities and Modeling Decisions

While simple problems should always be used as a starting point in becoming familiar with new analysis packages, it is also important that to understand techniques for ensuring that the appropriate modeling decisions are made instead of blindly accepting the results of an analysis. One of the most critical decisions that affects both accuracy and computational efficiency of a computer model is the appropriate mesh density for the problem. The accuracy of a finite element model generally improves with increased mesh density. Increasing mesh density generally involves providing more elements to model the structure. Provided the aspect ratio of the elements is maintained, increasing the number of elements in the model will lead to improved accuracy. However, with more elements the number of degrees of freedom in the structure also increases which therefore reduces the computational efficiency of the problem. The decrease in the computational efficiency results in longer analysis times as well as a higher demand on computer resources such as memory and disk space. The user must therefore weigh the benefits of the increase in accuracy compared to the reduction in computational efficiency.

The last subsection focused on second order analysis methods in which nonlinear geometry is considered. As discussed, some of the software packages may allow the user to establish the tolerance limits that will be used for evaluating convergence. Utilizing too large of a tolerance limit may lead to errors in the solution, while too small of a tolerance will lead to longer computational times with very little impact on the

accuracy. The user can experiment with the tolerance limits to determine appropriate limits that provide good estimates of the structural behavior. This is accomplished by starting out with relatively large limits and reducing the limits while comparing the relative change in the solution. The point when the changes in convergence tolerances have little impact on relative change in the solution usually signifies acceptable tolerances that will provide sufficient accuracy and computational efficiency.

SECTION 4. BEHAVIOR OF STRAIGHT AND HORIZONTALLY CURVED GIRDERS DURING LIFTING

The behavior of girders during lifting can be difficult to assess. Although a variety of lifting configurations may be used, a survey of erectors, contractors, and engineers conducted on TxDOT study 0-5574 (Stith et. al., 2009) found that most girder segments are lifted by a single crane with a spreader beam as depicted in Figure 6-12. The stability of the girder is a function of many factors. The connection to the crane typically consists of a simple lifting clamp connected to the top flange. The primary source of stability is the girder self-weight that acts at the center of gravity on the cross-section hanging below the crane lifting point. The girder receives a stabilizing effect because the self-weight acts at a location below the support point from the crane as depicted in Figure 6-13. Twisting of the girder results in a stabilizing component. However, lateral-torsional buckling is a major concern due to the long unsupported lengths of the girder. Depending on the locations of the crane pick points, stability can be controlled by either the overhang region of length “a” or by the region between the pick points of length “ L_{Lift} ” as indicated in Figure 6-12.

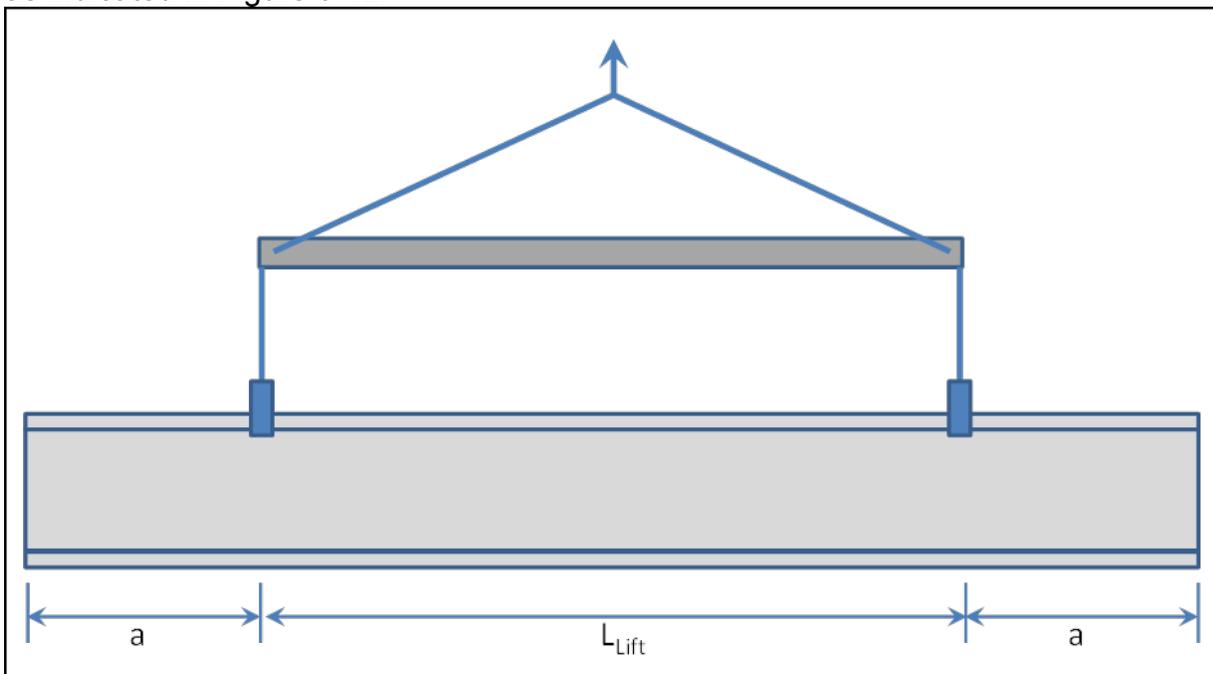


Figure 6-12 Schematic of Girder Segment during Lifting

The stability of girders during lifting was studied by Schuh (2008) and Farris (2008) who developed moment modification factors, C_b , that can be used to estimate the buckling behavior of girders during lifting. The moment modification factors are a function of the length of the overhang, a , versus the middle region of the girder, L_{Lift} .

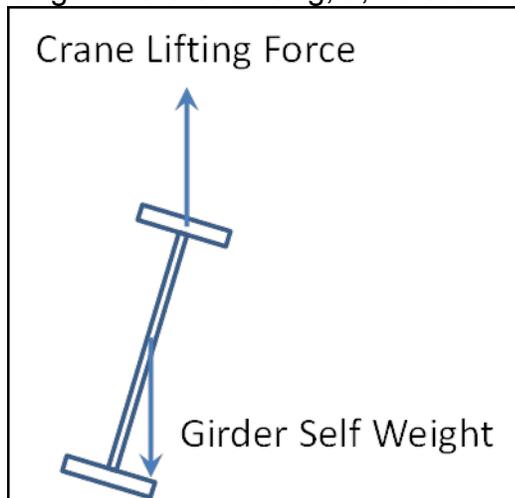


Figure 6-13 Stabilizing Effect of Girder Self-Weight Acting below Crane Pick Point

Another major problem that erectors face is the lifting of horizontally curved girder segments that can experience a rigid body rotation during lifting. Depending on the location of the lifting points, the geometric centroid of the horizontally curved segment may have an eccentricity to the line of support from the cranes as depicted in Figure 6-13 (Stith et. al., 2010). As a result, the girder will rotate until the geometric centroid of the segment lies beneath the line of support. As demonstrated in the figure, the girder rotation may be towards the outside or inside of the curve depending on the location of the lifting points. The rotation of the girder can be a major concern for erectors since excessive rotation of the lifted girder can make air splices such as the connection shown in Figure 6-14 difficult to complete.



Figure 6-14 Bolted Air Space [Stith et al. 2010]

A computational tool, UT lift, was developed as part of TxDOT research study 0-5574 to assist erection engineers with evaluating the lifting behavior of girder segments. The tool consists of an Excel-based spreadsheet that is applicable to straight and horizontally curved steel girder segments lifted at two locations along the length of the girder. The spreadsheet is available for free download at <http://fsei.engr.utexas.edu/software/index.cfm>. The spreadsheet consists of five sheets, one of which requires user input. The other pages provide calculations on the girder behavior during lifting. An input page for a sample girder segment is shown in Figure 6-15. The cells that are shaded are areas for requested input. In addition to some basic title and project information, the input allows the user to specify the number of section changes along the length of the segment. The problem shown in this example consists of a girder with a dapped end that has six section transitions along the length. The plate sizes for the flanges and webs, as well as the length of each section along the segment, are input. The weights of the various sections are calculated and the user can also specify a “girder scale factor” to account for the weight of stiffeners or other fabricated parts of the section. Also on the first sheet is information about the cross-frames that may be lifted with the girder segment as depicted in Figure 6-16. The user can specify the number of cross-frames that will be include on the segment and also provide the weight of the cross-frames so that the impact on the girder torque (and resulting twist) can be considered. The user can experiment with the behavior considering whether cross-frames are included only on the inside of the curve (I), only on the outside (O), or both inside and outside (I/O). The weight of the cross-frames are assumed to be applied a distance of $s/2$ from the girder centerline, where s is the girder spacing. If desired, the erector can also adjust the weight of one of the cross-frames to determine the necessary counterweight that could be applied to reduce the twist of the segment so that the splice is easier to complete.

Behavior of Curved Girder During Lifting

L_1 : Length of Section 1
 L_2 : Length of Section 2
 W_1 : Weight per Unit Length of Section 1
 W_2 : Weight per Unit Length of Section 2
 $\theta_0 = 0$
 θ_1 : Internal Angle from the Beginning of the Girder to the End of Section 1
 θ_2 : Internal Angle from the Beginning of the Girder to the End of Section 2
 s : Cross Frame Width
 L_{x3} : Length along the Girder to X-Frame 3
 θ_{x3} : Internal Angle from the Beginning of the Girder to X-Frame 3
 R : Radius of Curvature of the Girder

Girder Input:

Project : Horizontally Curved Gir. w/ Dapped End	Clear Input	User Input
Girder # : Girder 1 - Piece XXX		Notable Results

Number of Cross Sections: NUMSECTIONS = 6	Material Constants:
Radius of Curvature (ft): R	Girder Scale Factor: E = 29000 ksi
R = 1800 ft	S.F.girder = 1.05 G = 11154 ksi
	rho = 490 lbs/ft ³

	Section 1	Section 2	Section 3	Section 4	Section 5	Section 6	
TFLW	24	24	24	24	24	24	TFLT
TFLT	1.25	1.25	1.25	1.25	1.5	1.25	
DEPTH	51	62.5	74	74	74	74	DEPTH
WEBT	0.625	0.625	0.625	0.625	0.625	0.625	
BFLW	24	24	24	24	24	24	BFLT
BFLT	1.25	1.25	1.25	2	2.5	2	

	Section 1	Section 2	Section 3	Section 4	Section 5	Section 6
$L_1 =$	3.536	3.833	23.526	30.01	65	14.75
$W_1 =$	328.3	353.9	379.6	443.9	508.2	443.9
$\theta_1 =$	0.11	0.23	0.98	1.94	4.01	4.48
$J_1 =$	35.40	36.34	37.27	85.65	158.02	85.65
$C_{wl} =$	1965645	2926125	4077045	5068038	6238080	5068038

Length of Cross Sections (ft):	L_1
Weight per Unit Length (lb/ft):	W_1
Internal Angle from Beginning (deg.):	θ_1
Top Flange Width & Thickness (in):	TFLW, TFLT
Web Thickness & Depth (in):	WEBT DEPTH
Bottom Flange Width & Thickness (in):	BFLW, BFLT
Polar Moment of Inertia (in ⁴):	J_1
Warping Constant (in ⁶):	C_{wl}

Figure 6-15 Sample Input Page for Girder Properties in UT Lift

Cross Frame Input:							
Number of Cross Frame Locations: NUMXFRAMES = 7				Uniform Cross Frame Weights: Weight = 480 lbs			
Cross Frame Width: s = 10.00 ft				Constant X-Frame Weight			
Uniformly Spaced Cross Frames: Spacing = 20 ft				All Cross Frames on Inside of Curve, Outside of Curve, or Both			
Location of the 1 st Cross Frame: 1st X-Frame Loc. = 0 ft				Constant X-Frame Spacing			
				All I All O All I/O			
	X-Frame 1	X-Frame 2	X-Frame 3	X-Frame 4	X-Frame 5	X-Frame 6	X-Frame 7
$Lx_j =$	0	20	40	60	80	100	120
$Wx_j =$	480	480	480	480	480	480	480
I/O	I	I	I	I	I	I	I
$\theta x_j =$	0.000	0.637	1.273	1.910	2.546	3.183	3.820
$Lx_j =$							
$Wx_j =$							
I/O							
$\theta x_j =$							
Cross Frame Location along centerline (CL) of the Girder (ft): Lx_j							
Weight of One Cross Frame (lbs):				Wx_j			
Inside of Curve, Outside of Curve or Both (I, O, or I/O):				I/O			
Internal Angle from Beginning (deg.):				θx_j			

Figure 6-16 Sample Cross-frame Input for UT Lift

The next sheet of the spreadsheet provides information on the location of the geometric centroid of the section, as well as the optimum lift location for the girder segment as shown in Figure 6-17. The optimum lift location is the position that results in zero rotation of the segment during lift. The program also estimates the required spreader beam length that would be required for the problem. For the sample case in the figure, a spreader beam length of 98.684 ft with the first end positioned 23.78 ft from the first end will result in zero rigid body rotation in the segment.

The information on the optimum lift locations is provided primarily for information purposes, since erectors will generally not have the ability to exactly match the optimum lifting equipment. Therefore, as shown in Figure 6-18, the erector can input the length of the spreader beam that is in stock and adjust the lifting point locations on the beam to achieve a rigid body rotation that is acceptable to the desired performance. The user can also go back to the cross-frame information and adjust the number and location of the cross-frames to help modify the twist of the section. The spreadsheet provides a warning that the lifting forces may not be equal as indicated in the sheet; however, in this case the forces are within a few hundred pounds of each other which will not be an issue. If the forces are too much out of balance, the segment and spreader beam will rotate until equilibrium is achieved, which may result in difficulty making the connection. In this case, the location of the lift points can be adjusted until the forces are approximately equal. There is a button on the lower part of the page that the user can push to calculate the rotations and stresses in the girder. Because the program includes macros, the user will have to enable the macros in the spreadsheet. A one-dimensional finite element was incorporated into the macros of the spreadsheet to compute the twist of the girder segment from the torque that results from the geometry. Therefore, the program estimates both the rigid body rotation as well as the girder rotation due to torsion.

Figure 6-19 shows the sample output of the girder stresses and segment rotations with the input spreader beam and lifting locations. In this case the two ends of the segment are estimated to rotate 3.127° at one end and 1.900° at the other end. If these rotations are larger than the erector feels is necessary to make the connections at the ends of the girder, the lifting properties can be adjusted or the contractor can experiment with cross-frames included in the lift on the inside or outside of the girder to adjust the torque. This page also allows the user to estimate the buckling load of the segment with input of a dead load factor (g) and a resistance factor. The spreadsheet shows the maximum factored moment based upon the self-weight of the steel section (and load factor) and also estimates the buckling capacity using the AASHTO elastic buckling expression and the C_b factor from the work of Farris (2008). The buckling expression will be reasonably accurate for straight (or mildly curved) girders; however, for girders with significant curvature, the elastic buckling load is not necessarily a good indicator of safety due to the nonlinear response of the girder. The user can get an indication of potential problems with the girders by considering the magnitudes of the deformations that may occur during lifting. To help understand the girder behavior, sample graphs as depicted in Figure 6-20 are provided on the last page of the spreadsheet. If the deformations seem excessive, more detailed analysis of the girders may be necessary.

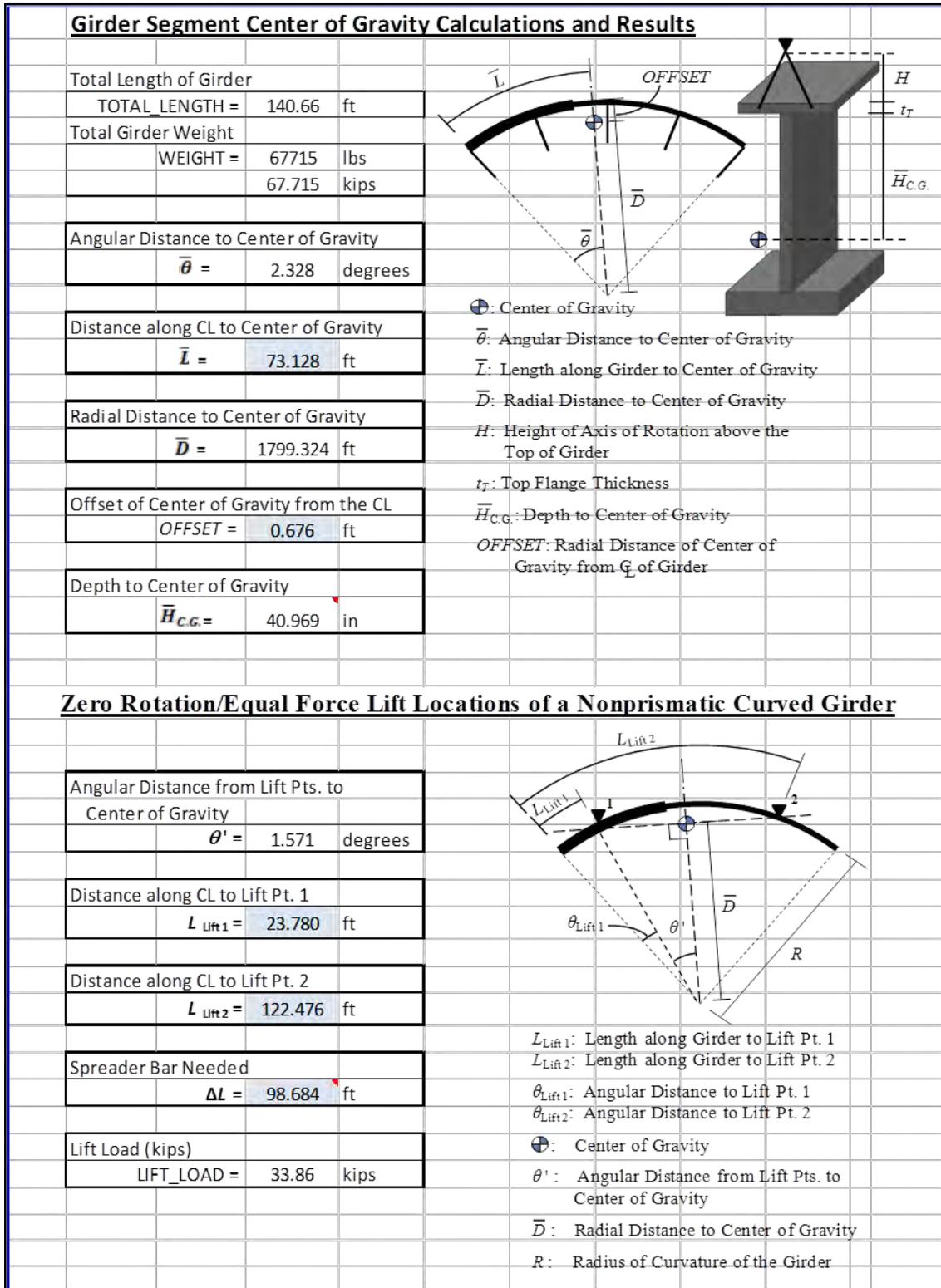
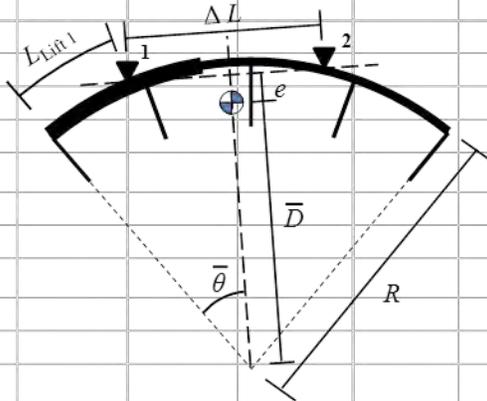


Figure 6-17 Sample Output of Optimum Lift Locations for Girder Segment for UT Lift

Predicted Rigid Body Rotation and Lift Reactions of a Nonprismatic Curved Girder



$L_{Lift 1}$: Length along Girder to Lift Pt. 1

ΔL : Chord Length to Lift Pt. 2
(Spreader Bar)

e : Eccentricity Between the Line of Support and the Center of Mass

⊕: Center of Gravity

$\bar{\theta}$: Angular Distance to Center of Gravity

\bar{D} : Radial Distance to Center of Gravity

R : Radius of Curvature of the Girder

Distance along CL to Lift Pt. 1	$L_{Lift 1} =$ 33.000 ft
Chord Length Between Lift Pts.	$\Delta L =$ 80.000 ft
Axis of Rotation above Top of Girder	$H =$ 30.000 in
Eccentricity	$e =$ 0.232 ft 2.783 in
Rotation	$\theta_{Rigid} =$ 0.039 rad 2.207 degrees
Reactions	Lift Point 1 = 33.75 kips Lift Point 2 = 33.96 kips

WARNING: Reactions Not Equal. Valid Only for 2 Cranes

Calculate Rotation & Stress

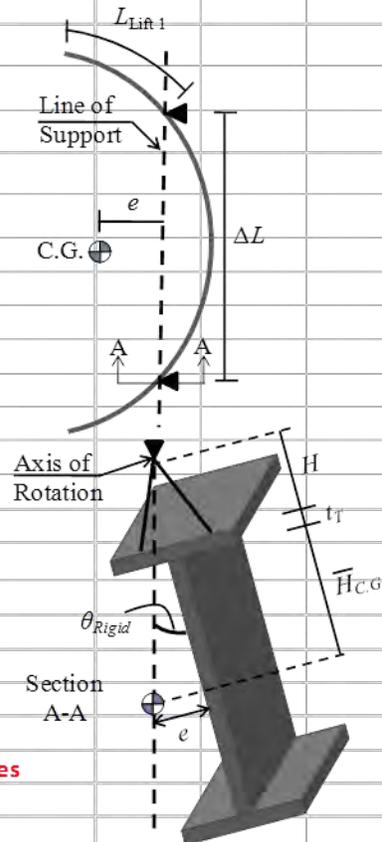


Figure 6-18 Sample Input Page for Actual Spreader Beam and Lift Locations from UT Lift

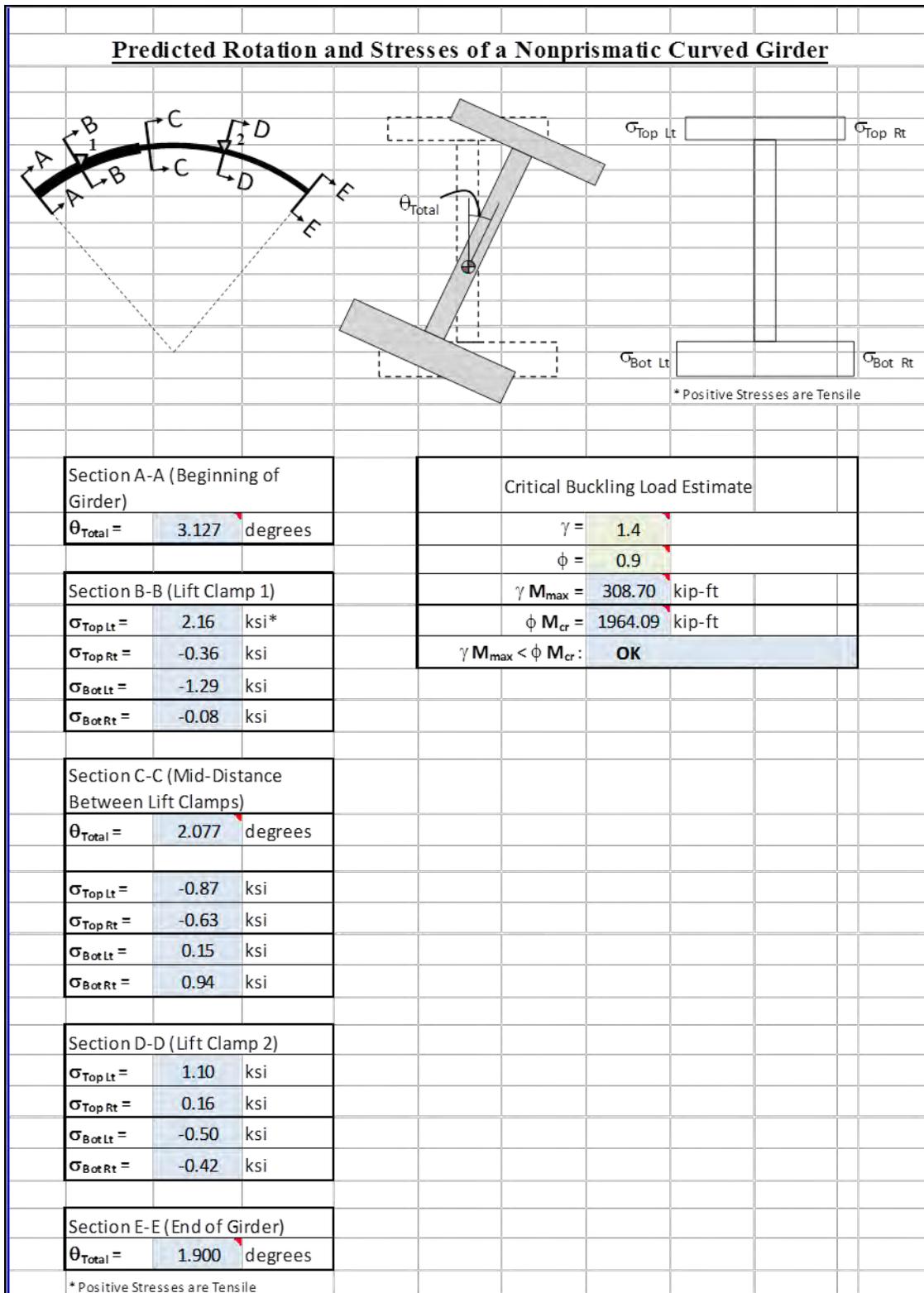


Figure 6-19 Sample UT Lift Output of Girder Stresses and Rotation at Ends and Middle of Segments

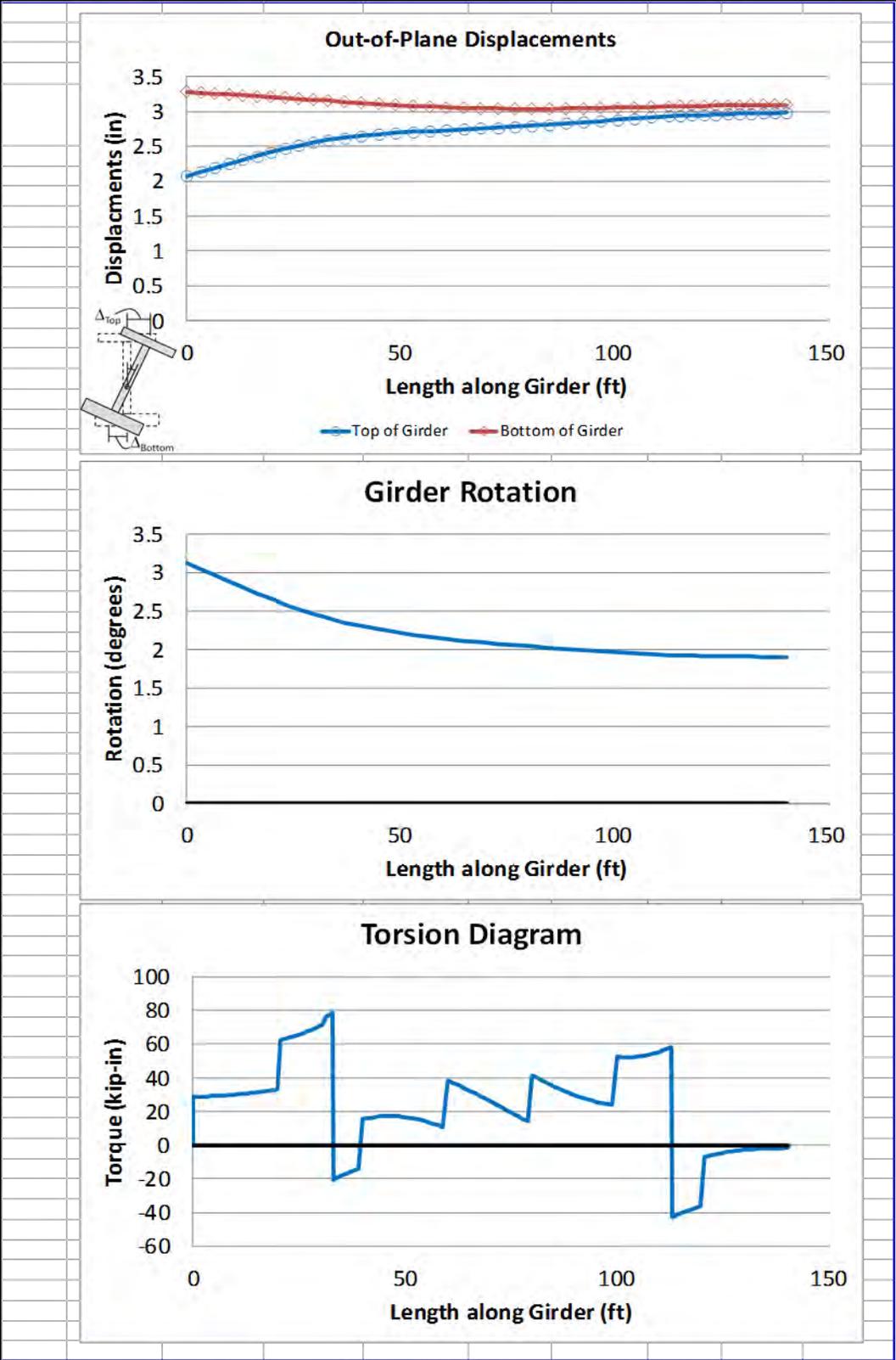


Figure 6-20 Sample UT Lift Graphs of Girder Displacements and Torque

SECTION 5. BEHAVIOR OF STRAIGHT AND HORIZONTALLY CURVED GIRDERS DURING ERECTION AND CONSTRUCTION

As noted earlier in the chapter, there are a variety of models and analysis methods that can be used to study the stability behavior of steel bridge systems. The accuracy of the various modeling techniques depend heavily on the types of details that are used in the bracing as well as the combination of temporary and permanent supports that may be used during construction. While grillage models are by far the easiest models to create, the simple models that result may be lacking sufficient detail to accurately capture the behavior of the system at critical stages in the construction. The stability of the system can be significantly affected by the support locations on the girder cross-section, as well as the interaction of the braces with the cross-sectional stiffness. The most accurate modeling method is, therefore, 3D models that can more accurately capture the impact of details and cross-sectional distortion on the bridge behavior. However, the drawback to utilizing 3D modeling is the time necessary to properly model the bridge, as well as validation methods that are necessary. The modeling time increases dramatically for bridges with horizontal curvature combined with potential support skew. Further complicating the modeling issues is identifying and modeling the critical stages in the partially-erected bridge. As part of TxDOT project 0-5574, a computational analysis tool named UT Bridge was developed to specifically address the analysis issues with steel bridges during construction. The UT Bridge program is a 3D finite element program that is free to download at <http://fsel.engr.utexas.edu/software/index.cfm>.

The goal of the software is to create accurate representations of the structural system during construction through the use of a user-friendly graphical user interface (GUI). The software consists of 14 input screens that prompt the user for input related to the basic bridge geometry, the erection sequence, locations of shore towers or temporary holding cranes, and the concrete deck casting sequence.

This section focuses on the determination of critical stages that should be evaluated throughout the construction process. Because the critical stages vary among bridges, results from the UT bridge program will be used to describe how the various stages can be identified and analyzed. Therefore this section has been divided into four subsections. Following this introduction, an overview of the UT Bridge software is provided. A typical erection sequence for a two-span horizontally curved girder is then presented followed by an evaluation of a staged concrete deck placement.

6.5.1 UT Bridge

The UT Bridge Software is a 3D finite element program that was specifically developed to target the behavior of straight and horizontally curved girders during erection. The software is capable of conducting a first-order structural analysis or an eigenvalue buckling analysis. The software consists of 14 input screens that prompt the user for input that is readily available from shop drawings or bridge plans. The software allows the user to fully define the steel bridge system and then experiment with a variety of

erection or concrete deck placement scenarios to evaluate the behavior of the bridge during construction and provide confidence in the erection and construction plans.

At the start of the program, the user can select to either start a new project or continue with an existing project. The user selects between conducting an erection analysis or a deck placement analysis on either a straight or horizontally curved girder system. The input panels prompt the user for the necessary information on the basic geometry of the bridge. For example, Figure 6-21 shows the third of fourteen input panels that prompts the user for the span lengths and types of bearings. In addition to the basic span lengths of the girders, the user can specify support skew. Many of the panels have a button with a magnifying glass that provides the user with additional information. For example, if the user selects the magnifying glass on the third panel, the information panel shown in Figure 6-22 appears and provides the user with the sign convention for the support skews.

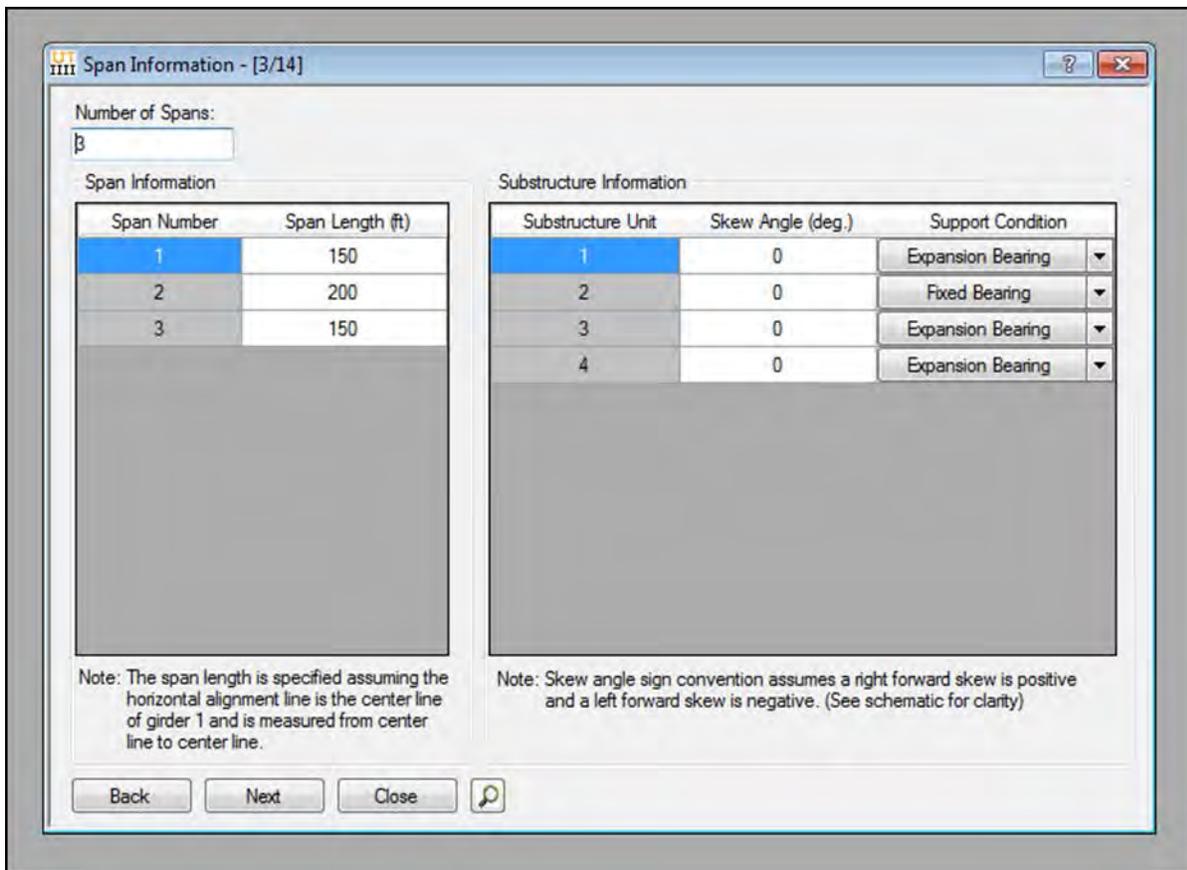


Figure 6-21 Typical UT Bridge Input Panel for Basic Bridge Geometry

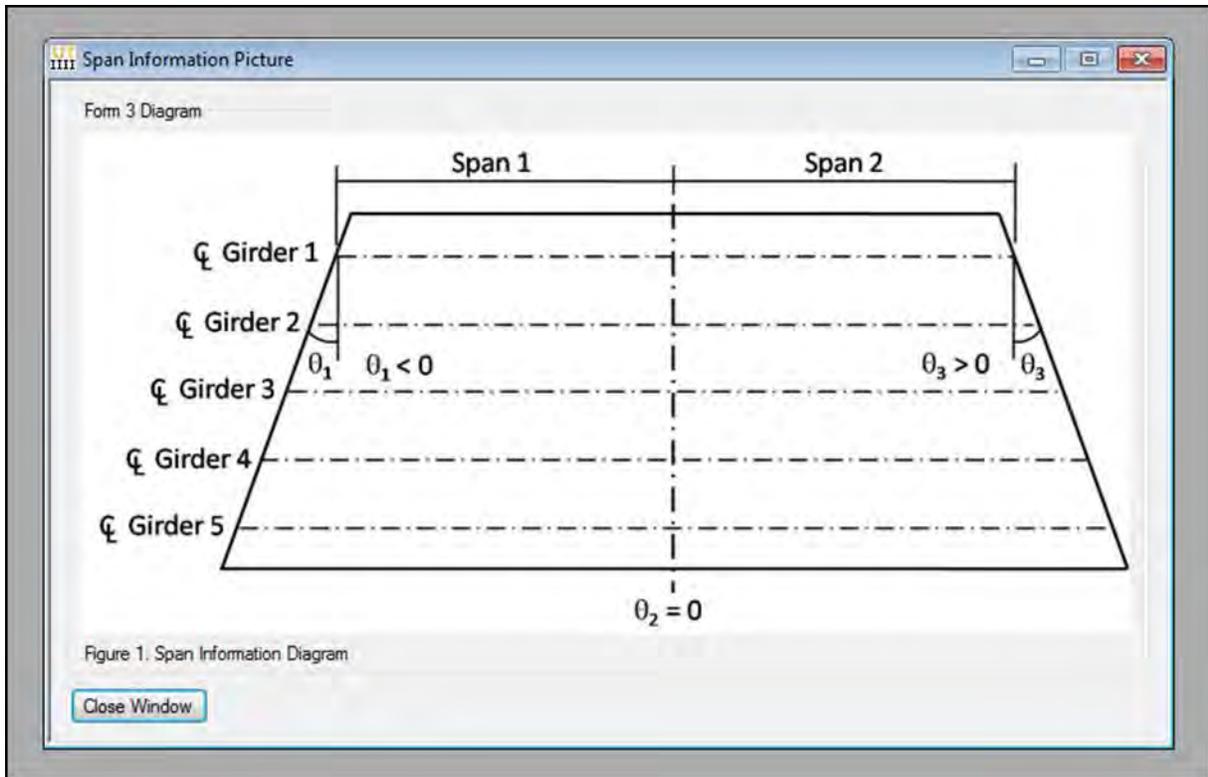


Figure 6-22 UT Bridge Pop-Up Screen with Additional Information on Skew Sign Convention

Figure 6-23 shows the input screen for the girder properties in which plate sizes are defined for the different girders in the bridge. Many of the input screens have options for the user to define the properties for girder 1 and then select “All Girders Uniform” so that the properties are copied to each girder. If the option is selected for horizontally curved girders the program assumes that the plate transitions are radial for the other girders. If the girders have skewed supports, the properties of each girder must be defined since the girder lengths and plate transitions may be much more variable. Some of the input screens have a button that shows a “pencil” which, if selected, initiates a pop-up window with the schematic of the defined element. For example, if the button is pushed while on the “girder 1” tab for the girder input shown in Figure 6-23, the pop up screen shows an elevation of girder 1 with the defined plate sizes as shown in Figure 6-24. The user can select to show either the plate thickness or the plate widths.

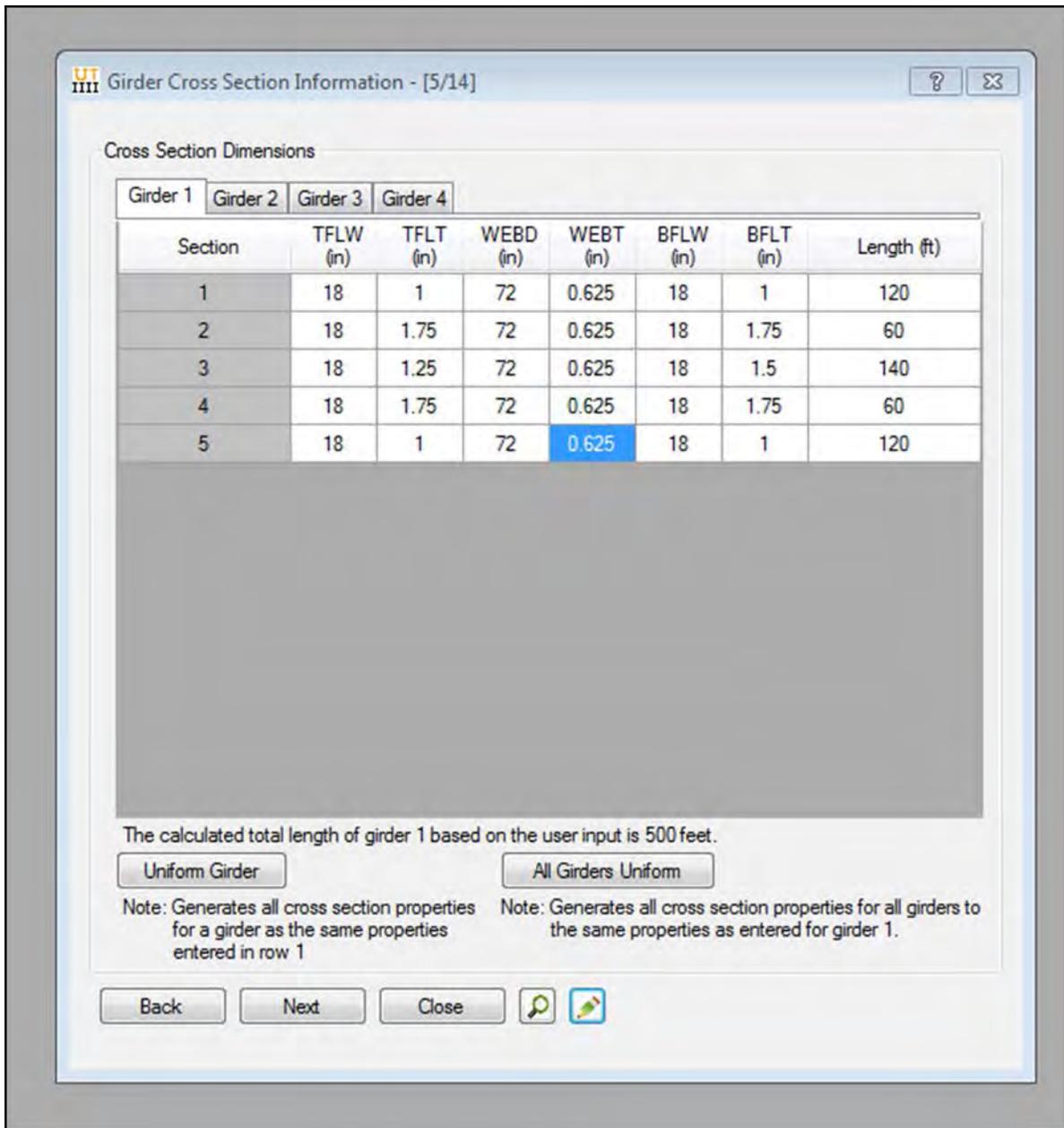


Figure 6-23 UT Bridge Input Screen for Girder Properties

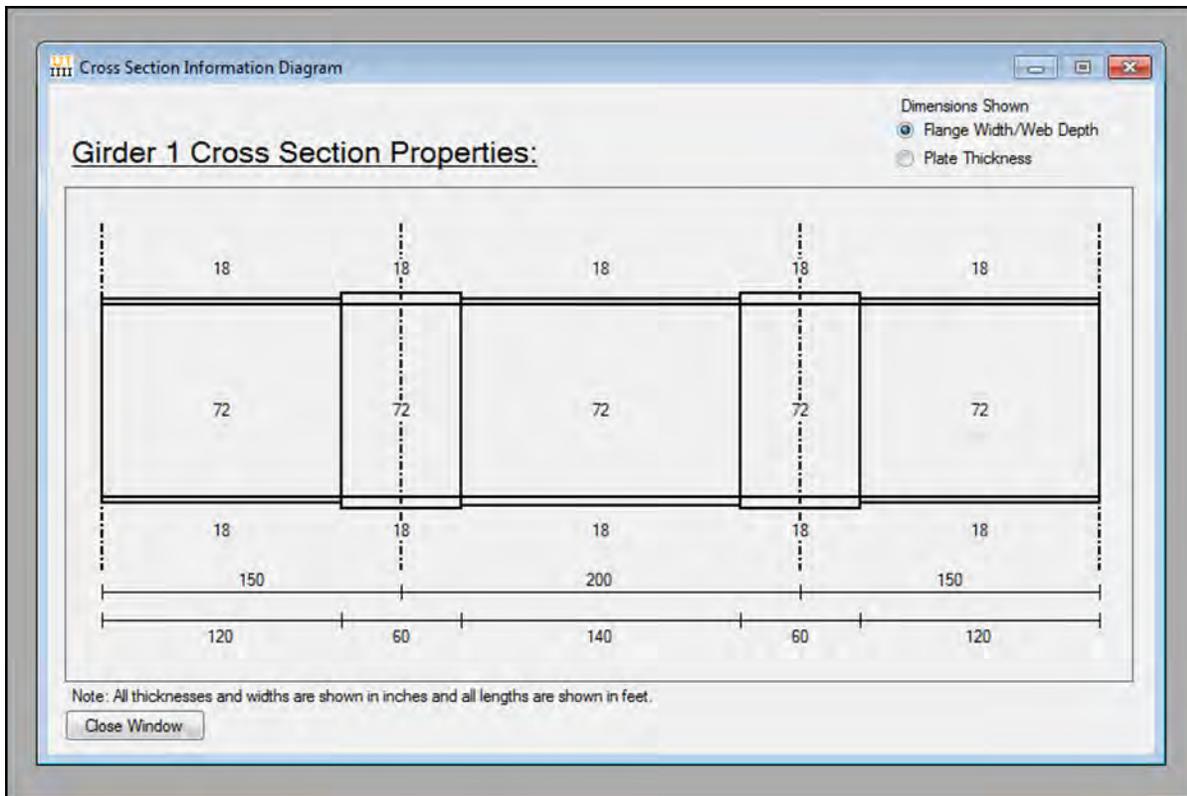


Figure 6-24 Information Panel on the Defined Model for UT Bridge

The support and intermediate cross-frames are specified separately. The program assumes that there is a connection plate (stiffener) at each cross-frame location and the user specifies the plate size on the same panel the cross-frame member sizes are specified. The cross-frames are assumed to be “X-Type” cross-frames with two diagonals and lateral struts at the top and bottom; however a tension-only diagonal system is assumed in which case only the tension diagonal contributes to the stiffness. For different cross-frames systems, equations for the stiffness relative to “X-type” cross-frames are available and the member sizes can be adjusted accordingly. For radial cross-frames, the software has a “uniform spacing tool” that can help to define cross-frames along the length of the girders. If the cross-frames are not radial, the user can specify the cross-frames separately between each set of girders. The user also specifies the cross-frames’ sizes at the supports and the size of the bearing stiffeners. The program offers two options for the bearing stiffeners, plate stiffeners or pipe stiffeners. Pipe stiffeners have been utilized on bridges with heavily skewed supports to facilitate stiffer connections between the cross-frames and the girders at the supports (Quadrato, et. al., 2010). The pipe stiffeners also offer a significant amount of warping restraint to the girders and the program is capable of including the benefits of these stiffeners on the torsional behavior. The user is prompted for the plate and width if a plate bearing stiffener is selected or the pipe diameter and thickness if a round pipe stiffener is specified at the support locations.

By the ninth input screen, the erection model has been fully defined and the user then begins to focus on the erection sequence. The user specifies the number of analysis runs that are to be conducted to analyze the entire erection sequence. The user can analyze the behavior as each individual girder segment is erected or may choose to only consider the analysis cases that are deemed “critical”. The following section outlines methods to evaluate the critical analysis cases that need to be considered. The software allows the user to input wind loads if desired to evaluate the behavior of partially erected systems under wind loads.

The software can allow the user to consider the effectiveness and placement of either shore towers or temporary holding cranes. For holding cranes, the user specifies the location of the holding crane force along the girder length and also on the cross-section, as well as the magnitude of the holding crane force. In many situations, the desired behavior of the holding crane is that it acts like a rigid support much in the way shore towers are considered to act. To determine the required crane force, the analysis can first be conducted by putting a shore tower at the particular location and determining the reaction. The analysis can then be rerun by replacing the shore tower with a temporary holding crane applying an upward force at the specified location.

The user can decide to either conduct a first order structural analysis or an eigenvalue buckling analysis for up to five buckling modes. As noted earlier in the chapter, the first mode is the primary mode of interest since it will have the lowest buckling load. Before the analysis is conducted the user has the ability to first visualize the bridge to make sure that the proper geometry has been captured and that the correct erection sequence that was desired is being modeled.

UT Bridge automatically creates the three-dimensional finite element mesh of the girder systems using a combination of shells and beam elements. The user can choose one of three meshing options located under the “Tools” category on the Menu Bar. There are three meshing options: Coarse, Normal, or Fine. In the coarse meshing option, element divisions are at a 4-foot-spacing along the length of the girder with four web elements through the depth of the web. In the normal meshing option, element divisions along the girder length are at a 2-foot-spacing with four elements through the depth of the web. In the fine meshing option, the element divisions are at a 1-foot-spacing along the girder length with four elements through the depth of the web. In most applications, the normal meshing option results in good estimates of the behavior, however if computer resources are short the user may opt to use the coarse mesh or the fine mesh may be selected to provide more nodes along the length to better capture the behavior. The finer mesh may provide better estimates of the behavior if local plate buckling is an issue on the bridge.

In addition to conducting a girder erection analysis, UT Bridge also allows the user to conduct a deck placement analysis from the same model that was described in the erection analysis. On input form 10, the user provides the information about the concrete deck properties. The pertinent geometrical information includes the thickness of the deck, the haunch properties, and the overhang widths on the two sides of the

bridge. The user also defines how many deck segments are cast and the number of concrete placements. The deck segments are usually divided into the number of individual positive moment regions and negative moment regions where the deck is likely to have construction joints. The number of deck placements refers to how many different time periods that the concrete will be placed. For example, the contractor may decide to place all of the positive moment regions in succession and allow that concrete to cure for a day or more before placing the negative moment regions. The reason that the number of concrete placements is important is that UT Bridge allows the user to take into consideration the time-dependent stiffness gain in the concrete based upon test results from Topkaya (2002).

Topkaya conducted experiments on the stiffness gain in the concrete during the hours immediately after casting. The concrete can quickly gain stiffness and create composite action between the freshly placed slab and the steel girders. Although the strength of the concrete is obviously not adequate to develop large increases in the bending strength of the cross-section, the stiffness gain can substantially affect the deformations in the girders and the required bracing. Therefore, UT Bridge allows the user to express the concrete placement in terms of hours from the start of the deck construction. The analysis then takes into consideration the impact of the freshly placed concrete on the bridge behavior. While the program allows the stiffness to be directly taken from the work from Topkaya (2002), the user can also directly input a different stiffness if the value is known or to obtain a measure of the impact of the variation of the stiffness on the behavior. If the user does not wish to consider the impact of the concrete stiffening on the behavior, a single placement can be specified and only the weight of the wet concrete is considered.

Because the program automatically generates the mesh, isolated cases have been found where a flange width transition or other localized discontinuity may lead to a meshing problem that has issues running. Changing the meshing option to one of the other two options (i.e. Normal to Course or Fine) typically fixes this problem.

6.5.2 Determining the Critical Erection Stages

As noted at the outset of the chapter, the difficult aspect of erection engineering is ensuring the stability of the bridge at early stages in the erection when not all of the bracing is present in the structure. In many situations the most precarious condition of the bridge is the first girder segment when there is no existing structure besides the pier caps/abutments to brace the girder. The use of shore towers or temporary holding cranes can significantly increase the cost and time of erection and therefore such falsework is preferred only in cases where it is absolutely necessary. Because the system generally becomes more stable as more of the structure is erected, not all stages of the erection need to be modeled. However, without experience, it is not always clear which stages need to be considered. In this section, the erection of a four-girder steel bridge is evaluated to demonstrate some of the critical stages that need to be considered in the erection. Due to the variability of geometries and loading conditions on bridges, the critical stages will vary from bridge to bridge; however with experience, most engineers will be able to rule out some of the specific stages that

need not be considered. The bridge that is to be modeled is depicted in Figure 6-25. The two-span unit represents a bridge that is simple enough in this example that the erection sequence can be covered in an efficient manner; however there are enough complexities that some of the difficulties in analysis can be discussed. The girders are doubly-symmetric and the plate sizes in the bridge are indicated. The bridge has two 175 ft-spans. The girders have 14 in. x 1 in. flanges in the positive moment regions and 16 in. x 1.5 in. flanges in the negative moment region. The web is 66 in. deep and 0.5 in. thick. The girders have a splice at the locations of the flange transitions so each girder will be shipped to the job site in three segments: two 135 ft segments and the 80 ft segment for the negative moment region. The cross-frames are spaced 25 ft on center along the bridge length; however, in many situations, not all of the cross-frames are installed as the girders are lifted into place. UT Bridge allows the user to fully define the bracing layout but to leave out every other cross-frame during the analysis for the erection scenario, which was assumed in this analysis.

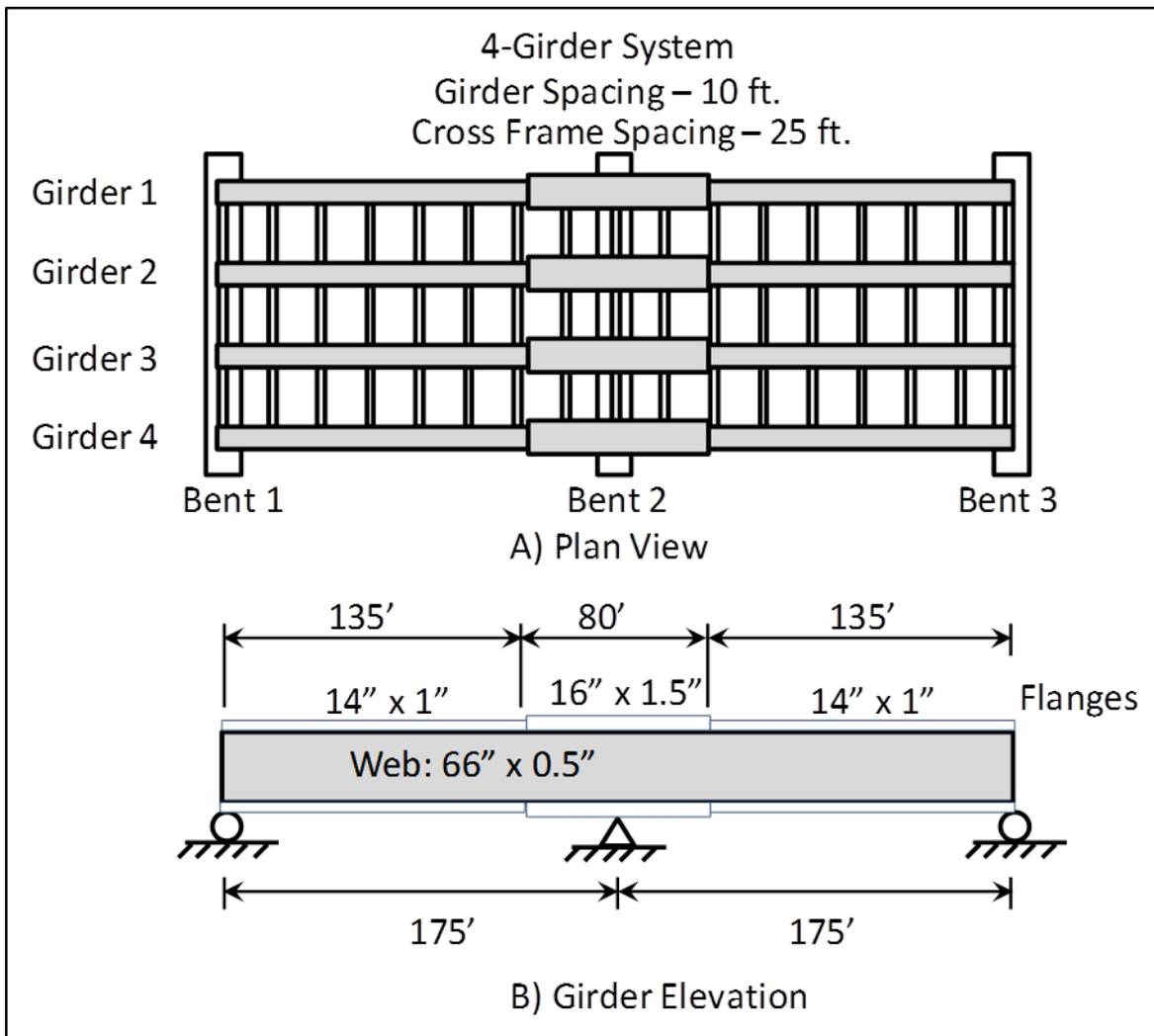


Figure 6-25 Plan View and Elevation of Two Span Bridge Modeled

There are a variety of methods that could be used to erect the four-girder system depicted in Figure 6-25. Perhaps the “easiest” method of erecting the girders is to place a shore tower between Bents 1 and 2 so that the 135 ft long segments can first be lifted into place and then the 80 ft long segments can be lifted and the splice can be completed in the air so that the girders cantilever over Bent 2 leaving the 135 ft long segments to be lifted and spliced to complete the girder erection. However, the drawback to such a scenario is the time required to assemble and erect the shore tower as well as the potential traffic interruption that such a tower will cause below the bridge area. As a result, the first preference for most cases will be to erect the bridge without the use of a shore tower. Therefore, in a different and more likely scenario, one of the girder splices would be completed on the ground to make 215 ft long segments that can be lifted into place and set on two of the supports (Bent 1 and Bent 2). Therefore the girders will cantilever over Bent 2 after the first segments are erected and the bridge would be completed in the manner described above with the final 135 ft long segment of each girder lifted and connected with an air splice, thereby completing each girder line.

Since the girder segments would be lifted individually in the latter scenario, there are eight separate lifts for the steel girder bridge as denoted in Figure 6-26. Although each case can be analyzed to evaluate the safety of the erection scenario, some of the cases are more critical than others and therefore each case does not necessarily have to be considered when evaluating the potential scenarios.

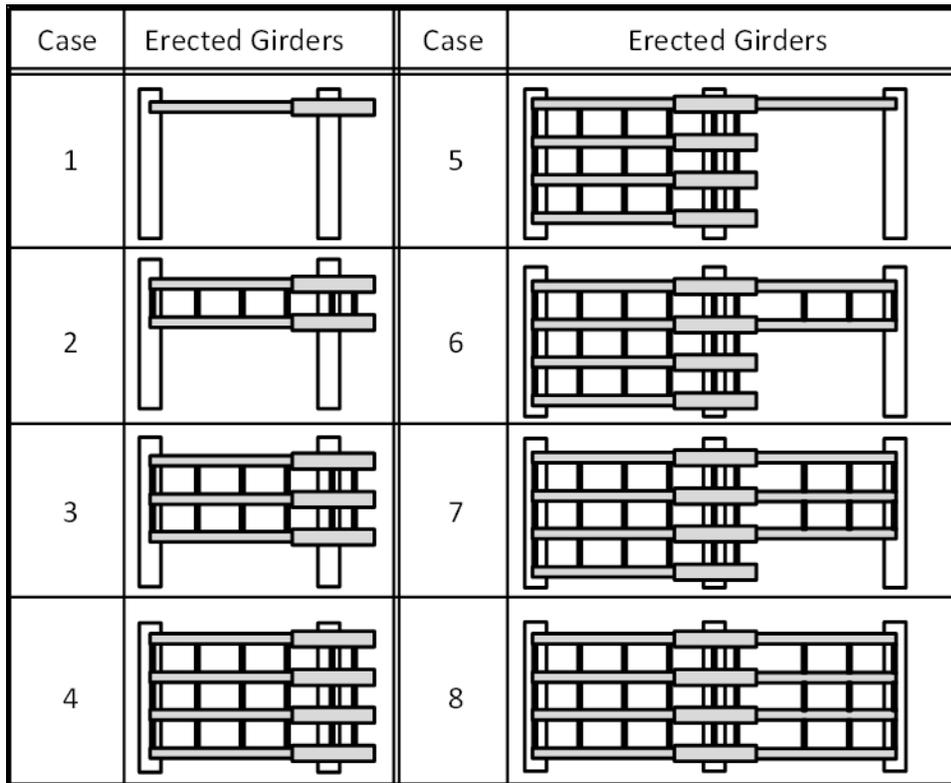


Figure 6-26 Plan View of Erected Girders for Eight Erection Stages

Table 6-1 shows a summary of eigenvalue buckling results for the eight different load cases. Some erectors may opt to initially only install half the cross-frames, so the analytical results are for cases where every other intermediate cross-frame was installed. The erector would then come back and install all the cross-frames prior to deck placement. The applied loading on the girders consisted of the unfactored self-weight of the steel section. Therefore, an eigenvalue greater than 1 indicates that the critical load is higher than the self-weight and a value less than 1 means that the critical load is less than the self-weight, indicating that the section will definitely buckle during erection.

Case	Eigenvalue
1	0.48
2	1.57
3	1.84
4	1.96
5	1.32
6	2.36
7	3.15
8	3.55

Table 6-1 Summary of Eigenvalues for Erection Scenario in Figure 6- 25 (half cross-frames installed)

With an eigenvalue of 0.48, the girder is clearly not adequate to support its self-weight in the case 1 lift where girder 1 is erected onto the abutments. Figure 6-27 shows the buckled shape of girder 1 from the Case 1 erection. The analysis assumes that the girder will be braced at the pier locations, but the eigenvalue shows that additional bracing or a temporary support will be required. When Girder 2 is added the eigenvalue increases to 1.57, which indicates that the critical load is 57% higher than the self-weight of the section. The buckled shape of Girders 1 and 2 for Case 2 in the erection scenario is shown in Figure 6-28. In analyzing the different cases, it is clear that Case 1 will usually always be one of the most critical cases. Depending on the length of the lifts, Cases 1 and 5 are potentially critical, since these cases will have the largest unsupported length of any of the cases. For this particular analysis, Case 5 has an eigenvalue of 1.32 which is probably borderline with regards to safety. Adding temporary bracing or a holding crane would probably be considered at this stage.

Considering the Case 5 buckled shape, Figure 6-29, the unbraced length of Girder 1 is 150 ft since there is one cross-frame on the cantilevered portion of the girders (i.e. 135 ft + 15 ft = 150 ft). The buckling capacity of Case 5 can be improved if the erector was able to add temporary bracing at the end of the cantilever section. The benefit can be found in UT Bridge by adding another cross-frame line located at 215 ft along the length of the bridge, which increases the eigenvalue to 1.50. The actual bracing that is used by the erector will not consist of a permanent cross-frame, but instead could consist of some wide flange shapes spanning between the cantilever portion of Girders 1 and 2

that could be clamped to the top and bottom flanges. Once the remainder of Girder 2 is added (Case 6), the eigenvalue increases to 2.36 and the temporary bracing could be removed.

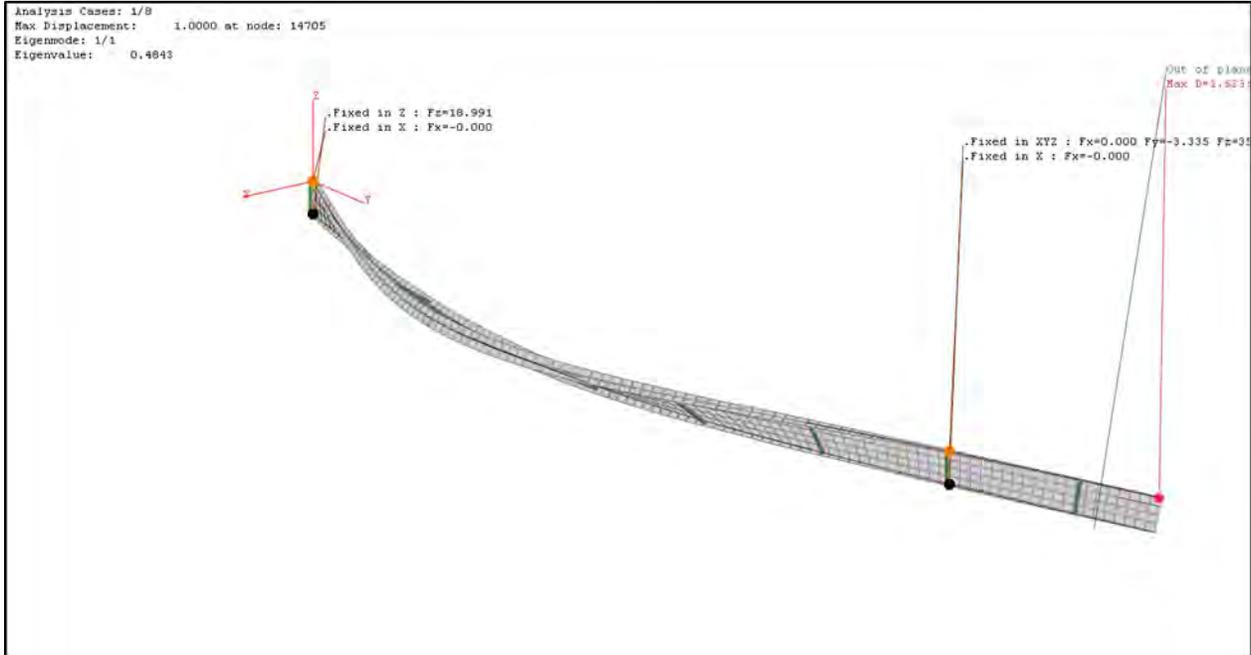


Figure 6-27 Buckled Shape of Girder 1 with No Intermediate Bracing (Case 1)

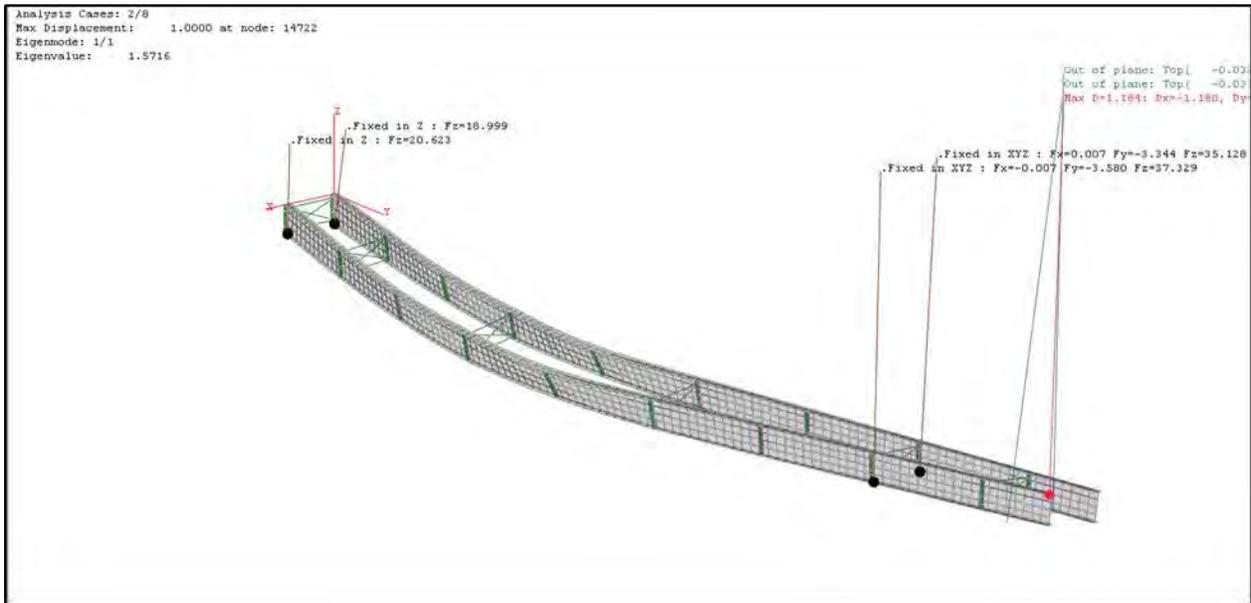


Figure 6-28 Buckled Shape of Girders 1 and 2 in Case 2

Figure 6-30 shows the buckled shape for Case 8 when all of the girders have been fully erected, but only half of the cross-frames are included. The behavior of the system if the full bracing is added is discussed later in this section.

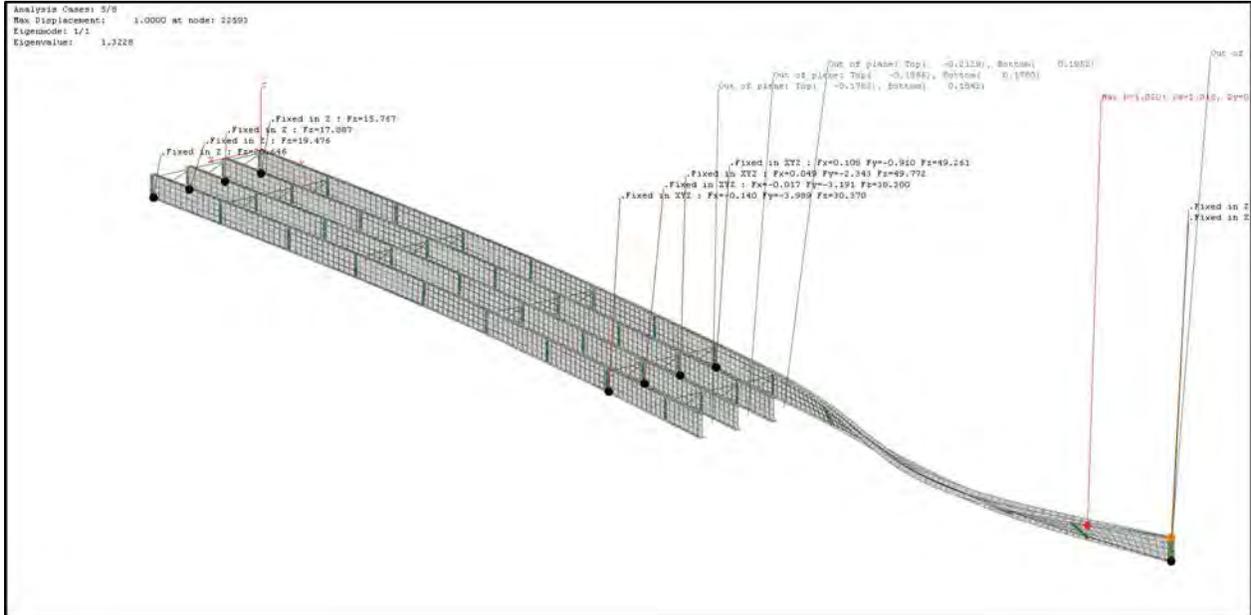


Figure 6-29 Case 5 Buckled Shape

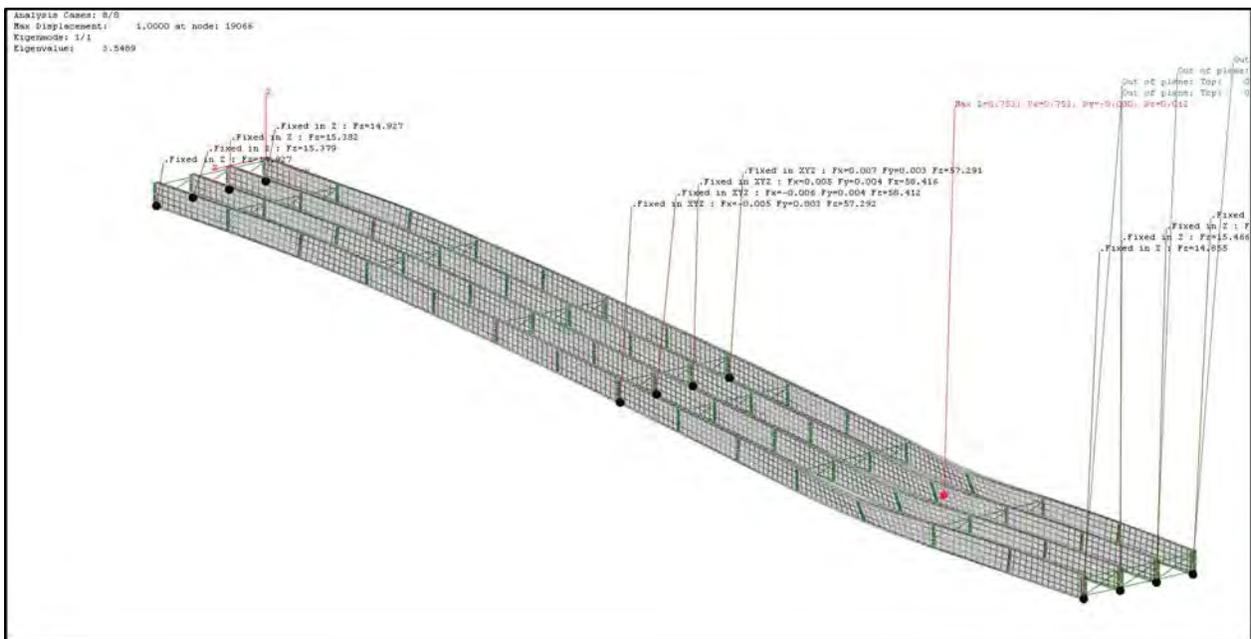


Figure 6-30 Case 8 Buckled Shape

Based upon the eigenvalues, the erection scenario is inadequate for the Case 1 condition. In an effort to improve the behavior of the girder during erection, the behavior with the addition of a holding crane is evaluated. However, if a holding crane is to be used, the force that the crane should apply to the girder at the lift point typically needs to be specified. If a shore tower were to be used in the erection scheme, the tower would have been erected to a height that puts the girder splice at the proper elevation to facilitate completion of the splice. To determine the required lifting force for the holding crane, an analysis will first be conducted in UT Bridge by modeling a shore tower in Analysis Case 1 with only Girder 1 erected. UT Bridge allows the user to specify the location of the shore tower/holding crane along the length of the girder. Depending on the site layout, the engineer may have limitations on where the shore tower or holding crane can be placed and therefore should select the appropriate location. Since there are no limitations in this problem, a location 90 ft from the first bent was selected for the support location, which is very near the middle of the 175 ft-span.

The buckled shape of the analysis with the shore tower is shown in Figure 6-31. The eigenvalue with the shore tower is 5.81 and the vertical reaction is 21.3 kips. Using this reaction, the shore tower can be replaced in UT Bridge with a holding crane in the form of an upward force of 21.3 kips at the same location, except the load is applied at the top flange since most holding cranes will connect to the girders with a flange clamp at the top flange. The buckled shape for the load case with the holding crane is shown in Figure 6-32. The eigenvalue with the holding crane is 4.58, which is less than the value of 5.81 for the case of the shore tower. The difference in the eigenvalues for these two cases with the falsework is due to the assumptions that twist and lateral movement is restrained at the shore tower. For the case with the holding crane, the analysis only applies an upward force, but the girder can still translate and twist at that location.

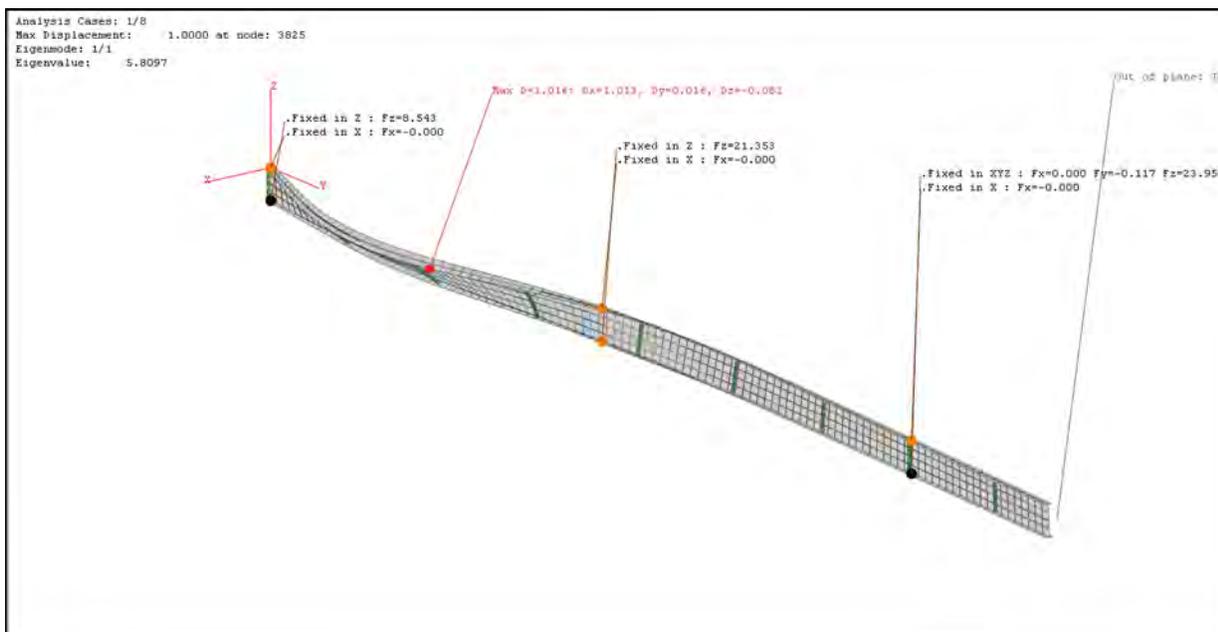


Figure 6-31 Buckled Shape for Case 1 with Shoring Tower 90 ft from Bent 1

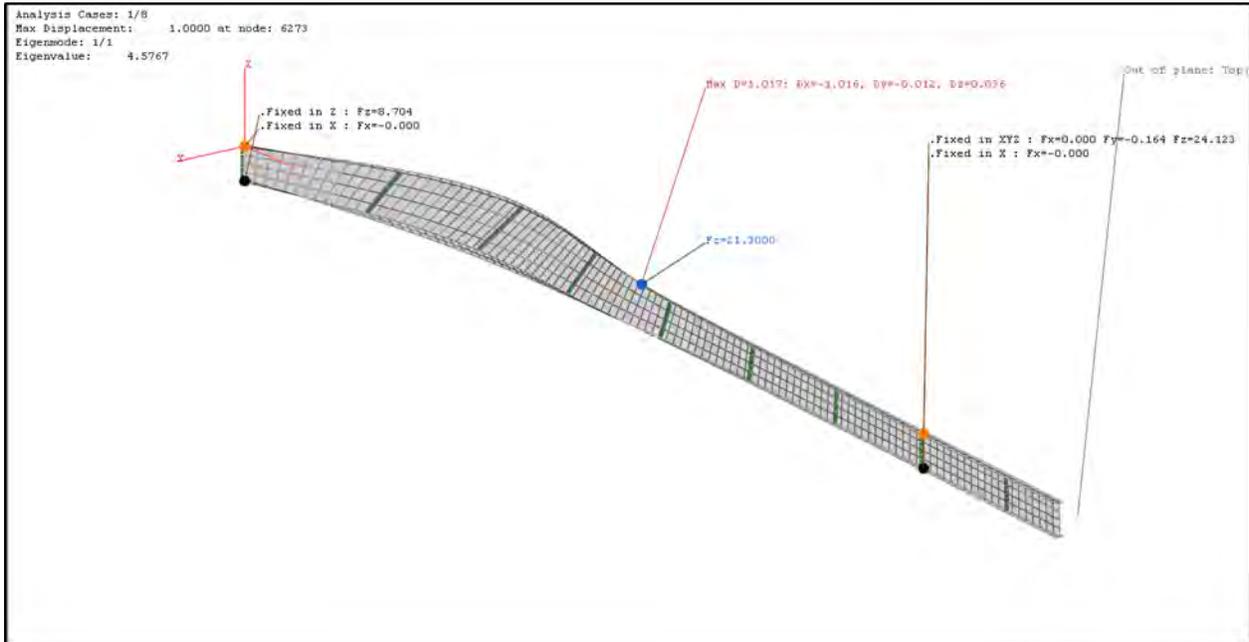


Figure 6-32 Buckled Shape for Case 1 with a Holding Crane 90 ft from Bent 1

If the holding crane is removed after Girder 2 is erected, the eigenvalue will be 1.57 as was given in Table 6-1. If the engineer requires all intermediate cross-frames to be installed between each girder at the time of erection (instead of only half), the buckling capacities of the system will increase to the values given in Table 6-2. The percent increase is substantial, however the engineer can evaluate the behavior and make decisions that are the most efficient in terms of time and resources available for the job.

Case	Eigenvalue	Percent Change Compared to Table 6.1 (half Cross-frames)
1	4.58 (holding crane)	854% (due to holding crane)
2	1.87	19%
3	2.59	41%
4	3.12	59%
5	1.33	1%
6	4.65	97%
7	5.74	82%
8	6.56	85%

Table 6-2 Summary of Eigenvalues for Erection Scenario in Figure 6- 25 All Cross-frames Installed and Holding Crane for Case 1

With the addition of the holding crane, Case 5 is the most critical case. As noted earlier, temporary bracing added near the end of the cantilever can raise the eigenvalue to 1.5.

A temporary holding crane could also be considered, which would greatly increase the stability much like the outcome of Case 1.

In the erection of curved girder systems, there are a number of other considerations that the engineer needs to be aware of. For example, consideration of the torsional stability of the partially erected system is a major consideration. In the curved girder system shown in Figure 6-27, when only two girders are on the supports, the interior girder has a negative vertical reaction which indicates uplift. UT Bridge gives a warning when uplift is a problem; however engineers need to check the vertical reactions to make sure that a typical erection scenario doesn't have uplift cases. For the example in Figure 6-27, the engineer may need to ensure that a tie down is provide at the first support to make sure that the girder system has adequate torsional stability.

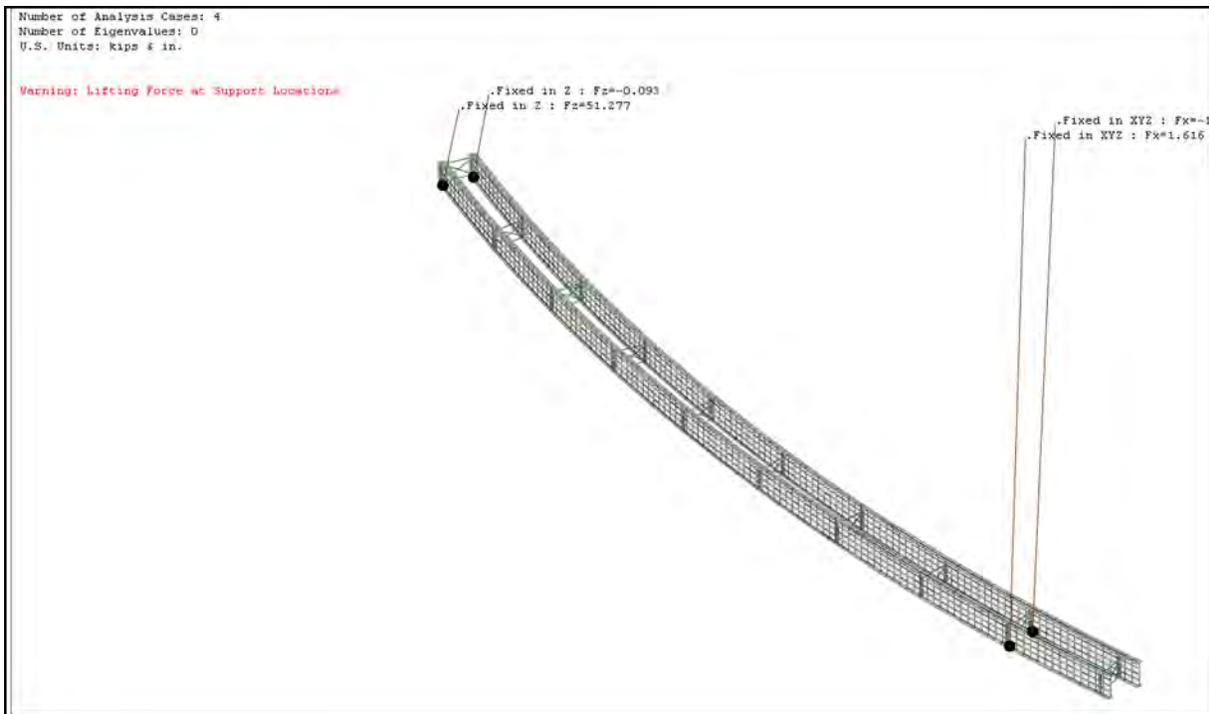


Figure 6-33 Potential Uplift in Horizontally Curved Girder during Erection

6.5.3 Evaluation of Concrete Placement

Another critical stage for the stability of a girder system is during the placement of the concrete deck. Although the full bracing is installed to the girders at this stage, the applied loads during deck placement are significantly higher compared to the erection stage where the loads only consist of the weight of the steel section. During placement, the concrete is a fluid and offers no restraint to the steel girder system. As a result, the deck placement conditions offer another critical stage that must be evaluated.

Consideration of the stability of the system during the deck placement will typically rely upon the resistance of steel girders and the bracing system, which usually consists of the cross-frame systems. Although stay-in-place forms do enhance the stability of the girder system, due to flexibility in the connections between the form work and the girders, AASHTO does not permit the forms to be relied upon for bracing. The UT bridge software can be used to conduct an eigenvalue analysis to evaluate the stability of the girder system during deck placement.

As discussed in Section 6-3.2, the eigenvalue represents a multiplier on the applied load. The loads will consist of the self-weight of the girders, concrete deck, formwork, and the construction personnel/equipment. There is often some unpredictability in the distribution of the applied construction loads during the deck placement. Relative to the applied serviced loads to the structure, a target eigenvalue during the deck placement will usually be in the range of 1.5-1.75. This range of eigenvalues will usually account for some of the uncertainty in the load distribution on the structure. In some cases, the engineer may be able to account for some of the beneficial restraint that comes from previously set concrete as is explained later in this subsection.

In addition to the stability of the system, there are a number of other behavioral concerns that must be evaluated during deck placement. Some of the factors include potential lift-off at the supports of continuous and/or horizontally curved girders as well as cambering requirements so as to achieve the proper deck stiffness. As noted earlier, UT Bridge includes the ability to consider the time dependent stiffness gain in the concrete. The stiffness gain of the concrete during the cast impacts the brace forces that develop in the cross-frames, the rotational behavior of the girders during construction, and also the vertical displacements of the bridge during deck placement.

For an example, the two-span straight girder considered in Section 6.5.2 was evaluated during a simulated deck placement. The deck consisted of a 10 in. thick concrete deck with 4 ft overhangs as shown in Figure 6-34. Although the 350 ft total length of the bridge may be a bit long to complete in a single cast, some contractors would like to conduct a single cast starting at one end of the bridge and placing the concrete towards the other. However, it's important to ensure that the girders will not lift off the support during the deck placement. The critical stage to check for lift-off will be when the concrete has been totally placed on one span as depicted in Figure 6-35. The UT Bridge solution for the critical case that might cause uplift is shown in Figure 6-36. The analysis shows that uplift would not occur; however the reactions at the far support are relatively small (approximately 1000 lbs. and 1500 lbs. at the exterior and interior girders respectively). Since a load factor of 1.0 was used in the analysis, these forces are generally too close to uplift and either a tie-down at the supports should be used or a staged deck cast should be used.

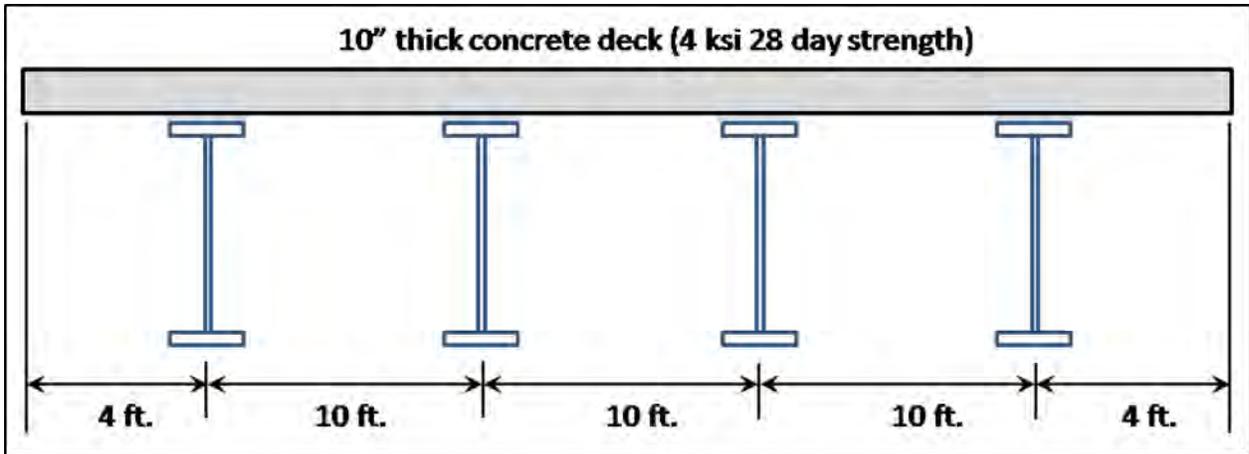


Figure 6-34 Cross-section of Four Girder Bridge Considered in the Concrete Placement Evaluation

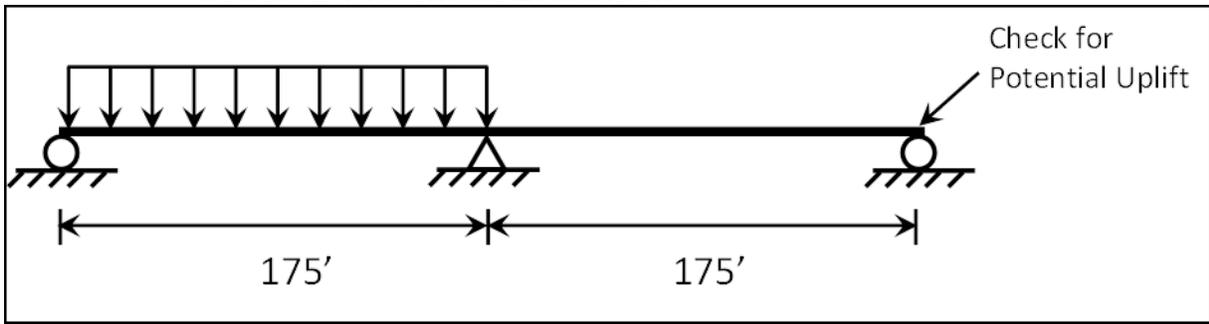


Figure 6-35 Critical Loading to Check Uplift When Continuous Deck Cast is Proposed

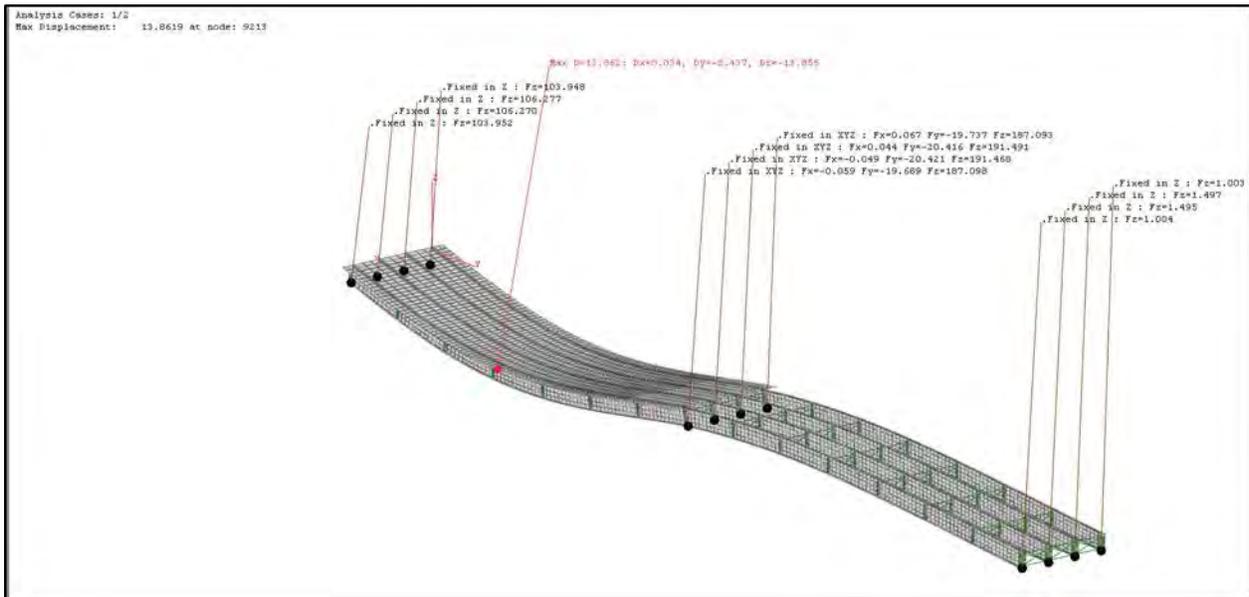


Figure 6-36 UT Bridge Results for Potential Uplift during Continuous Deck Cast

UT Bridge also provides graphs of nodal data such as stresses or displacements. Figure 6-37 shows a graph of the vertical displacement along the bridge length. As expected, the vertical displacements for the simulated continuous deck cast essentially give symmetric displacements in the two spans. There is a very small difference due to the rounding of the span lengths that happens in the meshing; however the predicted vertical displacement is approximately 8.5 in.

As noted earlier, these values can be affected due to the time dependent stiffness gain in the material. The program calculates the increase in concrete stiffness with age and incorporates these values in subsequent analyses. The effect of the concrete stiffening can be considered in cases where concerns exist about the stability of the systems since the partially cured concrete contributes to the girder stability; however neglecting this effect is generally conservative. The early stiffening of the concrete can also have an impact on the resulting camber and the software can account for the potential partial composite action that may develop. For example, instead of a continuous deck pour, a case where the deck is divided into three segments of length 135 ft, 80 ft, and 135 ft as depicted in Figure 6-38 was considered. The time of placement for the deck casting was taken as 0 hours for deck segment 1, 8 hours for deck segment 2, and 48 hours for deck segment 3 (the negative moment region). The difference in time between these stages can impact the vertical displacements during the deck pour. Figure 6-39 shows the unsymmetrical displacements that result from the concrete deck placement when early stiffening is accounted for. The maximum displacement occurs in the first span and has a value of approximately 10 in. The deflection in the second span is only about 7 in. Recall that the displacements in both spans were the average of these values, about 8.5 in, for the monolithic pour (Figure 6-37).

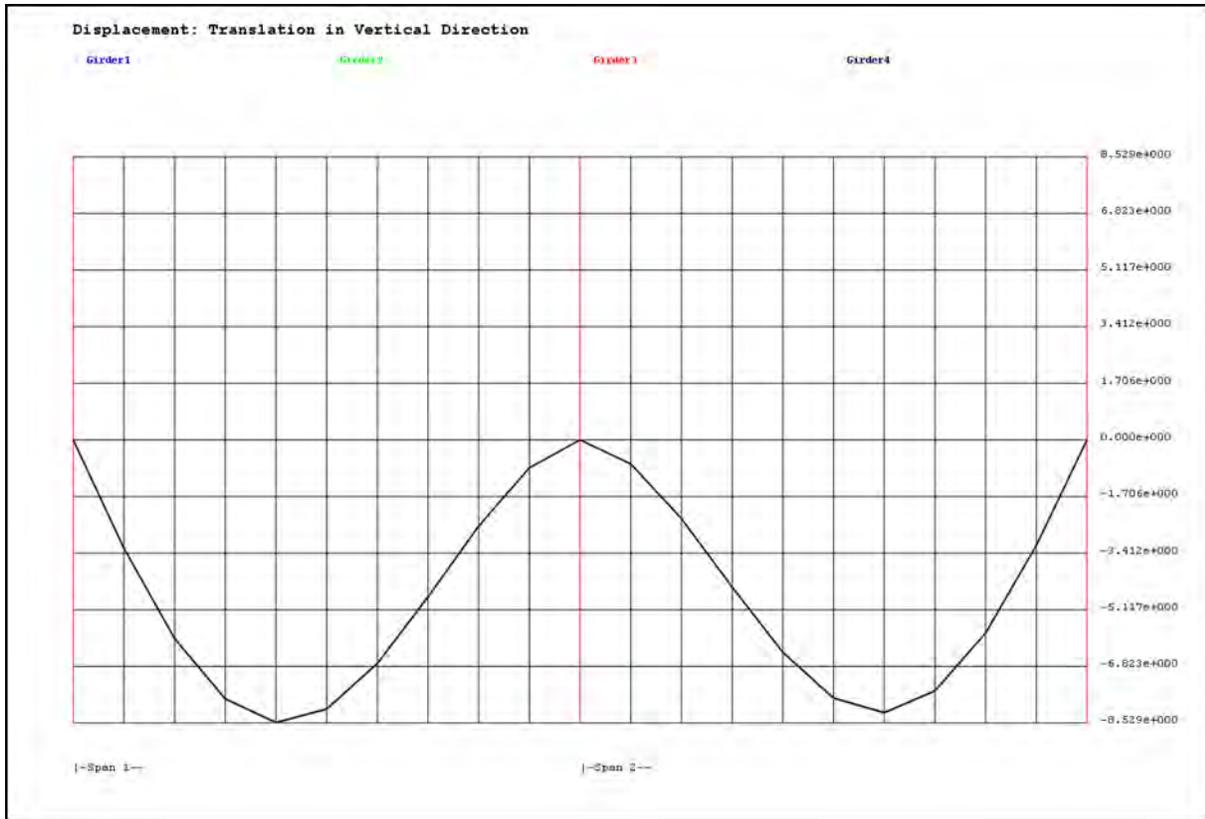


Figure 6-37 Estimated Dead Load Deflections from Uniform Load from Simulated Continuous Deck Placement

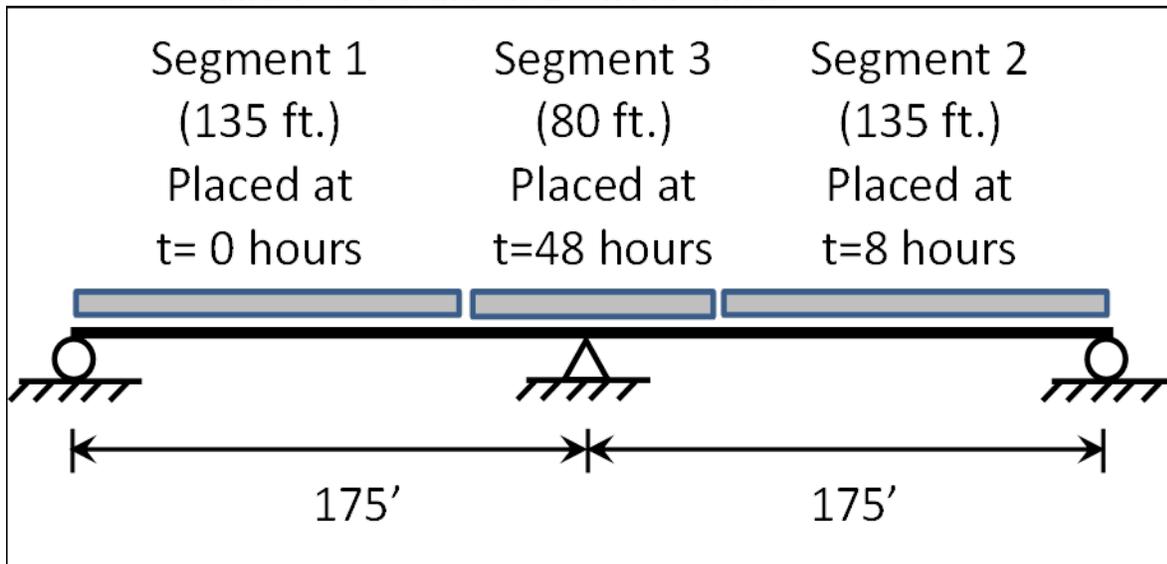


Figure 6-38 Concrete Pouring Sequence

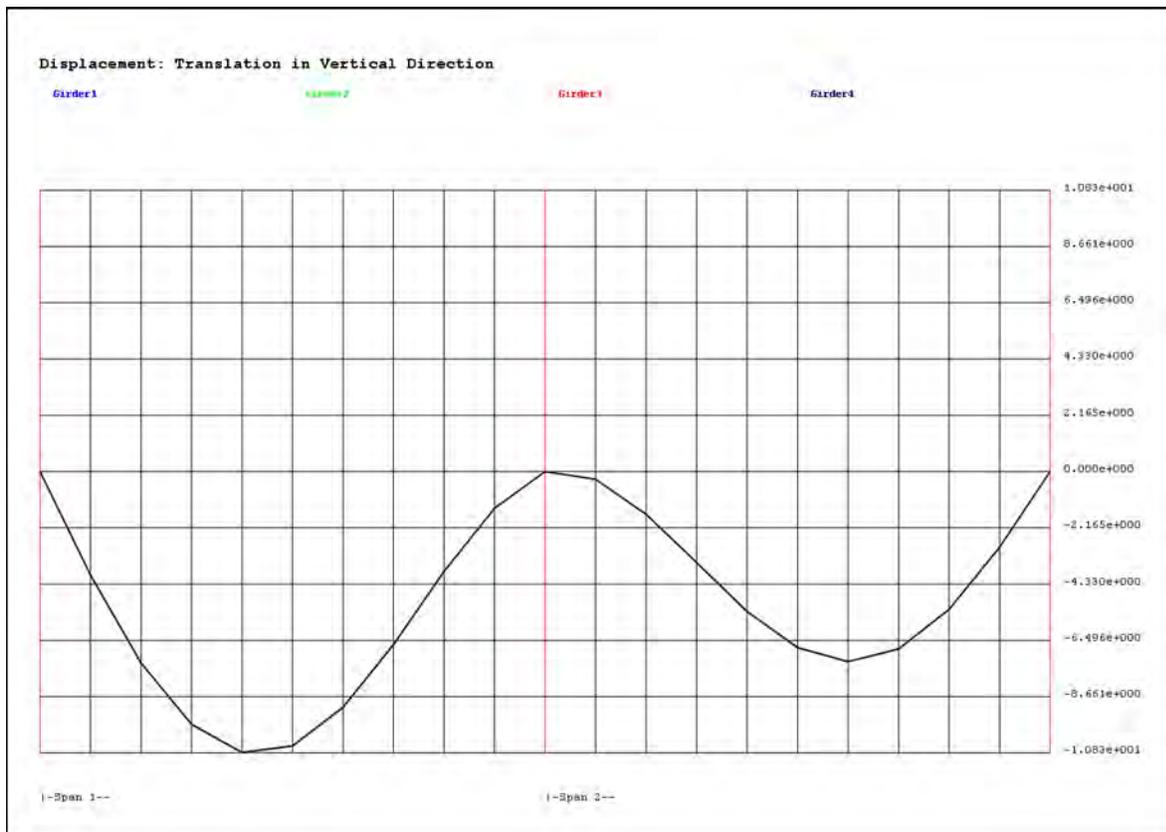


Figure 6-39 Differential Deflection due to Early Stiffening of the Concrete

SECTION 6. COMPARISONS BETWEEN AASHTO DESIGN EXPRESSIONS AND EIGENVALUE BUCKLING ESTIMATES

As noted earlier in the chapter, critical loads predicted from an eigenvalue buckling analysis will often have good correlation with estimates from closed-form equations that are included in design specifications such as AASHTO. However, simplifying assumptions have been made in some of the specification equations and the designer also must often make assumptions in using the expressions that can lead to differences between the buckling capacity predicted by the eigenvalue and specification expressions. This section provides a comparison between the eigenvalue buckling solutions and the AASHTO design expressions so that engineers can understand the source of the differences between these buckling estimates. For the purposes of comparison, variations in the geometries of a girder that is used in Example 1 of Appendix B are used to demonstrate potential differences in the buckling solutions. The eigenvalue solutions are obtained using the program BASP outlined earlier in the chapter. Comparisons are made with the buckling expressions included in AASHTO Section 10.6 and AASHTO Appendix A6.

6.6.1 Girder Geometry

Although the bridge geometry in Example 1 from Appendix B consists of a two-span bridge with six girders across the width of the bridge, comparisons in this section will focus on the stability of a single girder during the first lifting stage. The geometry of the Example 1 girder is shown in Figure 6-40 in which the girder consists of a non-prismatic section with larger flanges in the negative moment region compared to the positive moment region. In the positive moment region, a smaller top flange is used compared to the bottom flange, thereby creating a singly-symmetric section. In the positive moment region, the same size flanges are provided, resulting in a doubly-symmetric section. The flange transitions occur at the two splice points on the girder. During erection of the first segment, twist will often be provided by preventing lateral movement at the top and bottom of the girder as denoted by the “X” at these locations. A common erection scheme for the girder that would likely be considered would be to complete Splice 1 on the ground and lift a 212 ft long segment onto the supports with a 48 ft overhang to Splice 2. Because the girder is a non-prismatic section, an engineer desiring to use the AASHTO buckling expressions will need to make some simplifying assumptions. The most common approach to this problem is to treat the girder as a prismatic section with the properties of the positive moment region and an unbraced length of 164 ft, which is the total length of the first span.

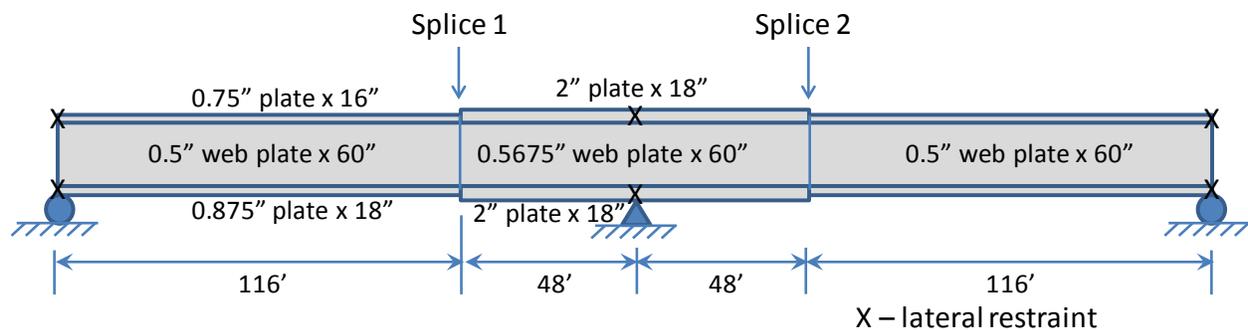


Figure 6-40: Geometry of Two-Span Non-Prismatic Girder

6.6.2 Comparisons Between AASHTO Equations and Eigenvalue Buckling Solutions

The AASHTO Specification has two different expressions that can be used to evaluate the lateral-torsional buckling capacity. AASHTO Section 6.10.8 has the following lateral-torsional buckling expression (Note R_b for the beam is 1.0 and is not shown in the equation):

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_t} \right)^2}$$

AASHTO Equation 6.10.8.2.3-8

This particular AASHTO expression neglects the St. Venant torsional stiffness and therefore only reflects the capacity based upon the warping torsional stiffness. Alternatively, the following AASHTO expression in Appendix A6 (Equation A6.3.3-8) can

be used to consider the contribution of both the warping and St. Venant torsional stiffness:

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_t}\right)^2} \sqrt{1 + 0.078 \frac{J}{S_{xc} h} \left(\frac{L_b}{r_t}\right)^2} \quad \text{AASHTO Equation A6.3.3-8}$$

The term inside the radical of the AASHTO Appendix A6 expression represents the St. Venant Stiffness of the section, while the term outside the radical is identical to the AASHTO Eq. 6.10.8.2.3-8. Since the actual plate sizes can be included in a finite element program, there will likely be additional differences between buckling estimates using the AASHTO expressions and the eigenvalue buckling solution. To demonstrate potential differences between eigenvalue solutions and the AASHTO expressions, the girder geometries shown in Figure 6-41 are considered. In Case 1, the girder is treated as a simply-supported prismatic girder with a total length of 164 ft. In Case 2, the girder is still treated as a prismatic section; however the 48 ft long overhang is considered. In Case 3, the non-prismatic girder with the overhang is considered.

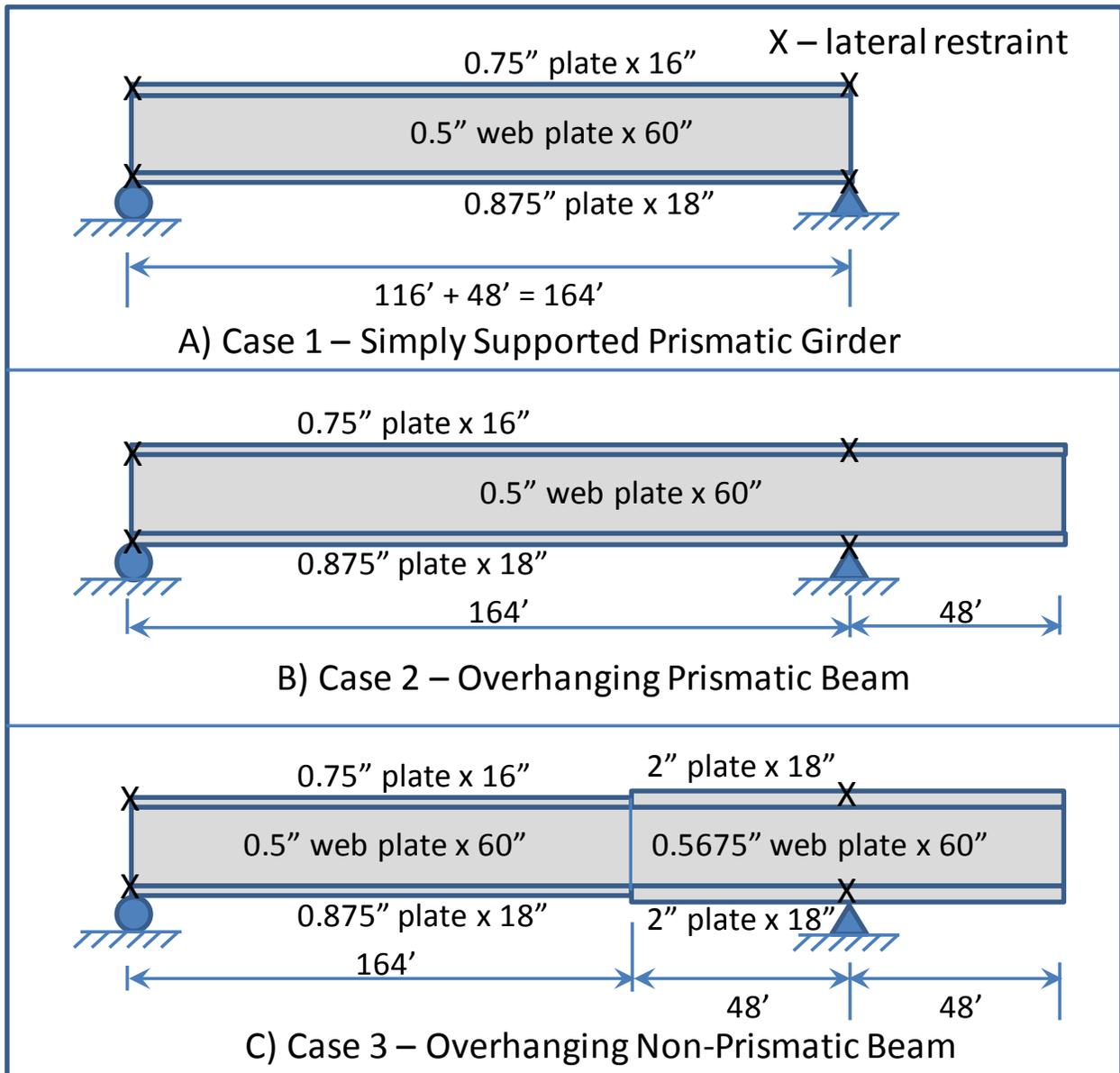


Figure 6-41: Cases Considered in Comparison of AASHTO Equations and Eigenvalue Buckling Solutions

The loading case that was considered consisted of uniform moment loading so that $C_b = 1.0$. For the purposes of comparison, the program BASP (Buckling Analysis of Stiffened Plates) was used with uniform moment loading to demonstrate the differences between the eigenvalue solution and the AASHTO expressions given above. Three different analyses were conducted in BASP to demonstrate the behavior. The results are summarized in the following table:

Analysis Case	Description	AASHTO Eq. 6.10.8.2.3-8	AASHTO Eq. A6.3.3-8	BASP (Eigenvalue)
1	No Overhang, Prismatic, Section	95.7 k-ft (49.6%)	185 k-ft (2.6%)	190 k-ft
2	Cantilever Overhang, Prismatic Section	N/A	N/A	208 k-ft
3	Cantilever Overhang Non-prismatic Section	N/A	N/A	318 k-ft

Table 6-3 Summary of analysis cases.

In analysis Case 1, the beam was modeled as a prismatic section with a 16" x 0.75" top flange, 18" x 0.875" bottom flange, and a 60" x 0.5" web. As shown in Figure 6.41, the beam in BASP for this case was modeled without the overhang beyond the interior support. This beam has the same section properties as utilized in the two AASHTO equations. The eigenvalue solution in this case has good agreement with the AASHTO Appendix A6 equation that includes both the St. Venant and warping stiffness. The percentages shown below the AASHTO expressions are the percent differences relative to the BASP Analytical Case 1 solution. The AASHTO Equation 6.10.8.2.3-8 that is frequently used in practice was nearly 50% conservative compared to the eigenvalue solution since the St. Venant stiffness is conservatively neglected in the AASHTO equation. Most designers would approximate the cross-section as shown in Case 1 for the geometries in Case 2 and 3, and therefore solutions are not given for these cases. In the Analysis Case 2, the portion of the erected beam that overhangs the interior support was included, which resulted in nearly a 10% increase to 208 k-ft in the eigenvalue buckling solution. The increase in the buckling capacity occurs because the overhanging beam provides some additional restraint at the support since the overhanging section does possess St. Venant stiffness. In the analysis Case 3, the much larger section in the negative moment region was included thereby forming a non-prismatic beam. The larger section in the negative moment region, increased the uniform moment buckling capacity to 318 k-ft

Another source of conservatism that will often exist in solutions using the expressions from AASHTO are the moment gradient factor. In many cases, engineers will use a $C_b = 1.0$ due to complexities associated with non-prismatic sections or moment distributions. The finite element solution will often reflect the benefits of moment gradient, which may often lead to additional capacity reflected in the eigenvalue solution. The factors outlined above will often lead to significant differences between hand calculations using the AASHTO equations and critical loads predicted from a finite element program. Such is the case as outlined later in Example 1 in Appendix B.

SECTION 7. SUMMARY

This chapter has provided a summary of several potential analytical methods for evaluating the stability of bridge systems during construction. In addition to the methods of analysis, several modeling considerations were outlined ranging from the necessary detail of the models, types of analyses and specific construction stages that might be critical. Simplified models consisting of line element representations of girders were discussed as well as detailed three-dimensional shell element models. The different stages that may control the stability of girders during erection were discussed including cases during lifting of individual girder segments, partially erected girder systems, and critical stages during concrete deck placement. The analytical procedures that were discussed in this chapter can be carried out on a variety of software packages that are commercially available as well as others that are in the public domain.

CHAPTER 7

DESIGN CRITERIA

SECTION 1. INTRODUCTION

At each stage of the erection of a girder bridge superstructure, both the member loads and the available resistances will differ as the configuration changes. Also, at each stage the member loads and available resistances will differ from those in the finished structure. For example, performance of an individual girder may be controlled by lifting conditions, or after placement of the girder but prior to its connection to the adjacent girders. Wind effects on an open girder grillage will also differ from those on the completed bridge. Criteria for the evaluation of girder-bridge superstructures during erection should take into account these varying conditions, and may differ from those provided for the finished bridge. In addition, criteria for evaluating members at some erection stages, i.e. lifting of girders, is not explicitly addressed in the AASHTO Bridge Design Specifications. The supplementary evaluation criteria presented in this chapter address conditions that affect the stability and performance of the girder-bridge superstructure during erection that are not directly addressed in the AASHTO Bridge Design Specifications. Bridge owners may also have provisions covering engineering analysis for construction conditions and required submittals. These account for owner practices, local load conditions and other factors, and must be adhered to in preparing erection analysis and procedures. Appendix D in this Manual provides recommended engineering criteria for girder-bridge superstructure evaluation during construction in a format suitable for possible inclusion in specifications that might be developed by the bridge owner.

The recommended engineering criteria are presented in an LFRD format to be consistent with the AASHTO LRFD Bridge Design Specifications (AASHTO Specifications). The applicable requirements provided in the AASHTO Specifications are not restated; however, relevant evaluation criteria and formulas not included in the AASHTO Specifications are presented herein. The recommended engineering criteria have been developed assuming that the design engineer and contractor have reasonable control over some erection activities, but perhaps less over others. For instance, an erection plan may limit lifting and setting of a girder to periods where the wind speeds are less than 20 mph. However, once the work for that shift ends, the wind speeds to which the partially erected girder is subjected and the scheduling of subsequent construction activity may be under the control of Mother Nature rather than the contractor.

The design of bridges is governed by the AASHTO Specifications, whereas the AASHTO LRFD Bridge Construction Specifications provides requirements pertaining to construction practices. Both Specifications reflect the directive made by the Federal Highway Administration (FHWA) that all bridges designed after 2007 use the load and resistance factor design, or LRFD, methodology. Various articles of these specifications

address matters related to the erection and performance of bridge superstructures during construction.

Article 2.5.3 of the AASHTO Specifications notes that constructability issues should include, but not be limited to, deflection and stability during critical stages of construction. It further requires that where a designer has assumed a particular sequence of construction to control dead load stresses, that the sequence be defined in the contract documents. For complex bridges, or bridges where an experienced contractor would not be expected to be able to predict and estimate a suitable erection procedure while bidding the project, at least one feasible method of construction should be included in the contract documents. Requirements for temporary bracing or support to execute the given erection plan should be included; however, this is generally limited to providing loads and recommended locations for the vertical shoring.

Article 3.4.2 of the AASHTO Specifications provides load factors for construction loads for use in evaluating structure strength and serviceability. The commentary defines construction loads as, “permanent loads and other loads that act on the structure only during construction”. The specified load factors, which are intended for use when investigating the applicable load combinations in Table 3.4.1-1 during construction, are in some cases reduced from those used for permanent loading cases.

The AASHTO “Guide Design Specifications for Bridge Temporary Works” includes provisions for bridge construction loads, deck construction, and design of falsework or shoring. This information should be referenced where appropriate in developing erection studies and temporary support design. It should be noted, however, that this document was developed primarily to address the design of falsework for cast-in-place concrete, and does not directly address girder erection issues.

Concrete girder design is covered under SECTION 5: *Concrete Structures* of the AASHTO Design Specifications. Specific guidance for construction conditions for girder bridges is limited. Article 5.5.4.3 – *Stability*, does state that “Buckling of precast members during handling, transportation, and erection should be investigated,” but the provision is silent regarding methods to perform the necessary investigations. In addition, Article 5.6.1, under Article 5.6 – *Design Considerations*, requires members and connections to be designed to accommodate construction loads. The AASHTO LRFD Bridge Construction Specifications provides requirements summarized as follows:

- 8.13.5 Storage and Handling: Requires precast girder support points to be within 2.5 ft. of the location shown in the shop drawings.
- 8.13.6 Erection: The contractor is responsible for safety of the precast members during all construction stages. Members should be temporarily braced as required to resist wind or other loads.
- 8.16.3.2 Design Calculations for Construction Procedures: Requires that calculations be prepared by a registered professional engineer for falsework, erection devices, or other temporary construction that may carry calculated stresses.

No specific guidance is provided as to what constitutes adequate methodology or submittal requirements to satisfy these responsibilities.

The design of steel structures, including steel girders, is covered in SECTION 6: *Steel Structures* of the AASHTO Specifications. The provisions of Article 6.10: *I-Section Flexural Members* address both straight and curved girders in a consistent manner, and include a unified approach for the investigation of the effects of combined major-axis bending and flange lateral bending stresses regardless of the source. Attention is called to the considerations of flange lateral bending stresses during construction for straight girders, as well as flange lateral bending stresses due to curvature in horizontally curved bridges, both during construction and in the final structure. Design equations that address the nominal flexural resistance of discretely braced compression flanges, a common case to be investigated during construction, are provided, which include provisions to account for the effects of flange lateral bending due to deck overhang bracket loads.

Articles 6.10.3 and 6.11.3 of the AASHTO Design Specifications for steel I-girder and box-girder members, respectively, present required design criteria for investigating the adequacy of the members during critical stages of construction. Criteria for deck placement effects are also provided. Article 6.10.3.1 requires stresses during erection to remain below nominal yield levels for all main load-carrying members, and reliance on post-buckling resistance is not permitted, except for potential local web yielding in hybrid sections.

Provisions for bracing design are limited. While bracing locations can be determined to control flange stresses using the flexure design equations, determining required strength and stiffness of the bracing members is addressed only through the requirement for a rational analysis. Design of individual bracing members is addressed in this Manual, and general guidance on bracing locations and configurations is noted in several articles.

One of the significant changes that was implemented in the AASHTO Design Specifications, which dates back to the First Edition (1994) of the specifications, was the removal of the maximum 25 foot cross-frame spacing limit for straight steel-girder bridges. The cross-frame spacing is now to be based on a “rational analysis”, and the tendency is to attempt to minimize the number of cross-frames in order to eliminate fatigue prone details and improve economy. Special maximum cross-frame spacing requirements remain for curved steel girders. The tendency to reduce the number of cross-frames makes proper design checks of the girders and the bracing at critical construction stages more important.

Section 11 - *Steel Structures* of the AASHTO LRFD Bridge Construction Specifications requires the contractor to provide calculations verifying erection methods that vary from the contract documents, and to be responsible for any temporary bracing or shoring. Additional requirements are provided for curved girders. The contractor should provide

a construction plan detailing fabrication, erection procedures, and deck placement for curved girder bridges based either on the plan shown in the contract documents or the contractor's own plan. The plan, in either event, should "demonstrate the general stability of the structure and individual components during each stage of construction" and be stamped by a registered Professional Engineer.

In addition to the AASHTO provisions, several other engineering specifications and industry publications provide guidance or design provisions that can be used for erection evaluation. The American Institute of Steel Construction (AISC) requirements for the design flexural strength of steel I-section members are covered in Chapter F of the Specification for Structural Steel Buildings (2010) and includes a design equation to determine when lateral-torsional buckling needs to be considered. The available lateral-torsional buckling capacity is determined based on the length of the compression flange between brace points, and a moment-gradient modification factor, C_b , to account for the beneficial effect of a nonuniform moment between brace locations. This is consistent with the AASHTO provisions, which should be used in the erection evaluation of steel-girder bridges in lieu of the AISC provisions. Once the location of brace points is determined to provide the required member strength, the bracing can be designed using the provisions of Appendix 6 of the AISC Specification. Appendix 6 provides equations to determine the required brace strength as well as required stiffness for both lateral bracing and torsional bracing. The brace strength and stiffness requirements are dependent upon the required flexural strength and the C_b factor (among other inputs). Equations are included for both the LRFD and Allowable Strength Design (ASD) load combinations. Once the required brace strength and stiffness is determined, member design is completed in the standard manner for the member type(s) selected. These equations are not currently provided in the AASHTO provisions, but are provided and discussed further herein.

The Precast Prestressed Concrete Institute, PCI, provides design, production, and installation guidance for a wide range of products, including building components, piling, and bridge members. The PCI Bridge Design Manual, Chapter 3 Fabrication and Construction, provides general information concerning member lifting, lifting loop installation, and girder setting. PCI also produces the "Erector's Manual: Standards and Guidelines for the Erection of Precast Products, MNL-127-99." Chapter 8, Design Theory and Procedure, of the PCI Bridge Design Manual provides methods to calculate lateral stability of precast members during transporting, lifting, and placement in Section 8.10, Lateral Stability of Slender Members.

While the American Concrete Institute's "Building Code Requirements for Structural Concrete and Commentary" (ACI 318) are not written to address concrete bridges, the provisions of Appendix D, Anchoring to Concrete, provide design information not in AASHTO that may be required in design of the connections for bracing or hold downs to concrete during erection. Chapter 10 – Flexure and Axial Loads, limits the unbraced length of a girder to 50 times the least width of the compression flange.

SECTION 2. STRUCTURE ANALYSIS

7.2.1 Member and Component Evaluation

Bridge girders must be designed to provide adequate strength and stability during all stages of erection. The loads to which each erection stage is subjected not only differ from those used for the design of the finished structure, but generally also vary with the particular erection stage. For many simple structural systems, the erector or their engineer can justify the erection procedures by judgment or documented experience on bridges of similar span, slenderness, lateral stiffness, and bracing configuration. The justification of these erection procedures can be made through hand-calculations, an eigenvalue buckling analysis, and comparisons with previous erection procedures on similar structural systems. However, in cases where significant uncertainty exists in the system stability as a result of procedures or geometrical conditions, a global stability analysis that considers the impact of deformations on the geometry of the system should be conducted to verify adequate stability for the erection scheme and appropriate load conditions. Girder design must be based on the most critical conditions, and these may not be the same for every girder in a system. Similarly, calculated loads used to size cranes, and design shoring systems and bracing must represent the most severe load conditions. In addition:

- All members should be shipped, lifted, supported, connected, and braced in such a way that no limit states are violated at any time and damage such as yielding, buckling and concrete cracking is avoided. Stability includes local, member, global and rigid body (rollover) stability.
- Analysis methods used should be sufficiently refined to evaluate the limit states of concern for each stage of girder erection. For setting of a single girder, a line girder analysis may be satisfactory; however, refined grid or 3D analysis methods are necessary for girder systems unless it is determined that girder interaction effects can safely be neglected.
- The boundary conditions assumed used in the analysis model should be representative of those specified in the erection plans, and provided in the field.
- The boundary conditions should recognize the absence of any vertical restraint in the investigation of uplift scenarios.
- Nominal yielding or reliance on post-buckling resistance should not be permitted for main steel load-carrying members for all construction conditions, except for potential local yielding of the web in hybrid sections.

7.2.2 Critical Erection Stages

The erection plan and supporting engineering calculations must address both strength and stability at each stage of erection. Deformations associated with each stage should also be evaluated. Critical erection stages for the girder bridge structure during construction normally consist of at least the following:

- Lifting of girders/members
- Placement of the initial girder and any associated temporary bracing used to hold the girder in place
- First pair of girders set with permanent bracing installed
- All girders and bracing installed prior to the deck placement
- All girders and bracing installed during the deck placement
- Application of the deck overhang bracket loads to the fascia girders during the deck placement

The last two stages shown above are evaluated by the Engineer of Record; however, if the deck placement sequence changes from that shown in the contract documents, these stages must be re-evaluated. Each of these conditions for the partially completed structure is accompanied by differing loading. Primary loads include structure and deck concrete self-weight (dead load), wind load, construction loads from formwork, materials, workers and concrete-placing equipment.

7.2.3 Analytical Modeling

The selection of an analytical model and analysis method must consider the type of information required in subsequent analyses of the structural components. To investigate a single girder placed into position, a line girder analysis may be readily used with the girder dead load acting vertically and wind acting laterally on the girder face. As additional girders are set, the load distribution becomes affected by the stiffness contributed to the system by diaphragms or cross-frames, the effects of which are not accounted for in a line-girder analysis. For curved or skewed bridges, these effects are of increased importance, though they can have effects even on parallel girder spans, particularly as skews increase (Fisher 2006).

The stability issues that often occur with concrete systems are primarily related to roll stability. Many solutions for evaluating roll stability are based upon simple free-body diagrams and are discussed in Sections 5-7 and 7-7.2.

A two-dimensional (2D) grid analysis (also referred to as a grillage analysis) is a finite element application that models the structure as a grid of elements and is often used in bridge design. The AASHTO Specification provides some guidance on the use of 2D grillage models in Chapter 4, *Structural Analysis and Evaluation*. Two-dimensional beam elements capture flexure, shear, and Saint Venant torsion effects, but do not normally model flange bending (warping) stresses, which are important for curved girder analyses. The cross-frames or diaphragms in 2D grid analyses are modeled as line elements, requiring the designer to input appropriate stiffness properties based on the diaphragm or cross-frame configuration. One approach is to model the equivalent flexural stiffness of the cross-frame separately, by applying a unit force couple to one end, calculating the resulting deflection, and computing the end rotation. This rotation is

then applied to a propped cantilever beam with length equal to the cross-frame width, so that the associated moment of inertia for the propped cantilever can be back calculated and used as the line element stiffness for the cross-frame.

Another approach is to compute the shear stiffness of the cross-frame due to a unit vertical deflection, and utilize this as the cross-frame stiffness for the line element. The cross-frame support conditions assumed in the above models can provide additional variations to the modeling, with each model providing different results. Both the flexural and shear stiffness of the brace influence a bridge's behavior. Differential girder deflections are predominately affected by cross-frame shear stiffness, while rotation of the girders is more likely to be affected by the flexural stiffness. Running several analyses using varying cross-frame stiffness values can provide a means to determine the structure's sensitivity to the stiffness variations and provide a range of values for design.

White et al. (2012) as part of their work under the National Cooperation Highway Research Project 725, provided recommendations for improved modeling techniques in 2D analysis. The use of a shear-deformable (Timoshenko) beam element improves the accuracy of the 2D-grid analysis method. This approach involves calculation of both an equivalent moment of inertia along with an equivalent shear area. Guidance on appropriate calculation methods is provided in the National Steel Bridge Alliance publication "G13.1 Guidelines for Steel Girder Bridge Analysis" (2014).

Three-dimensional finite element analyses (3D-FEM) use a computerized structural analysis model of the structure in three dimensions. Various software packages possess the ability to model the bridge components with a variety of different element types. The girder flanges and webs, bracing members, and deck can be modeled with a wide array of element configurations including line/beam elements, plate/shell elements, truss elements, solid elements, etc. as appropriate. Three-dimensional FEM is considered the most "precise" method for analysis; however, structure modeling and subsequent output analysis requires more time than with other analysis techniques. In addition, although these modeling concepts are perceived to be the "most accurate", the actual accuracy is contingent on a number of key factors. Users of 3D-FEM computer programs must understand the element formulations and results can be sensitive to proper modeling and input.

Advantages of the 3D-FEM include output that provides translation and rotation at each node in the structural model. This allows determination of member deflections and rotations both vertically and laterally at discrete points along the length of the girder flanges and webs. The three-dimensional model has the capability of modeling the flexibility in the web and flanges that can therefore capture local effects on the cross-section. The solution results from such a model have the ability to better predict potential erection stability issues or other significant problems compared to simple models. Bracing forces and deformations can be directly obtained from the analysis.

Analysis of models for erection typically do not consider composite action or live loads. As a result, the load modeling and the extraction of moments and shears is less complicated than that for “full design” in 3D.

Advances in computer capabilities and software development have made the use of 3D-FEM analysis more wide spread. The use of the software packages for the erection analysis of complex straight and curved girder bridges can avoid problematic situations that result in unsafe conditions or expensive field solutions.

7.2.3.1 Selection of Analysis Methods for Steel I-Girder Bridges

Part of the work completed under the National Cooperative Highway Research Project (NCHRP 725) (NCHRP, 2012), was a comparative study of the analysis results for curved, skewed, and curved bridges with skew obtained from different bridge structural analysis methods to those obtained from a detailed 3D finite element solution, deemed to provide the “correct” results. A review of a large data base of bridges resulted in an overview of the accuracy of the various methods of analysis. Based upon the results, limitations of the methods were identified and recommendations for the usage of simplified grid models were developed.

This information was then assembled into a matrix to aid in the selection of an appropriate approximate method of analysis of for I-girder bridges, and tub-girder bridges.

The results indicate that traditional 2D-grid and 1D-line girder analysis methods predict major-axis bending stresses and vertical deflections within 20% accuracy for I-girder bridges, except for girders with small radii of curvature. While both methods were able to compute cross-frame stresses and flange lateral bending stresses for curved girders within 7% - 20% accuracy, they were not able to determine these forces for skewed bridges. Complete details are available in the NCHRP 725 Report.

Figure 7-1 summarizes the results for the various methods and responses monitored for I-girder bridges. Figure 7-1 can be used to assess when a certain analysis method can be expected to give acceptable results. The grading rubric used to assign the letter “grades” to each method for the required analysis output data was as follows:

- A grade of ‘A’ is assigned when the normalized mean error is less than or equal to 6%, reflecting excellent accuracy of the analysis predictions.
- A grade of ‘B’ is assigned when the normalized mean error is between 7% and 12%, reflecting a case where the analysis predictions are in “reasonable agreement” with the benchmark analysis results.
- A grade of ‘C’ is assigned when the normalized mean error is between 13% and 20%, reflecting a case where the analysis predictions start to deviate “significantly” from the benchmark analysis results.

- A grade of 'D' is assigned when the normalized mean error is between 21% and 30%, indicating a case where the analysis predictions are poor, but may be considered acceptable in some cases.
- A score of 'F' is assigned if the normalized mean errors are above the 30% limit. At this level of deviation from the benchmark analysis results, the subject approximate analysis method is considered unreliable and inadequate for design.

Response	Geometry	Worst-Case Scores			Mode of Scores		
		Traditional 2D-Grid	1D-Line Girder	Improved 2D-Grid ^f	Traditional 2D-Grid	1D-Line Girder	Improved 2D-Grid ^f
Major-Axis Bending Stresses	C ($I_c \leq 1$)	B	B	A	A	B	A
	C ($I_c > 1$)	D	C	A	B	C	A
	S ($I_s < 0.30$)	B	B	A	A	A	A
	S ($0.30 \leq I_s < 0.65$)	B	C	A	B	B	A
	S ($I_s > 0.65$)	D	D	A	C	C	A
	C&S ($I_c > 0.5$ & $I_s > 0.1$)	D	F	A	B	C	A
Vertical Displacements	C ($I_c \leq 1$)	B	C	A	A	B	A
	C ($I_c > 1$)	F	D	A	F	C	A
	S ($I_s < 0.30$)	B	A	A	A	A	A
	S ($0.30 \leq I_s < 0.65$)	B	B	A	A	B	A
	S ($I_s > 0.65$)	D	D	A	C	C	A
	C&S ($I_c > 0.5$ & $I_s > 0.1$)	F	F	A	F	C	A
Cross-Frame Forces	C ($I_c \leq 1$)	C	C	B	B	B	A
	C ($I_c > 1$)	F	D	B	C	C	A
	S ($I_s < 0.30$)	NA ^a	NA ^a	B	NA ^a	NA ^a	A
	S ($0.30 \leq I_s < 0.65$)	F ^b	NA ^c	B	F ^b	NA ^c	A
	S ($I_s > 0.65$)	F ^b	NA ^c	B	F ^b	NA ^c	A
	C&S ($I_c > 0.5$ & $I_s > 0.1$)	F ^b	NA ^c	B	F ^b	NA ^c	A
Flange Lateral Bending Stresses	C ($I_c \leq 1$)	C	C	C	B	B	B
	C ($I_c > 1$)	F	D	C	C	C	B
	S ($I_s < 0.30$)	NA ^d	NA ^d	NA ^d	NA ^d	NA ^d	NA ^d
	S ($0.30 \leq I_s < 0.65$)	F ^b	NA ^e	C	F ^b	NA ^e	B
	S ($I_s > 0.65$)	F ^b	NA ^e	C	F ^b	NA ^e	B
	C&S ($I_c > 0.5$ & $I_s > 0.1$)	F ^b	NA ^e	C	F ^b	NA ^e	B
Girder Layover at Bearings	C ($I_c \leq 1$)	NA ^f	NA ^f	NA ^f	NA ^f	NA ^f	NA ^f
	C ($I_c > 1$)	NA ^f	NA ^f	NA ^f	NA ^f	NA ^f	NA ^f
	S ($I_s < 0.30$)	B	A	A	A	A	A
	S ($0.30 \leq I_s < 0.65$)	B	B	A	A	B	A
	S ($I_s \geq 0.65$)	D	D	B	C	C	A
	C&S ($I_c > 0.5$ & $I_s > 0.1$)	F	F	B	F	C	A

^aMagnitudes should be negligible for bridges that are properly designed & detailed. The cross-frame design is likely to be controlled by considerations other than gravity-load forces.

^bResults are highly inaccurate due to modeling deficiencies addressed in Ch. 6 of the NCHRP 12-79 Task 8 report. The improved 2D-grid method discussed in this Ch. 6 provides an accurate estimate of these forces.

^cLine-girder analysis provides no estimate of cross-frame forces associated with skew.

^dThe flange lateral bending stresses tend to be small. AASHTO Article C6.10.1 may be used as a conservative estimate of the flange lateral bending stresses due to skew.

^eLine-girder analysis provides no estimate of girder flange lateral bending stresses associated with skew.

^fMagnitudes should be negligible for bridges that are properly designed & detailed.

^gThe improved 2D-grid method requires the usage of an equivalent St. Venant torsion constant, which estimates the influence of the girder warping response on the torsional stiffness, as well as a Timoshenko beam cross-frame model that accounts for both the shear and bending flexibility of the cross-frames. See Articles 3.11 and 3.12 of this document for detailed discussions of these improvements. In addition, the improved 2D-grid method is limited to the analysis of systems with at least two girders connected by enough cross-frames such that I_c is less than or equal to 20.

Figure 7-1 Matrix for Recommended Level of Analysis – I-Girder Bridges
Information on the development of the grading system and the methodology of determining the normalized mean error can be found in the NCHRP 12-79 Report.

In Figure 7-1, scoring for the various measured responses is subdivided into six categories based on the bridge geometry. These categories are defined as follows:

- Curved bridges with no skew are identified in the Geometry column by the letter "C."

- The curved bridges are further divided into two sub-categories, based on the connectivity index, defined as:

$$I_c = \frac{15000}{R(n_{cf} + 1)m}$$

Equation 7-1

Where:

- R = the minimum radius of curvature at the centerline of the bridge cross-section throughout the length of the bridge (ft)
- n_{cf} = the number of intermediate cross-frames in the span
- m = a constant taken equal to 1 for simple-span bridges and 2 for continuous-span bridges

In bridges with multiple spans, I_c is taken as the largest value obtained from any of the spans.

- Straight skewed bridges are identified in the Geometry column by the letter “S.”
- The straight skewed bridges are further divided into three sub-categories, based on the skew index:

$$I_s = \frac{w_g \tan \theta}{L_s}$$

Equation 7-2

Where:

- w_g = the width of the bridge measured between fascia girders (ft)
- θ = the skew angle measured from a line perpendicular to the tangent of the bridge centerline (degrees)
- L_s = the span length at the bridge centerline (ft)

In bridges with unequal skew of their bearing lines, θ is taken as the angle of the bearing line with the largest skew. Bridges that are both curved and skewed are identified in the Geometry column by the letters “C&S.”

Two letter grades are indicated for each of the cells in Figure 7-1. The first grade corresponds to the worst-case results encountered for the bridges studied by NCHRP Project 12-79 within the specified category. The second grade indicates the mode of the letter grades for that category, i.e., the letter grade encountered most often for that category. It is useful to understand the qualifier indicated on the “C&S” bridges, i.e., “($I_c > 0.5$ & $I_s > 0.1$)” in Figure 7-1. If a bridge has an $I_c < 0.5$ and an $I_s > 0.1$, it can be considered as a straight-skewed bridge for the purposes of assessing the expected analysis accuracy. Furthermore, if a bridge has an $I_c > 0.5$ and an $I_s < 0.1$, it can be considered as a curved radially-supported bridge for these purposes.

7.2.3.2 Selection of Analysis Methods for Steel Tub-Girder Bridges

Similar to the matrix developed for I-girder bridges, NCHRP 725 developed a matrix to aid in the selection of an appropriate approximate method of analysis for tub-girder bridges, Figure 7-2.

Response	Geometry	Worst-Case Scores		Mode of Scores	
		Traditional 2D-Grid	1D-Line Girder	Traditional 2D-Grid	1D-Line Girder
Major-Axis Bending Stresses	S	B	B	A	B
	C	B	C	A	B
	C&S	B	C	B	B
Girder Torques	S	F	F	D	F
	C	D	D	A	B
	C&S	F	F	A	B
Vertical Displacements	S	B	B	A	A
	C	A	B	A	A
	C&S	B	B	A	A
Girder Layover at Bearing Lines	S	B	B	A	A
	C	NA ^a	NA ^a	NA ^a	NA ^a
	C&S	B	B	A	A

^a Magnitudes should be negligible where properly designed and detailed diaphragms or cross-frames are present.

Figure 7-2 Matrix 1 for Recommended Level of Analysis – Tub-Girder Bridges

Figure 7-2 scores for the major-axis bending stresses and vertical displacements are relatively good for all analysis methods. However, the worst-case scores for the internal torques are generally quite low. These low scores are largely due to the fact that the internal torques in tub-girder bridges can be sensitive to various details of the framing, such as the use and location of external intermediate cross-frames or diaphragms, the relative flexibility of these diaphragms as well as the adjacent internal cross-frames within the tub-girders, skewed interior piers without external cross-frames between the piers at the corresponding bearing line, incidental torques introduced into the girders due to the specific orientation of the top flange lateral bracing system members (particularly for Pratt-type TFLB systems), etc.

Similar to the considerations for I-girder bridges, the external diaphragms and/or cross-frames typically respond relatively rigidly in their own plane compared to the torsional stiffness of the girders (even though the torsional stiffness of tub-girders is generally

significantly large with respect to comparable I-girders). Therefore, the girder layovers at skewed bearing lines tend to be proportional to the major-axis bending rotation of the girders at these locations. As a result, the errors in the girder layover calculations obtained from the approximate methods tend to be similar to the errors in the major-axis bending displacements.

The connectivity index, I_C , does not apply to tub-girder bridges. This index is primarily a measure of the loss of accuracy in I-girder bridges due to the poor modeling of the I-girder torsion properties. For tub-girder bridges, the conventional Saint Venant torsion model generally works well as a characterization of the response of the pseudo-closed section tub-girders. Hence, I_C is not used for characterization of tub-girder bridges in Figure 7-2. Furthermore, there is only a weak correlation between the accuracy of the simplified analysis calculations and the skew index I_s for tub-girder bridges. Therefore, the skew index is not used to characterize tub-girder bridges in Figure 7-2 either. Important differences in the simplified analysis predictions do exist, however, as a function of whether the bridge is curved, "C," straight and skewed, "S," or curved and skewed "C&S." Therefore, these characterizations are shown in the table.

A discussion of the specific characteristics of the 2D – Program 1 computer program showing the tables can be found in the NCHRP 725 Report.

In addition to the above assessments, the accuracy of the bracing component force calculations in tub-girder bridges is assessed separately in Figure 7-3. It is useful to address the accuracy of these response calculations separately from the ones shown in Figure 7-2, since the simplified bracing component force calculations take the girder major-axis bending moments, torques, and applied transverse loads as inputs and then apply various useful mechanics of 51 material approximations to obtain the force estimates. That is, there are two distinct sources of error in the bracing component forces relative to the 3D FEA benchmark solutions:

7. The error in the calculation of the input quantities obtained from the 1D line-girder or 2D-grid analysis, and
8. The error introduced by approximations in the bracing component force equations.

Chapter 2 of the NCHRP 12-79 Task 8 report provides an overview of the bracing component force equations evaluated here, which are used frequently in current professional practice. It should be noted that the calculation of the top flange lateral bending stresses in tub-girders is included with the bracing component force calculations. This is because these stresses are influenced significantly by the interaction of the top flanges with the tub-girder bracing systems.

The NCHRP 725 research observed that in many situations, the bracing component force estimates are conservative relative to the 3D FEA benchmark solutions. Therefore, it is useful to consider a signed error measure for the bracing component force calculations. In addition, the bracing component dimensions and section sizes often are repeated to a substantial degree throughout a tub-girder bridge for the different types of components. Therefore, it is useful to quantify the analysis error as the

difference between the maximum of the component forces determined by the approximate analysis minus the corresponding estimate from the 3D FEA benchmark, i.e.:

$$e_{\max} = \left(R_{\text{approxmax}} - R_{\text{FEAmax}} \right) / R_{\text{FEAmax}} \quad \text{Equation 7-3}$$

for a given type of component. The grades for these responses were assigned based on the same scoring system used for the assessments based on the normalized mean error with one exception: Separate grades were assigned for the positive (conservative) errors and for the negative (unconservative) errors in Figure 7-3. In situations where no negative (unconservative) errors were observed in all of the bridges considered in a given category, the symbol "--" is shown in the cells of the matrix and the cells are unshaded.

The mode of the grades is shown only for the top flange diagonal bracing forces in Figure 7-3. The mode of the grades for the other component force types are not shown because of substantial positive and negative errors in the calculations that were encountered in general for the tub-girder bridges, and because, in cases where a clear mode for the grades existed, the mode of the grades was the same as the worst-case grade.

In addition to the above considerations, it should be noted that current simplified estimates of the tub-girder bridge bracing component forces are generally less accurate for bridges with Pratt-type top flange lateral bracing (TFLB) systems compared to Warren-and X-type systems. A small number of tub-girder bridges with Pratt-type TFLB systems were considered in the NCHRP 725 research. Therefore, the composite scores for these bridges are reported separately in Figure 7-3.

Response	Sign of Error	Geometry	Worst-Case Scores		Mode of Scores	
			2D-P1	1D-Line Girder	2D-P1	1D-Line Girder
TFLB Diagonal Force	Positive (Conservative)	S	D	D	D	C
		C	D	F	B	F
		C&S	D ^a	F	B	F
		Pratt TFLB System	C	F	A	F
	Negative (Unconservative)	S	F ^b	C		
		C	-- ^c	--		
		C&S	--	--		
		Pratt TFLB System	--	--		
TFLB & Top Internal CF Strut Force	Positive (Conservative)	S	C	C		
		C	F	F		
		C&S	F	F ^d		
		Pratt TFLB System	F	F		
	Negative (Unconservative)	S	C	C		
		C	--	A		
		C&S	--	C		
		Pratt TFLB System	D	D		
Internal CF Diagonal Force	Positive (Conservative)	S	NA ^e	NA ^e		
		C	F	F		
		C&S	F	F		
		Pratt TFLB System	--	F ^f		
	Negative (Unconservative)	S	NA ^e	NA ^e		
		C	--	--		
		C&S	--	D		
		Pratt TFLB System	B	--		
Top Flange Lateral Bending Stress (Warren TFLB Systems)	Positive (Conservative)	S	C	C		
		C	F	F		
		C&S	F	F ^d		
	Negative (Unconservative)	S	C	C		
		C	--	A		
		C&S	--	C		

^a Modified from a C to a D considering the grade for the C and the S bridges.

^b Large unconservative error obtained for bridge ETSSS2 due to complex framing. If this bridge is considered as an exceptional case, the next worst-case unconservative error is -15 % for NTSSS2 (grade = C).

^c The symbol "--" indicates that no cases were encountered with this score.

^d Modified from a B to an F considering the grade for the C bridges.

^e For straight-skewed bridges, the internal intermediate cross-frame diagonal forces tend to be negligible.

^f Modified from an A to an F considering the grade for the C and C&S bridges.

Figure 7-3 Matrix 2 for Recommended Level of Analysis – Tub-Girder Bridges

7.2.4 General Girder Proportions

7.2.4.1 Steel I-Girders

The flange width provides a significant contribution to the warping stiffness, and hence torsional stiffness, of I-girders, which is particularly important during construction of the bridge superstructure. Article 6.10.2.2 of the AASHTO Design Specifications, limits the flange width, b_f , for steel I-girders to a minimum of one-sixth of the girder depth, D ($b_f \geq D/6$). Owners may have more stringent requirements; for instance, the Texas Department of Transportation limits flange widths to $b_f \geq D/4$ for straight girders and $b_f \geq D/3$ for curved girders. Work by Stith, Petruzzi, et.al, 2010, showed that limiting the b_f/D ratio to a minimum of $1/4$ significantly reduces second-order effects in curved girders. For these cases, it was shown that cross-frame forces due to curvature could usually be well predicted by a first-order structural analysis.

A minimum flange width is generally sought by designers though for reasons of materials costs in the finished structure. Girder dimensions are determined in accordance with AASHTO and Owner requirements by the design Engineer of Record and are provided in the contract drawings. Thus, it is highly unlikely that any of these dimensions are subject to change based on the erection engineer's evaluation.

7.2.4.2 Steel Box-Girders

Article 6.11.2 of the AASHTO Design Specification, provides limits for web and flange proportions for steel box-girders. These limit the web thickness of unstiffened webs to the web depth divided by 150, and to the web depth divided by 300 if the web has longitudinal stiffeners. The requirements for top flanges of box sections are the same as for the top flange of I-sections:

$$b_f / 2t_f \leq 12$$

$$b_f \geq D/6$$

$$t_f \geq 1.1t_w$$

Where:

b_f = flange width (in)

t_f = flange thickness (in)

D = girder depth (in)

t_w = web thickness (in)

The National Steel Bridge Alliance, "Practical Steel Tub Girder Design" recommends that girder webs be at least 5 ft. high, with typical span-to-girder depth ratios between 25 to 35. The maximum width/thickness (b_f/t_f) ratio of bottom flanges in compression is 60, and longitudinal stiffeners are typically required when the b_f/t_f ratio exceeds 45. A minimum bottom flange thickness of 0.5 in. is recommended for tension zones, with minimum b_f/t_f ratios in the range of 80 to 120. The Texas Department of Transportation,

as an example, limits the b_f/t_f for the bottom flange to 80, with a minimum plate thickness of 0.75 in.

7.2.4.3 Steel I-Girder Erection Conditions

The use of the ratio of the girder length to flange width (L/b_f) to judge stability of a steel girder during erection has a long and generally successful history. The limit of L/b_f less than 85 appears in U.S. Steel erection manuals for straight girders. While erectors use various limits, they generally range as follows (Zhao, Yu, Burdette, 2010):

- Less than 60, girder will be stable
- 60-80, girder should be checked by calculation
- Over 80, temporary shoring will be required
- Use one-half of limits for cantilevers

Based on the work of Zhao, et. al (2009), when the L/b_f limits for cantilever beams are below 30, stability will be satisfactory, while values over 60 require temporary support. For values between these limits, calculations should be performed. Similar values for simply supported beams were 45 and 85. Clearly, these are large ranges, and such rules-of-thumb should be used only as rough guidelines.

7.2.5 General Concrete Girder Proportions

Article 5.14.1.2 of the AASHTO Design Specifications provides minimum thicknesses for precast concrete beams as follows:

Top flange	2 inches	
Web	5 inches	
Web (post tensioned)		6.5 inches
Bottom flange	5 inches	

The bottom flange minimum thickness primarily relates to box-type sections. Actual dimensions are established by flexural design efficiency and durability considerations. A review of standard AASHTO girder sections indicates that actual thicknesses, particularly for flanges, exceed the minimum. As noted previously, the resulting girder cross-sections normally possess large torsional stiffness such that roll stability, rather than lateral-torsional buckling, governs erection stability design.

SECTION 3. LOAD COMBINATIONS AND LOAD FACTORS

Article 3.4.2.1 of the AASHTO Design Specifications provides load factors and combinations for strength load combinations to be used for construction loading cases. The totaled factored force effect at each stage of construction should be taken as:

$$Q = \sum \gamma_i Q_i$$

Equation 7-4

Where:

Q = Total factored force effect

γ_i = Load factors specified in Table 7-5

Q_i = Force effects from loads during construction specified herein

For the cases of construction dead load, live load and wind load, the AASHTO load factors provided in Article 3.4.2.1 are similar to those presented in ASCE 37-0 2 for structures under construction. Recommended load factors and combinations for evaluating structures during erection, based on the provisions of Article 3.4.2.1, are given in Table 7-1.

	DC	C_{DL}	C_{LL}^(d)	C_w
Strength I	1.25	1.50	1.50	—
Strength III	1.25	1.25	—	1.0 ^(b)
Strength VI ^(a)	1.40	1.40	1.40	—
Service	1.00	1.00	1.00	0.7 ^(b)
Uplift ^(c)	0.90/1.35	0.90/1.35	—	1.0 ^(b)

- a) Steel structures only for the case of placing deck on completed steel. Use Strength I or III for intermediate steel conditions.
- b) The specified load factor of 1.0 for wind load force effects is based on basic wind speed maps wherein the load factor of 1.4 is built into the maps to increase the applied wind speed to the acceptable risk recurrence level. Wind loads may be computed using a wind velocity based on the Wind Velocity Modification Factor specified in Table 7-2 for the construction phase duration under investigation. Note that the wind load factor is less than 1.0 for the service load combination.
- c) Where a construction load produces uplift at the location being investigated, the maximum load factor should be applied to the load effect and where the load resists uplift, the load effect should be multiplied by the smaller load factor.
- d) Include dynamic load allowance if applicable.

Table 7-4 Load Factors and Load Combinations

The Strength VI load case is only used to verify the strength and stability of the completed steel superstructure under the loads from deck placement. Due to normal construction practices and common limits on wind during deck concrete-placement due to drying shrinkage concerns, wind load is not included in Strength VI. The intermediate stages of steel erection may be evaluated from the Strength I or Strength III load combinations. The use of the 1.25 load factor on DC in these load combinations recognizes that steel self-weight, taken from shop drawings, is known with a high

degree of accuracy, while the 1.4 load factor in Strength VI accounts for variability in how the deck concrete is placed (i.e., mounding of concrete, vertical drop dynamic effects, etc.).

The resistance, or phi (ϕ), factors utilized for the evaluation should conform to the AASHTO Design Specifications for steel and concrete as appropriate. Additional resistance factors for equations presented in this Manual are as follows:

ϕ_b , resistance factor for lateral-torsional buckling = 0.90

ϕ_{br} , resistance factor for steel girder bracing stiffness = 0.75

ϕ_{bk} , resistance factor for girder system buckling = 0.90

Safety factors on lifting accessories, jacks, and other manufactured items shall be in accordance with manufacturer's recommendations.

SECTION 4. LOADS

7.4.1 Material Weights

In the absence of more precise information, material unit weights used in the computation of permanent dead loads may be taken from Table 3.5.1-1 of the AASHTO Design Specifications. The weight of steel members should be computed using a unit weight for steel of 490 pounds per cubic foot. The weight of cast-in-place concrete should be computed based on an in-place unit weight of 150 pounds per cubic foot (pcf) for normal weight concrete and 125 pcf for structural lightweight concrete, unless mix-specific data is available. The above unit weights for concrete include the weight of prestressing steel and reinforcing steel, assumed as 5 pcf. For high strength concretes or heavily reinforced members, member weight calculation in accordance with AASHTO Table 3.5.1-1 is recommended.

7.4.2 Permanent Dead Load (DC)

Permanent dead load includes the weight of the partially completed structure, including the weight of the deck concrete during placement and any stay-in-place forms. Dead loads of the girders, framing, and deck, should be computed from design plans or shop fabrication drawings (when available). As a result, the dead loads and their distributions along the span may be expected to be accurate. Material unit weights used in computing dead loads should be in accordance with AASHTO Design Specifications, as discussed above. The weight of stiffeners, splices, studs, bolts, paint and other miscellaneous items may be taken as an equivalent uniformly distributed weight along the girder, or accounted for using an increased material density. However, when large field splice components are present at the end of cantilever segments during erection,

including the concentrated load effect may be warranted. The weight of stay-in-place corrugated metal deck formwork may be taken as 20 pounds per square foot, including the weight of concrete in the flutes. Where stay-in-place precast concrete deck forms are used, the weight should be computed from formwork drawings.

7.4.3 Construction Dead Load (C_{DL})

Construction dead load should include the weight of removable formwork and any shielding or work platforms that are only on the bridge during construction. The weight of formwork is normally taken at 10 pounds per square foot (psf) for wood or metal deck forms to account for material weight plus miscellaneous fillers, connectors, etc. Cantilever formwork, including walkway and railing, may weigh in the 15 to 20 psf range. A project specific value may be calculated as well.

7.4.3.1 Construction Live Load (C_{LL})

Construction live loads include the weight of workers, miscellaneous tools and supplies, materials and equipment that are only on the bridge during construction. A minimum uniform live load of 20 psf should be applied to the deck area to account for workers and miscellaneous light tools, etc. A 75 pound per linear foot (plf) live load should be applied along the outside edge of bridge deck overhangs. The construction live load should also include the actual weight of any equipment or materials loads, applied as a concentrated or locally uniform load to the appropriate location on the deck.

Equipment reactions should include the full weight of any equipment based on its maximum operating weight unless specific means are in place to ensure a reduced loading. Loads should be calculated based on data obtained from the equipment manufacturer or supplier for the specific equipment to be utilized.

Equipment, other than concrete-placing/finishing equipment, commonly termed a Bidwell[®] or Gomaco[®] machine, is not normally present on new bridge decks during construction, however in some cases other equipment may be present. When concrete is placed using motorized buggies, an additional 25 psf construction uniform live load should be applied to account for their use. If concrete is to be placed using a conveyor system as shown in Figure 7-4 the support reactions should be treated as equipment loads. In Figure 7-4, the conveyors for concrete delivery are shown on each side of the deck.

Jacks or jacking systems may be used during erection operations to allow adjustments in elevation, or to shift members laterally. Jacking force effects should be based upon the maximum jack capacities unless positive means are provided to limit the applied jacking forces. Safety factors on jack capacity should follow manufacturer's recommendations.

When bridges are being demolished or rehabilitated, loads from construction equipment may be present on portions of a bridge adjacent to areas of removed deck or in other locations that can produce adverse effects on member stability. In these cases, the actual equipment loads and their locations, including moving load effects, should be used in evaluating the adequacy of the structure at the various stages of construction.



Figure 7-4 Concrete-placing and Finishing Equipment on Bridge Deck

7.4.3.2 Dynamic Load Allowance

Dynamic load allowance, or impact, should be applied to equipment loads to account for effects of equipment moving on the structure as well as operating effects when stationary. The dynamic load allowance for operating equipment should be as recommended by the equipment manufacturer; however, a minimum dynamic load allowance of 0.10 of the equipment weight for equipment that can move on the structure, and 0.10 of the operating load for equipment when stationary should be used. Since equipment moving atop the structure does so at a very slow speed, a dynamic load allowance less than used for normal bridge traffic can be appropriate. Dynamic load allowance is typically not applied to the loads from the deck concrete-placing/finishing machine.

A dynamic load allowance is also not normally applied to a crane's lifted load. This is consistent with crane design and reflects the controlled procedures for lifting as well as the crane operational characteristics. When crane lifts involve removal of existing members, the calculated lifted load should include the member weight and rigging plus the addition of a minimum dynamic load allowance equal to twenty percent of the calculated member weight. This is added to account for lifting resistance due to interaction with other members, miscellaneous debris on the member, etc.

In investigating construction load effects for concrete segmental bridges, Article 5.14.2.3 of the AASHTO Design Specifications specifies a dynamic load allowance to account for the effect of accidental release of a precast segment or "sudden application of an otherwise static load..." The dynamic load allowance is specified as 100 percent of the segment dead load, to be added to the dead load of the segment. While this load condition is for segmental construction, similar dynamic effects might be applicable to specialized girder bridge lifting configurations using cables or winches or similar arrangements.

During bridge demolition or reconstruction, heavy equipment such as hydraulic concrete breakers, end loaders, trucks, etc. may operate from portions of the structure. In these cases, the dynamic load effects should be as recommended by the equipment suppliers, but not be less than the vehicle's dynamic load allowance computed from the AASHTO Specifications.

7.4.3.3 Incidental Loads

During superstructure erection, incidental loads may affect the stability of the members. As an example, a crane hook or lifted load might bump a member previously placed, or equipment operating beneath previously placed girders may accidentally strike a member. While these loads are not definable, consideration should be given to such accidental occurrences. It is recommended that bridge girders be designed to resist a minimum lateral load, (5 psf is suggested herein), to account for these possibilities. Some owners may require design for higher minimum lateral loads. This load should be applied to the vertical face area of the girder at the centroid of the loaded area.

7.4.4 Wind Load (C_w)

Article 3.8.1 of the AASHTO Design Specifications provides wind loads applicable to the design of the finished bridge structure. The minimum total wind load on the windward beam or girder is given as 300 plf of span and is applied to the girders with the deck in place. However, until the deck is placed, the girders form an open system that may behave differently under wind load than the finished structure.

Though not stated explicitly, it may be inferred that the specified load on the permanent structure is based on a wind speed recurrence interval of 50 years, or longer. Since the construction duration for a bridge superstructure is much less than this, it is reasonable

to base the construction period wind speed on some lesser recurrence interval, resulting in a reduced design wind speed.

During the actual lifting and setting of girders and other members, wind speeds will seldom exceed 20 mph due to girder positioning and fit-up difficulties as well as safety considerations. Permissible crane operating loads are typically reduced for wind speeds exceeding 15 to 20 mph. Manitowoc® Crane Manufacture's lifting charts for their crane models 888 and 2250, as examples, prohibit even reduced operations for wind speeds above 35 mph. Based on these considerations, a one-day duration wind speed of 20 mph can be used to assess girder stability up to the time that construction operations cease for the shift or the day, provided that the wind velocity limits are included as part of the erection procedures.

When investigating the girders in their final erected condition for the loads due to deck placement, the wind load may be neglected. While restrictions on equipment operation and worker safety would preclude placement in winds exceeding perhaps 20 to 25 miles per hour, restrictions on wind speed to preclude rapid water loss from the concrete deck (associated with deck cracking) limit winds to much less than even these values. The strength gain of the deck concrete is such that even if stronger winds should occur only a few hours after deck casting, the deck will have gained adequate strength to provide lateral bracing to the girders.

Where reduced wind speeds are used in design, provisions must be made to address any unanticipated high wind events due to sudden storms. These provisions need to recognize that the erection contractor may only have control of the project site for limited time periods, and hence coordination with the general contractor is necessary to assure that someone will be available at all times should supplementary bracing be required on short notice. The erection plans must clearly state the design wind loads associated with each stage of erection and provide provisions for any supplementary bracing.

Wind speed maps used by ASCE 7-10 *Minimum Design Loads for Buildings and Other Structures* to compute wind pressures are based on a 3-second gust wind speed measured at 33 feet above grade. When comparing wind speeds from weather forecasts to those used for design, the sampling periods used in the predictions must be known so the forecast, or reported, wind speed can be adjusted to be consistent with the design basis wind. ASCE 37-02 *Design Loads on Structures during Construction*, notes that to convert fastest-mile or 1-minute average wind speeds to 3-second gust speeds, the speed must be multiplied by 1.25, and to convert mean-hourly speeds to 3-second gust speeds, the speed must be multiplied by 1.55.

It is recommended that wind pressures and loads for superstructure erection stages be calculated in accordance with the provisions of Chapter 29, Wind Loads on other Structures and Building Appurtenances MWFRS of ASCE 7-10 *Minimum Design Loads for Building and Other Structures*, as modified herein. The wind velocity for the required Risk Category should be used along with the appropriate Velocity Modification Factor

from Table 7-4 to calculate the design velocity pressure. The Velocity Modification Factors in Table 7-4 are similar to those contained in ASCE 37-02, but have been computed to account for the changes in the wind provisions of ASCE 7-10. The recalculation reflects the fact that the wind velocity maps of ASCE 7-10 now directly include the Importance Factor and Wind Load Factor, which previously were separately applied to the wind velocity obtained from the wind maps.

Construction Duration	Velocity Modification Factor (V_m)
0 – 6 weeks	0.65
6 weeks – 1 year	0.75
1 year – 2 years	0.80
2 years – 5 years	0.85

Table 7-5 Wind Velocity Modification Factors

Wind velocity should be adjusted for height effects based on the top of bridge elevation, including superelevation, above grade. Table 7-3 provides height effect adjustments to the velocity pressure for various Exposure Categories.

Based on Table 1.5-1, "Risk Category of Buildings and Other Structures for Flood, Winds, Snow, Earthquake, and Ice Loads," of ASCE 7-10, a Risk Category of II should be used for bridge structures during erection, unless otherwise required by the bridge Owner. Circumstances for which the engineer or owner may consider a higher risk category include construction immediately adjacent to essential facilities in which an erection failure could imperil facility operation or occupant safety. For a Risk Category other than II, the appropriate wind speed maps of ASCE 7-10 should be used, in lieu of Figures 7-5 and 7-6.

The design wind load, C_w , should be determined from the following formula:

$$C_w = q_z G C_f A_f \quad \text{Equation 7-5}$$

Where:

q_z = Velocity pressure at height z above grade (psf) (see Equation 7- 7-6)

G = gust effect factor, use 0.85

C_f = net force coefficient (Tables 7-4, 7-5)

A_f = exposed projected area of girder or truss (ft^2)

The effects of superelevation and horizontal curvature are to be considered in determining the exposed projected area. Velocity pressure, q_z , in psf evaluated at height 'z' should be calculated from the following equation:

$$q_z = 0.00256 K_z K_{zt} K_d (V_m V)^2 \quad \text{Equation 7-6}$$

Where:

K_z = velocity pressure exposure coefficient (from Table 7-5)

K_{zt} = topographic factor, use 1.0

K_d = wind directionality factor, use 0.85

V = basic wind speed (Figures 7-5, 7-6) (mph)

V_m = wind velocity modification factor (Table 7-2)

z = height of top of bridge deck above grade / water (ft)

Basic wind speed, V , should be taken from Figure 7-5 and Figure 7-6, Basic Wind Speeds, which give Risk Category II wind speeds, and then be adjusted for the construction duration period in accordance with the Wind Velocity Modification Factors from Table 7-2. The wind velocity for a single work shift, or multiple shifts not exceeding one day should not be taken as less than 20 mph, except for deck placement where wind may be neglected.

In determining the design wind speed and selecting a velocity modification factor, both local site conditions and seasonal weather variations should be considered. For instance, when it can be ensured that erection will not take place during hurricane season, a basic wind speed reflecting this condition may be considered. Local conditions, perhaps erection within a canyon, where high winds might develop rapidly, must also be considered based on local experience.

The velocity pressure exposure coefficient, K_z , is a function of the ground surface roughness and height above grade and should be determined from Table 7-5. For each wind direction considered, the upwind exposure should be based on ground surface roughness that is determined from natural topography, vegetation, and constructed facilities. For bridge girder evaluation during erection, only the wind direction perpendicular to the girder is normally evaluated. A ground surface roughness category should be determined for a distance upwind of the site, and should be used to establish the exposure category.

Surface Roughnesses are defined as follows:

Surface Roughness B: Urban and suburban areas, wooded areas, or other terrain with numerous closely spaced obstructions having the size of single-family dwellings or larger.

Surface Roughness C: Open terrain with scattered obstructions having heights generally less than 30 ft. This category includes flat open country and grasslands.

Surface Roughness D: Flat, unobstructed areas and water surfaces. This category includes smooth mud flats, salt flats, and unbroken ice.

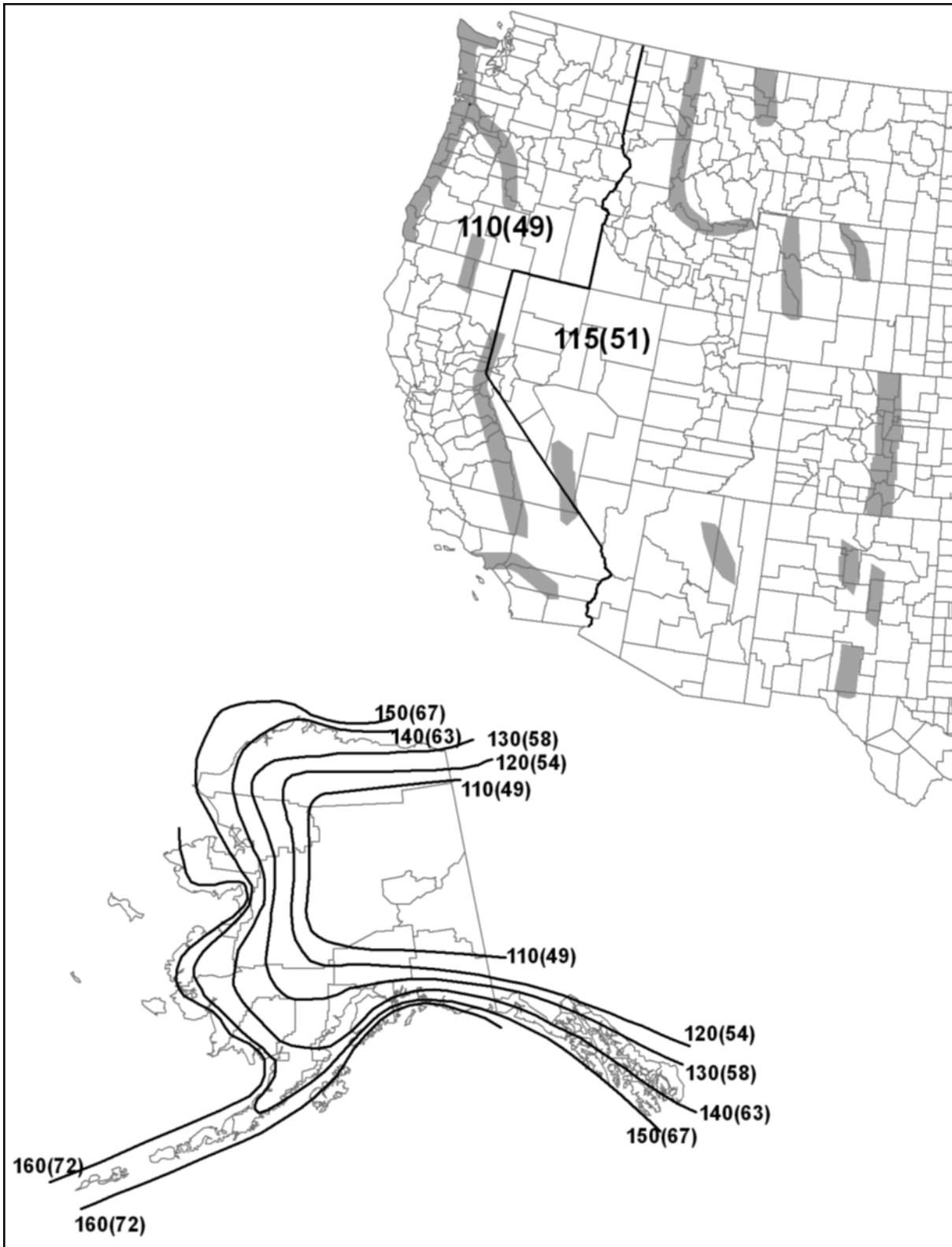


Figure 7-5 Basic Wind Speeds (with permission from ASCE)

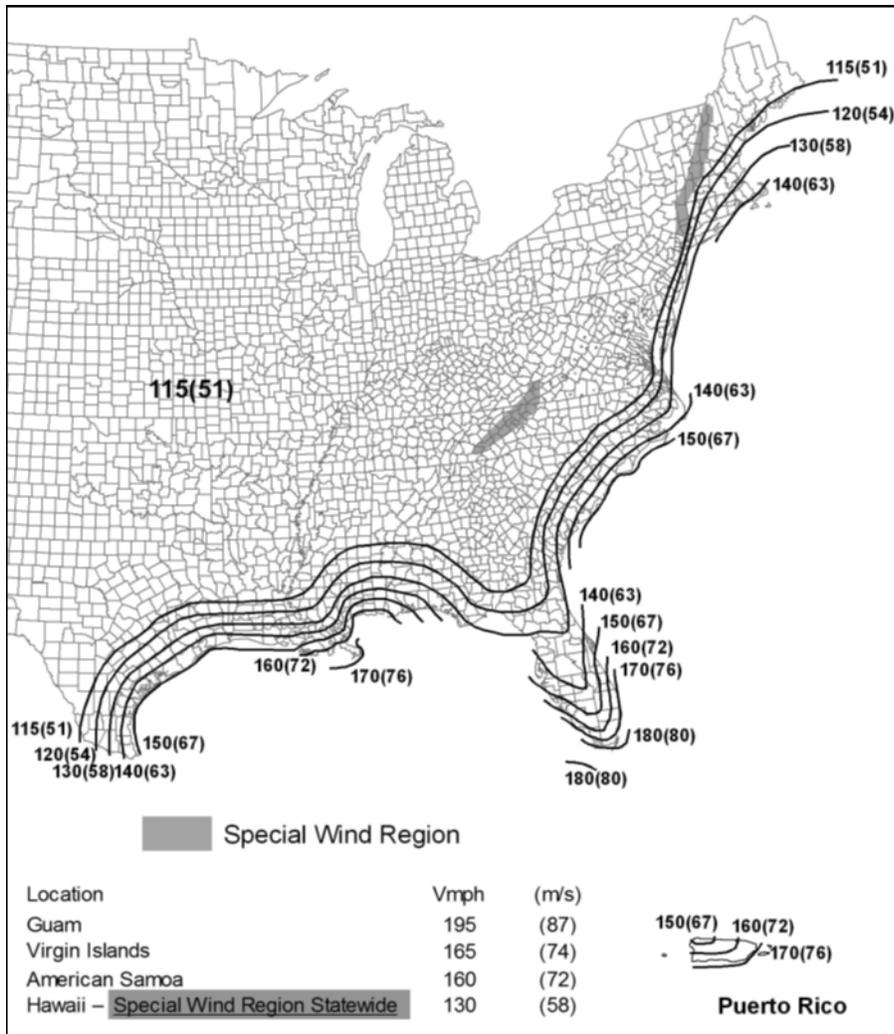


Figure 7-6 Basic Wind Speeds (cont.) (with permission from ASCE)

Notes:

1. Values are nominal design 3-second gust wind speeds in miles per hour (mph) at 33 ft. (10 m) above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour should use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions should be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (Annual Exceedance Probability = 0.00143, MRI = 700 Years).

Exposure Categories should be determined as follows:

Exposure B: For bridges less than or equal to 30 feet high, Exposure B should apply where the ground surface roughness, as defined by Surface Roughness B, prevails in the upwind direction for a distance greater than 1,000 feet. For bridges with a mean height greater than 30 feet, Exposure B should apply where Surface Roughness B prevails in the upwind direction for a distance greater than 2,600 feet or 20 times the height of the bridge, whichever is greater. Figure 7-7 shows an example of Exposure B with low rise structures in a suburban setting. The buildings are generally less than 30 feet high with Surface Roughness Category B terrain around the site for a distance greater than 1500 feet in any wind direction. Figure 7-8 shows an example of Exposure B in an urban setting where terrain representative of Surface Roughness Category B extends more than 20 times the height of the structure or 2600 feet whichever is greater, in the upwind direction.

Exposure C: Exposure C should apply for all case where Exposure B or D does not apply. Figure 7-9 shows an example of Exposure C with open grassland and scattered obstructions generally less than 30 feet high. Figure 7-10 shows an example of Exposure C with open terrain and scattered obstructions generally less than 30 feet high for most wind directions. The structures are less than 1500 feet or 10 times the height of the structure, whichever is greater, from an open field. This prevents the use of Exposure B.

Exposure D: Exposure D should apply where the ground surface roughness, as defined by Surface Roughness D, prevails in the upwind direction for a distance greater than 5,000 feet or 20 times the bridge height, whichever is greater. Exposure D should also apply where the ground surface roughness immediately upwind of the site is B or C, and the site is within 600 feet or 20 times the bridge height, whichever is greater, from an Exposure D condition as defined in the previous sentence. Figure 7-11 shows an example of Exposure D. The wind is flowing over open water for at least one mile. Shorelines with Exposure D include inland waterways, The Great Lakes, and coastal areas of California, Oregon, Washington and Alaska.

For a site located in the transition zone between exposure categories, the category resulting in the largest wind forces should be used.



Figure 7-7 Exposure B - Suburban Residential Area with Mostly Single Family Dwellings (with permission from ASCE)



Figure 7-8 Exposure B - Urban Area with Numerous Closely Spaced Obstructions With Size of Single Family Dwellings or Larger (with permission from ASCE)



**Figure 7-9 Exposure C - Flat Open Grassland with Scattered Obstructions
(with permission from ASCE)**



Figure 7-10 Exposure C - Open Terrain with Scattered Obstructions (with permission from ASCE)



Figure 7-11 Exposure D - A Building at the Shoreline (with permission from ASCE)

Height above ground level, z		Exposure Category		
ft	(m)	B	C	D
0-15	(0-4.6)	0.57	0.85	1.03
20	(6.1)	0.62	0.90	1.08
25	(7.6)	0.66	0.94	1.12
30	(9.1)	0.70	0.98	1.16
40	(12.2)	0.76	1.04	1.22
50	(15.2)	0.81	1.09	1.27
60	(18)	0.85	1.13	1.31
70	(21.3)	0.89	1.17	1.34
80	(24.4)	0.93	1.21	1.38
90	(27.4)	0.96	1.24	1.40
100	(30.5)	0.99	1.26	1.43
120	(36.6)	1.04	1.31	1.48
140	(42.7)	1.09	1.36	1.52
160	(48.8)	1.13	1.39	1.55
180	(54.9)	1.17	1.43	1.58
200	(61.0)	1.20	1.46	1.61
250	(76.2)	1.28	1.53	1.68
300	(91.4)	1.35	1.59	1.73
350	(106.7)	1.41	1.64	1.78
400	(121.9)	1.47	1.69	1.82
450	(137.2)	1.52	1.73	1.86
500	(152.4)	1.56	1.77	1.89

Table 7-6 Velocity Pressure Exposure Coefficient K_z

Notes: Linear interpolation for intermediate values of height z is acceptable.

The wind net force coefficient, C_f for girder bridges should be taken from Figure 7-4, while C_f for truss bridges should be taken from Figure 7-5.

As noted above, during erection, bridge girders form an open system of members exposed to the wind. Depending upon girder depth and spacing, the windward girder(s) may shield the more leeward girders from the wind loads, see Figure 7-12. This shielding effect will increase as the ratio of girder spacing to girder depth, s/d , decreases. This effect is accounted for in this Manual in the wind net force coefficient, C_f , used to calculate the wind load to the girder system. The wind net force coefficients, C_f , in Table 7-4, are taken from the Florida Department of Transportation 2010 Structures Manual, Section 2.4.3, "Wind Loads during Construction" pressure coefficients, C_p .

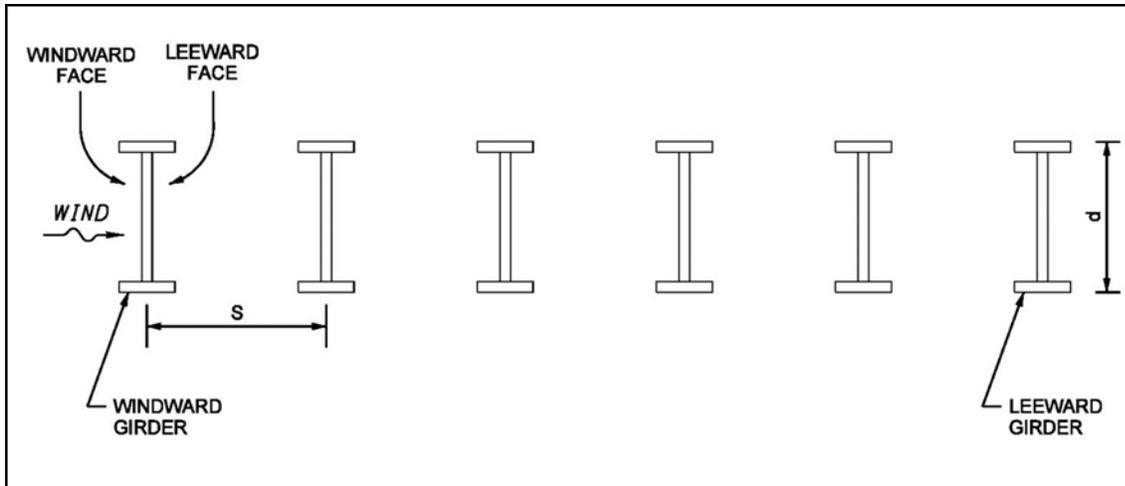


Figure 7-12 Girder Wind Load Terminology

The shielding effects of a two-girder system can be considered using information from “Wind Effects on Structures (3rd Edition)” by E. Simiu and R. Scanlan. Based on limited test data on a two-girder system, a C_f value of 2.2 is appropriate for systems where the s/d ratio is less than 2, which is normally the case for multi-girder bridges. The theoretical C_f for a single plate girder is given as 2.04 in Table 4.5.1 of “Wind Effects on Structures”.

The provisions of British Standard 5400, Design Manual for Roads and Bridges, Volume 1, Section 3, Part 14 (Loads for Highway Bridges), specify a force coefficient of 2.2 for a single girder, or $C_f = 2(1+0.05s/d)$ with a minimum C_f of 2.2, for multi-girder systems. This equates to the use of $C_f = 2.2$ for girder spacing up to twice the girder depth, which covers the usual cases. Note that the $C_f = 2.2$ is consistent with the values computed from Simiu and Scanlan’s book, as well as the Florida DOT values.

Based upon the information discussed above, the C_f for girder bridges should be taken as 2.2 applied to the windward area of the windward girder for single girders or for multiple girders where s/d is 2 or less. Where s/d is greater than 2, $C_f = 2(1+0.05s/d) \leq 4$ should be used. For bridges with superelevation, the windward area should be taken based on the overall superstructure depth; however, critical loading conditions typically occur with only one or two girders in place.

Research of wind loads on open girder systems recently completed under sponsorship of the Florida DOT (Consolazio and Gurley, 2013) suggests that for some wide multi-girder configurations the effective drag coefficient may be larger than has commonly been used. This research also indicates that the bracing between the windward and first interior girder may see increased loads due to suction between the girders. However, until additional verification of the proposed wind load model from the report is completed, any use of the proposed wind shielding model should be made with caution so that it is not inappropriately applied.

For box girders with sloping webs, the vertical wind load component should be considered.

The vertical wind effect on girder bridges during erection before deck formwork is placed acts on a relatively small exposed horizontal area and counteracts the self-weight of the girders; thus, it need not normally be included as a design loading. Even with deck forms in place, uplift effects on the girders are rarely a concern, since the formwork weight opposes the wind uplift, although the formwork itself may be affected. When erection takes place over active traffic, some vertical wind load is produced by truck traffic passing under the bridge. As there is evidence that these loads are small and affect only a limited tributary area they are normally neglected. Where girder bridges are built on a significant grade or superelevation, or where unique terrain is present, such as a bridge along a hillside or canyon wall, the effects of uplift may need to be considered. In this case, the uplift may be computed from provisions of ASCE 7-10 treating the girders, and formwork if present, as a roof.

COMPONENT TYPE	CONSTRUCTION CONDITION	FORCE COEFFICIENT (C_f)
I-Shaped Girder Superstructure	Deck forms not in place	2.2 (1)
	Deck forms in place	1.1
U-Shaped and Box-Girder Superstructure	Deck forms not in place	1.5
	Deck forms in place	1.1
Flat Slab or Segmental Box-Girder Superstructure	Any	1.1

Table 7-7 Wind Net Force Coefficient, C_f , for Girder Bridges during Construction

(1) When s/d is greater than 2, where s is the girder spacing and d is the girder depth, C_f is equal to $2(1+0.05 s/d) \leq 4.0$

ϵ	FORCE COEFFICIENT (C_f)
<0.1	2
0.1 to 0.29	1.8
0.29 to 0.7	1.7

Where ϵ is the Solid Area of the Truss Members divided by the Gross Area of the Truss

Table 7-8 Wind Net Force Coefficient, C_f for Truss Bridges (during Construction) (After ASCE 7-10)

7.4.5 Seismic Loading

Seismic loads are not usually applied to girder bridges while under construction. Some owners may have requirements where girders are erected over traffic or other specific conditions. Reduced seismic loads are appropriate and should be based on owner guidance.

SECTION 5. GIRDER LIFTING

7.5.1 General Considerations

Lifting procedures should be developed to ensure that the girder has adequate strength and buckling resistance during lifting operations. Girder weight should be based on shop drawing weight or computed from member dimensions accounting for fabrication tolerances in finished dimensions. Steel girders should be lifted near their quarter points whenever possible, while concrete girders are normally lifted near the ends. Where a spreader beam is employed, the line of support, or line running through the girder lifting points, should pass through the center of gravity of the member, and the lifting reactions at each pick point should be equal. Cross-frames attached to the girders prior to lifting, and any other variations to the girder configuration should be considered in determining the center of gravity.

The girder bending moments should be determined by treating the girder as simply supported at the lift points. Lift points should not be assumed to provide any lateral-torsional restraint.

7.5.2 Steel Girders

7.5.2.1 Straight Girders

Lifting at two points is recommended. When I-girders are lifted by a single lifting cable, the girder is treated as two cantilever sections. If the girder is lifted at two locations using inclined cables, the girder is analyzed as a beam-column on two supports with the axial load equal to the horizontal load component in the lifting cables. Where the girder is lifted at two locations along its length using vertical lifting cables (or slings), analysis of the beam can be performed based on a beam with two supports. Ideally, lift points should be set at the girder quarter points, as discussed above.

For straight, doubly-symmetric girders lifted at two points (Figure 7-13), the elastic buckling moment capacity can be computed from the following:

$$M_u < \phi_b M_{cr} = \phi_b C_{bL} \frac{\pi}{L_b} \sqrt{EI_y GJ + E^2 I_y C_w \left(\frac{\pi^2}{L_b^2} \right)} \quad \text{Equation 7-7}$$

Where:

- M_u = Factored maximum moment from static analysis
- M_{cr} = Critical buckling moment
- ϕ_b = Buckling resistance factor = 0.9
- L_b = Unbraced length = L (total length of girder segment)
- E = Modulus of elasticity (ksi)
- I_y = Weak axis moment of inertia (in^4)
- G = Shear modulus (ksi)
- J = Torsional constant (in^4) = $\sum \frac{bt^3}{3}$
- C_w = Warping constant (in^6) = $\frac{I_y h^2}{4}$
- C_{bL} = Lift adjustment factor

$$C_{bL} = 2.0 \text{ for } \frac{L_{Lift}}{L} \leq 0.225 \quad \text{Equation 7-8}$$

$$C_{bL} = 6.0 \text{ for } 0.225 < \frac{L_{Lift}}{L} < 0.3 \quad \text{Equation 7-9}$$

$$C_{bL} = 4.0 \text{ for } \frac{L_{Lift}}{L} \geq 0.3 \quad \text{Equation 7-10}$$

Where:

$L_{\overline{\text{Lif}}}$ = Average length from the lift points to the ends of the girder (Figure 7-13)

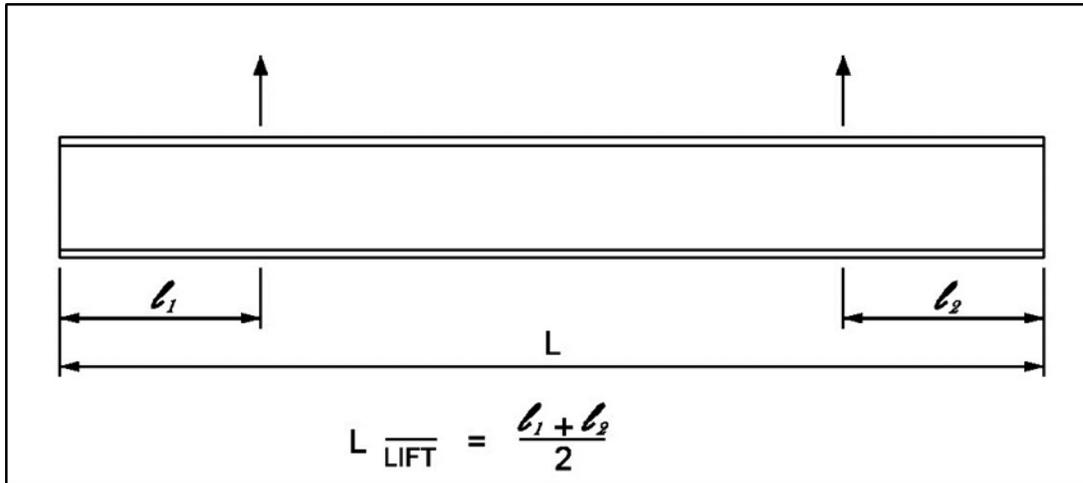


Figure 7-13 Girder Lifting Length Installation

The C_{bL} factors were developed by Schuh and Farris (2008). For singly-symmetric members, M_{cr} may be evaluated in accordance with the AASHTO Design Specifications Appendix D6.4.2.

7.5.2.2 Curved Girders

Curved girders are subject to a rigid-body rotation and cross-sectional twisting during lifting. Curved girders are normally lifted at two points using a spreader beam. When possible, the line of support, the line running between the lifting points, should pass through the center of gravity of the girder. Otherwise twisting will occur as shown in Figure 7-14. When girders are lifted with cross-frames attached, their contribution as well as variation in girder configuration must be included in determining the center of gravity. To maintain rotational stability of the spreader beam, the lifting reactions at each pick point must be equal.

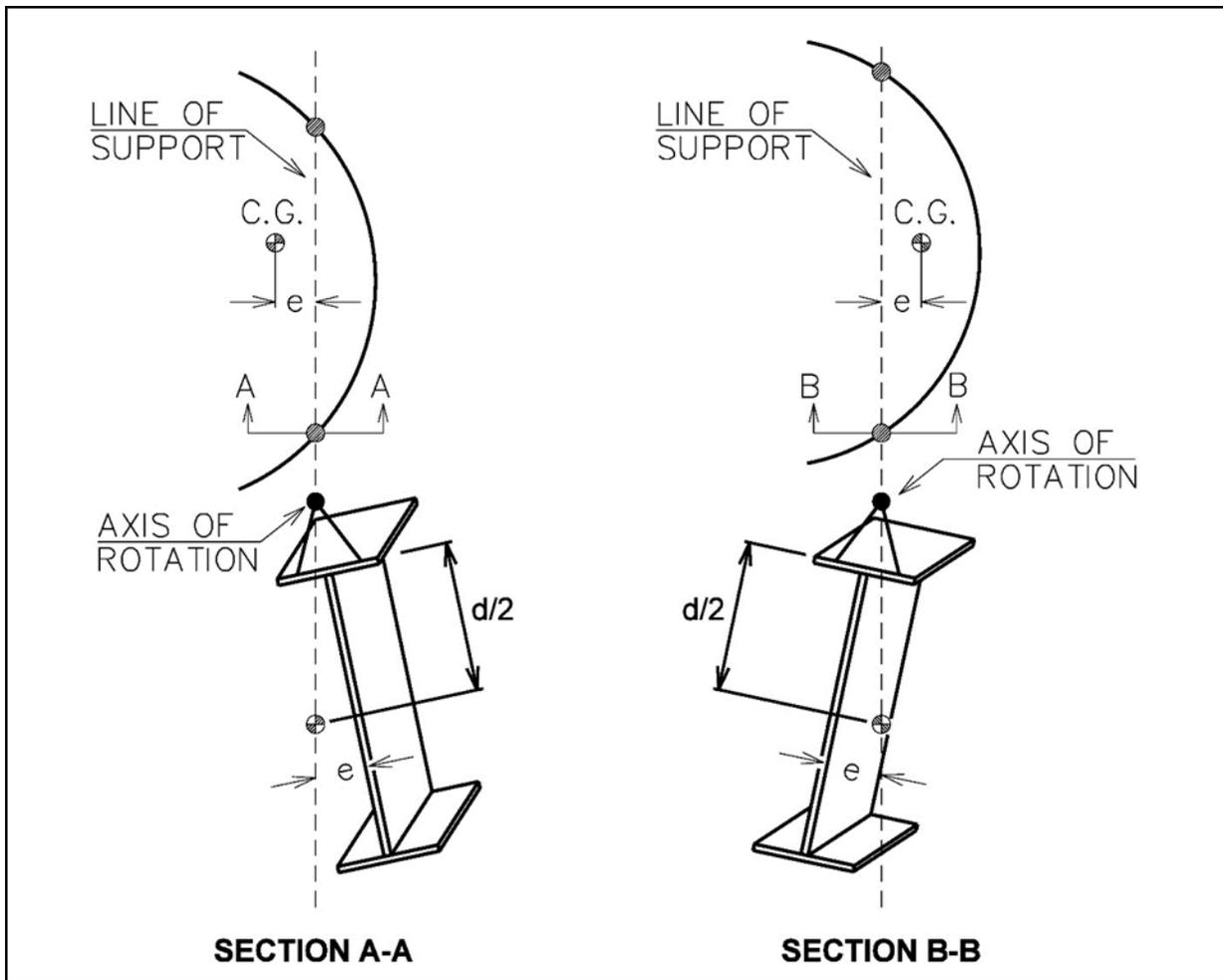


Figure 7-14 Effect of Eccentricity on Curved I-Girder Tilt

Figure 7-14 shows the geometry associated with lifting a curved girder. Girder pairs or box-girders may be handled in a similar manner. The lift points are to be selected to pass through the center of gravity. To produce equal lift cable loads, the lift points are placed equidistant from the center of gravity, Figure 7-15, as determined by the following equations (Stith, Schuh, et al 2010):

$$\cos \theta' = \frac{\bar{D}}{R} \quad \text{Equation 7-11}$$

$$\theta' = \cos^{-1} \left(\frac{\bar{D}}{R} \right) \quad \text{Equation 7-12}$$

$$\theta_{Lift1} = \bar{\theta} - \theta'$$

Equation 7-13

$$L_{Lift1} = R\theta_{Lift1}$$

Equation 7-14

$$L_{Lift2} = R(\theta_{Lift1} + 2*\theta')$$

Equation 7-15

Where the terms are as shown in Figure 7-15:

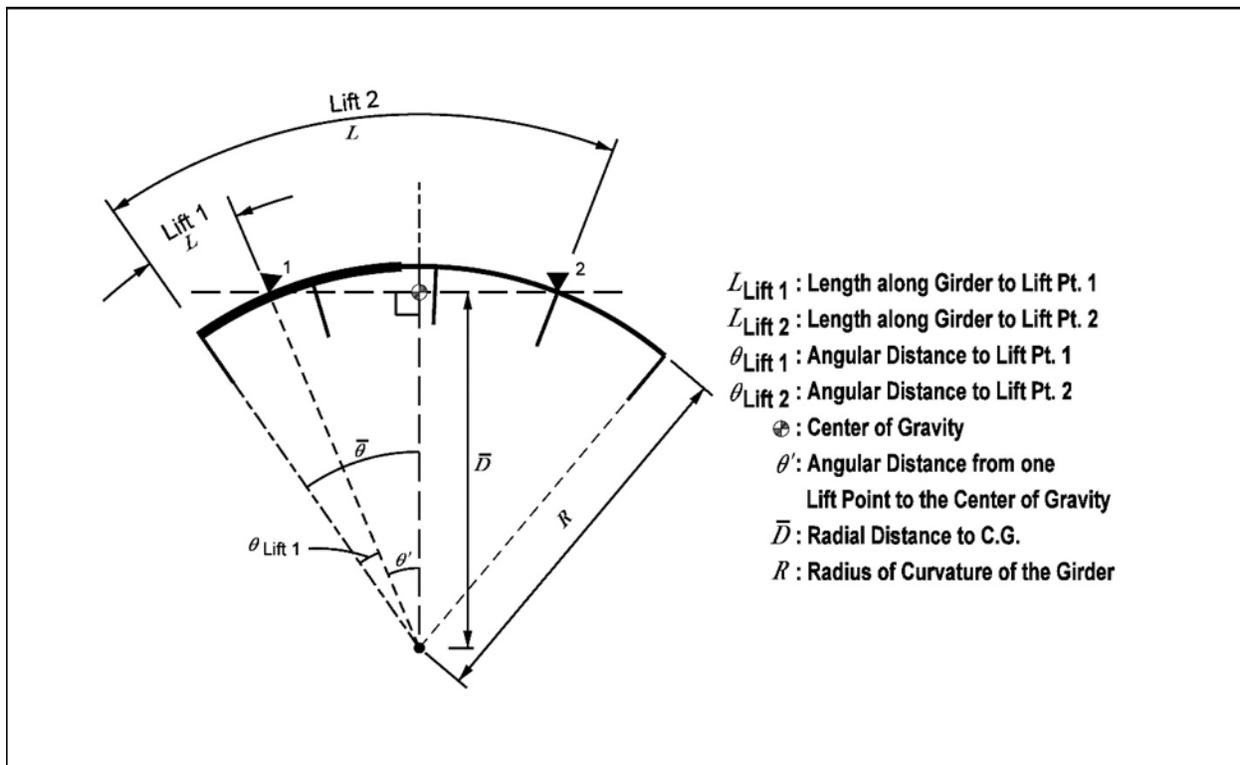


Figure 7-15 Zero Rotation/Equal Force Lift Equation for Curved I-Girder

The lift cable load is then one half the total girder weight, and the spreader beam length is:

$$L = 2R \sin(\theta')$$

Equation 7-16

When the central angle subtended by the girder length for a curved girder is less than 3 degrees, the girder buckling capacity may be calculated as a straight girder using Equation 7-7

When the girder(s) cannot be lifted through their center of gravity, girder roll and torsional loading should be investigated. A calculation method for this case can be

found in Stith, Schuh, et al (2010). Solution of this problem can be performed using a public domain spreadsheet program developed by the University of Texas, UT Lift, as discussed in Chapter 6.

7.5.3 Concrete Girders

When a precast, prestressed concrete beam hangs from flexible supports, such as cables during erection, it is free to roll about the line passing through the center of rotation at each lifting point, or the roll axis, as shown in Figure 7-16. Due to beam sweep tolerances and lifting point placement tolerances, the center of gravity of the beam will be located on one side or the other of the roll axis. This eccentricity causes the beam to rotate about the roll axis by a small angle, which causes a component of the beam self-weight to be applied about the weak axis of the beam. The lateral component of the self-weight creates a lateral deflection of the beam and increases the roll angle, which causes further lateral deflection. Depending on the lateral stiffness of the beam, it will reach equilibrium at a slightly larger roll angle or the roll angle will increase to a point where the lateral bending is sufficient to initiate failure of the beam. The final equilibrium position of the hanging beam is shown In Figure 7-16.

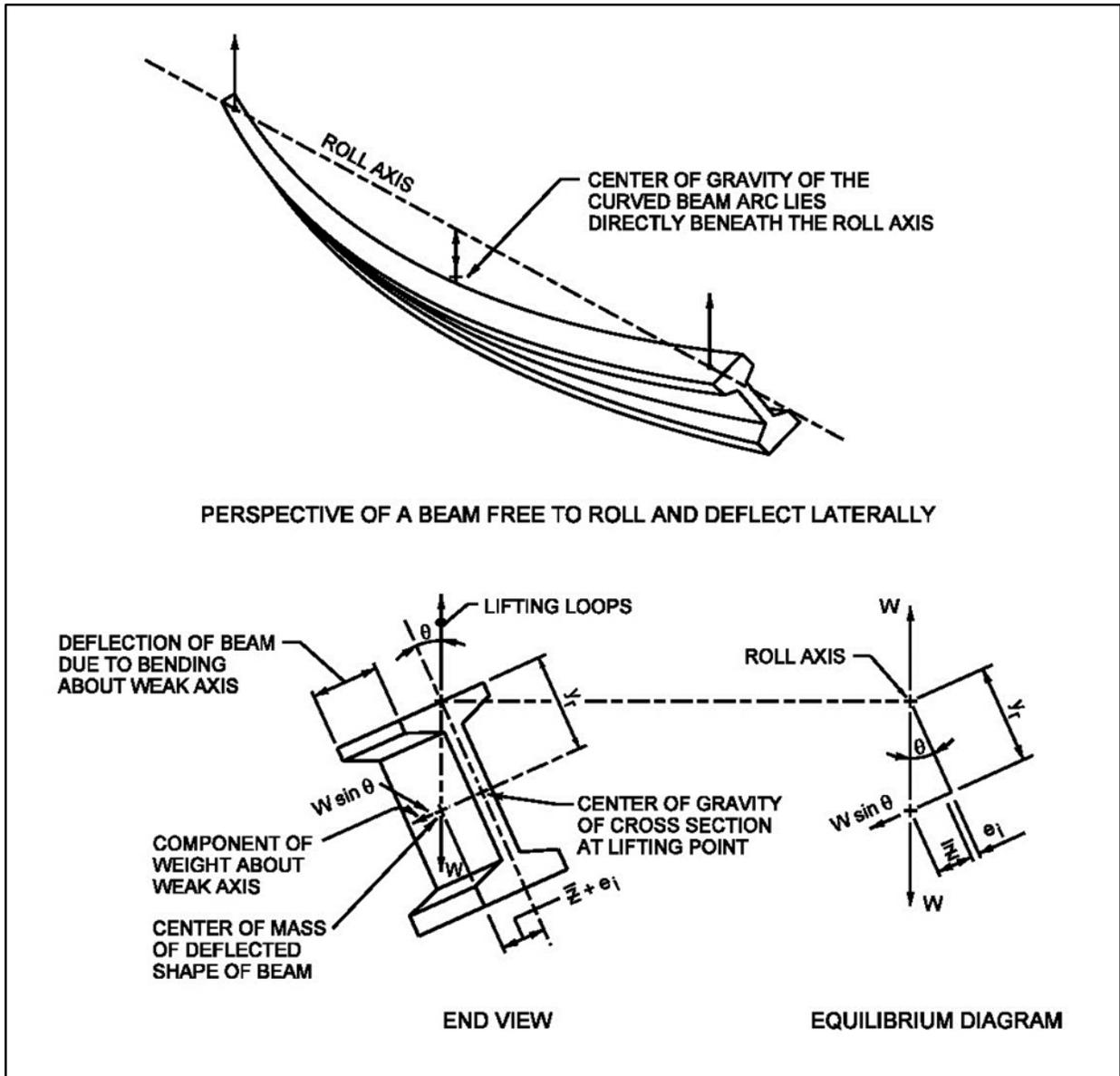


Figure 7-16 Final Equilibrium Position of Hanging Concrete Precast Beam

The hanging beam can be analyzed using the equations presented in the PCI Bridge Design Manual, Third Edition (2011). The equations presented herein are taken from Chapter 8 of that publication and were originally developed in the work of Mast (1993). Critical conditions for lifting normally occur when the girders are initially moved from the casting beds by the precaster as the concrete strength will be less than when the girder is set in the bridge. Analysis of the girders for lifting at the plant and for transport to the job site is carried out by the precaster and is not discussed further in this Manual. Should re-transport of girders be necessary after site delivery and unloading, the girders should be re-evaluated using the equations for transport presented in the PCI Bridge Design Manual.

Based upon past successful experience, a safety factor of 1.0 is used in checking beam cracking and 1.5 is used for girder roll stability (PCI, 2011). Thus the controlling equations are as follows:

$$\text{Rollover: } 1.5 \leq \left[\frac{y_r \theta'_{\max}}{\bar{z}'_o \theta'_{\max} + e_i} \right]$$

Equation 7-17

$$\text{Cracking: } 1.0 \leq 1 \div \left[\frac{\bar{z}_o}{y_r} + \frac{\theta_i}{\theta_{\max}} \right]$$

Equation 7-18

The tilt angle at rollover (failure), θ'_{\max} , may be taken as:

$$\theta'_{\max} = \sqrt{\frac{e_i}{2.5 \bar{z}_o}}$$

Equation 7-19

In which:

- e_i = the initial lateral eccentricity of the center of gravity with respect to the roll axis (in).
- y_r = the height of the roll axis above the center of gravity of the beam (in).
- \bar{z}_o = the theoretical lateral deflection of the center of gravity of the beam, computed with the full weight applied as a lateral load, measured to the center of gravity for the deflected arc of the beam (in).
- θ_i = e_i / y_r (rad)
- θ_{\max} = tilt angle at which cracking begins, based on tension at the top corner equal to the concrete modulus of rupture (rad)
- \bar{z}'_o = lateral deflection of girder center of gravity including rotation effects (in)
 $= \bar{z}'_o (1 + 2.5 \cdot \theta'_{\max})$

The assumed eccentricity, e_i , may be taken as a minimum of 1/4 inch to account for misplacement of the lifting device (loops, etc.) plus 1/8 inch for each 10 feet of beam length or fraction thereof. This is based on standard PCI fabrication tolerances, and the recommendations of PCI (2011).

Use of a total sweep of 1/16-inch per 10 feet of beam length is based on using the beam sweep tolerance for I-girders or bulb-tee girders as given in the PCI Manual for Quality Control for Plants and Production of Structural Precast Concrete Products, MNL 116-99, Fourth Edition. Work by Cojocar (2012), based on field measurements of precast girders, concluded that the PCI sweep tolerance is a reasonable value. However, for longer girders or where girders are stored for extended time periods, actual measurements of sweep are recommended to be used in stability calculations.

The effect of camber should be included in computing the center of gravity of the girder.

For a beam with overall length, l , and equal overhangs beyond the lifting points of length, a , at each end:

Equation 7-20

$$\bar{z}_o = \frac{w}{12EI_y l} \left[0.1(l_1)^5 - a^2(l_1)^3 + 3a^4(l_1) + 1.2(a^5) \right]$$

Where:

$$l_1 = l - 2a$$

θ_{max} = the tilt angle at cracking which equals M_{lat} divided by M_g

M_g = service level strong-axis moment in girder due to self-weight

M_{lat} = service level weak-axis moment that would cause cracking in top flange of girder

SECTION 6. STEEL GIRDER STABILITY AND BRACING

7.6.1 General

As discussed in Chapter 5, girder bracing systems provide stability to the girders and improve the lateral and torsional stiffness of the bridge during construction and in its completed state. The AASHTO Design Specifications consider bracing members that carry calculated loads based on a structural analysis to be primary members, e.g. bracing members in curved steel bridges, otherwise bracing members may be considered to be secondary members. A first-order analysis of a straight bridge under construction may indicate zero forces in the bracing members, yet failure to properly install the necessary bracing during construction can lead to failure due to instability. Lateral-torsional buckling often controls steel I-girder design during construction. In order to control girder stresses and deformation bracing systems must possess adequate strength as well as stiffness. Further discussion on strength and stiffness requirements for bracing is contained in Chapter 5.

Cross-frames or diaphragms between adjacent girders are termed nodal torsional braces herein since the bracing restrains girder twist at the brace location. The distance between the nodal braces defines the girder unbraced length. For straight steel girder bridges, the bracing system design is normally dominated by stability and skew effects. The effects of torsion and lateral flange bending generally control bracing design for curved steel girder bridges.

In assessing the stability of the bridge during construction, the bracing requirements should be investigated at each stage of the erection sequence. The investigation of the need for bracing in I-girder bridges should consider the following as a minimum:

- Transfer of lateral wind loads between girders and to the supports. Wind loads will be carried by both flanges in proportion to their lateral stiffness.

- Stability of the top flange in compression, particularly during initial girder placement and deck placement
- Stability of the bottom flange in compression zones
- Global stability for the girder system
- Provision of lateral support to the bottom flange when deck overhang brackets are used
- Control of flange lateral bending
- Control of girder geometry

These checks should also be performed on bridges undergoing renovation, particularly during redecking of composite girder spans and box-girders.

7.6.2 Lateral Bracing

In steel I-girder bridges, lateral bracing may be required to resist wind loads and provide girder stability prior to deck construction. When lateral bracing is deemed necessary, it should ideally be connected directly to the flange being braced using a bolted connection. However, bolting directly to the top flange may interfere with construction using stay-in-place forms. In these cases bracing should connect as near the top flange as practical. The lateral-bracing system is designed as a truss.

Placing flange lateral bracing in the same plane as the top flange minimizes live load lateral bracing stresses in the finished bridge. Permanent bottom lateral bracing in I-girder bridges has the effect of creating a quasi-closed section in combination with the hardened concrete deck, and the bracing may attract significant forces due to live load bending of the girders. In some cases, removal of lower lateral bracing that is only required during construction might be considered. This also minimizes potential fatigue prone details.

Bracing design forces include computed forces due to vertical and lateral loads as well as stability induced forces. Design of bracing at each stage of erection must be consistent with the forces and bracing that are associated with that stage. Design of bracing components should be in accordance with the design equations presented in Section 3 of Chapter 5 for this particular bracing configuration.

7.6.3 Global Stability

Systems of two to four noncomposite I-girders interconnected by cross-frames or diaphragms should be investigated to assure that failure in a system buckling mode as discussed in Section 4 of Chapter 5 does not occur. System buckling may be investigated using Equation 5-12, or an eigenvalue analysis as discussed in Chapter 6.

When evaluating global stability using Equation 5-12, use of a 1.25 dead load factor is recommended. Appropriate ranges of eigenvalues are discussed in Chapter 6. The effects of lateral load in combination with the system buckling resistance calculated from Equation 5-12 may be evaluated using the AASHTO Specifications Equation A6.1.1-1, by replacing the term $\phi_f M_{nc}$ on the right hand side of the equation with $\phi_{bk} M_{gs}$ calculated by Equation 5-12 (and adjusted for the number of girders in the system). Note that ϕ_{bk} is defined above in Section 3.

$$M_{gs} = \frac{\pi^2 SE}{L^2} \sqrt{I_{eff} I_x}$$

The Yura equation: has recently been incorporated into AASHTO LRFD as Eq. 6.10.3.4.2-1 for checking system buckling during deck pour. The approach presented above is applicable for intermediate steel checks prior to the deck pour ($M_u \leq \phi M_{gs} / n$), but the AASHTO approach omits the ϕ factor and limits the total sum of the factored positive girder moments to 50% M_{gs} during the deck pour (Strength VI load combination). Should the sum of the moments exceed 50%, the design can add flange level lateral bracing, revise the girder spans/sizes to increase system stiffness, or evaluate the amplified girder second-order displacements and verify that they are within owner tolerances. Note that amplification can also occur under steel-only dead load as the buckling limit is approached, but the recommended system buckling ϕ factor and Strength I/III load factors should provide an adequate level of safety for most narrow systems subject to buckling in the steel-only condition.

SECTION 7. CONCRETE GIRDER STABILITY AND BRACING

7.7.1 General

Precast concrete girders may be susceptible to lateral-torsional buckling or rollover instability prior to the girders being stabilized by the casting of the top slab, which provides continuous lateral support to the girders, preventing buckling. The condition when the girder is set on the bearings during erection is represented by a simply supported girder, supported on bearings at the ends, with no overhangs. The supports typically allow rotation about both the girder's strong and weak axes, with elastomeric bearings pads most commonly used. Factors such as bearing slope, bearing skew relative to the girder centerline, bearing type, and girder imperfections can have significant effects on the stability behavior.

The School of Civil and Environmental Engineering of the Georgia Institute of Technology prepared a study of the stability of precast prestressed girders for the Georgia Department of Transportation (Sureick, Kahn, et. al., 2009). This work included calculation of lateral-torsional buckling and rollover stability for simply supported AASHTO Type I through Type IV girders, with a minimum concrete compressive strength of 6 ksi, and their ends restrained against rotation. The results are shown in Table 7-6, and provide girder lengths below which detailed analysis for lateral-torsional buckling and rollover may not be required. These lengths include a safety factor of 1.5.

AASHTO Girder Type	Length below which lateral-torsional buckling does not occur (feet)	Length below which rollover about support does not occur (feet)
I	127	75
II	133	80
III	155	100
IV	175	110
V	197	135
VI	193	140

Table 7-6 AASHTO Concrete Girder Stability Limits

Prestressed, precast concrete bridge girders are designed to preclude flexural cracking under their self-weight. As a result, the girder section weak axis bending moment of inertia and torsional stiffness are sufficiently large to prevent lateral-torsional buckling for AASHTO and PTI standard bulb-tee and similarly stiff sections (Hurff and Kahn, 2012). Stability is therefore controlled by rollover. This is consistent with the guidance in Chapter 8 of the PCI Bridge Design Manual.

7.7.2 Simply Supported Condition - Girder Rollover

For prestressed concrete bridge girders, rollover will typically control over buckling for cases where the ends are not laterally braced because the elastic buckling load will be greater than the self-weight. Rollover is caused by initial girder rotation compounded by a lack of flatness of the bottom flange of a prestressed concrete girder and the roll flexibility of the bearings increasing the girder rotation. The rotation of a girder causes a component of the girder weight to be applied about the weak axis of the girder, which causes lateral deflection and further shifts the center of gravity of the girder which causes further lateral deflection. Figure 7-17 shows lateral deflection and rotation imperfections at equilibrium of a prestressed girder on elastic supports. Note that for skewed supports out-of-plane rotations need to be considered in addition to in-plane rotations.

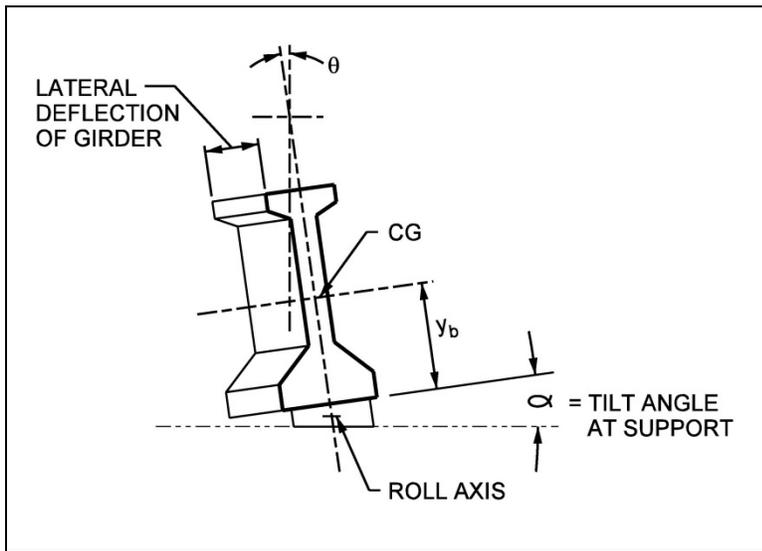


Figure 7-17 Equilibrium of Girder on Elastic Support

Rollover of a simply supported girder is determined primarily by the properties of the support rather than the girder. Prestressed concrete girders of ordinary proportions have been found to usually have sufficient lateral bending strength to withstand greater angles of inclination than can be resisted by the supports. Therefore, it is necessary to determine the properties of the supports in order to perform the rollover analysis. Elastomeric bearing pads are the most common bearing type used with precast girders and provide a springy support for the concrete girders. The rotational stiffness constant, K_{θ} , is determined by the dimensions and properties of the pad and may be calculated from the pad's vertical stiffness properties as shown in Yazdani et.al. (2000). Girder camber and roll effects contribute to an uneven load distribution to the bearing pad which effectively reduces its stiffness. Refer to NCHRP Report 298, "Performance of Elastomeric Bearings" and NCHRP Report 596, "Rotation Limits for Elastomeric Bearings" for information on bearing pad properties.

When the bearing pads are set on a skew to the girder centerline, an additional uneven load distribution to the pad is created that further reduces its effective stiffness. In addition, the variation in bearing pad stiffness is not linear with load. The axial stiffness

under girder self-weight alone will be less than that under service load. Girders should be evaluated to ensure a sufficient safety factor on rollover and cracking. A minimum safety factor on rollover of 1.5 is recommended by Mast (1993) and PCI (2013) along with a minimum safety factor on cracking of 1.0.

Equation 7-21 below can be used to compute the factor of safety for a simply supported precast concrete girder against cracking (Mast, 1993):

$$1.0 \leq \frac{r(\theta_{\max} - \alpha)}{z_o\theta_{\max} + e_i + y\theta_{\max}} \quad \text{Equation 7-21}$$

Where:

- e_i = initial lateral eccentricity of center of gravity with respect to roll axis (in) (minimum of 1 inch plus 1/8 inch for each 10 feet of girder length or fraction thereof)
- h_r = distance from bottom of girder to roll axis
- w = girder weight per unit length (k/in)
- E_c = modulus of elasticity of concrete
- I_y = moment of inertia of beam about weak axis (in.⁴)
- L = girder span (in)
- z_{\max} = maximum resisting moment arm (typically half of bottom flange width) (in)
- $\theta_{\max} = M_{lat}/M_g$
- M_{lat} = service level weak-axis moment that would cause cracking in top flange of girder (k-in)
- M_g = service level strong-axis moment in girder due to self-weight (k-in)
- r = radius of stability = K_θ / W (see Figure 7-19)
- K_θ = sum of rotational spring constants of supports (k-in/rad)
- W = total weight of beam (k)
- y = height of center of gravity of beam above roll axis (in)
- α = tilt angle of support (rad)

$$\theta'_{\max} = \frac{(z_{\max} - h_r\alpha)}{r} + \alpha \text{ (rad)}$$

$$\bar{z}_o = \left(\frac{1}{120} \right) \left(\frac{wL^4}{E_c I_y} \right)$$

$$\bar{z}'_o = \bar{z}_o (1 + 2.5\theta'_{\max})$$

The height of the center of gravity of the beam above the roll axis should include beam camber. The tilt angle of the support should include both the out of flatness tolerance of the bottom of the beam and the bearing seat cross slope.

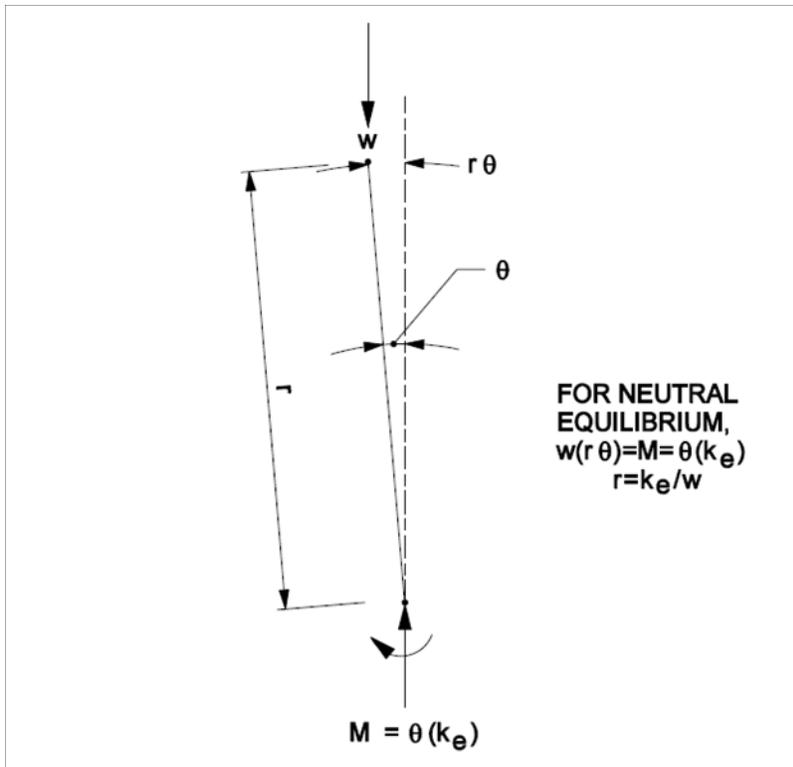


Figure 7-18 Definition of Radius of Stability (r)

Equation 7-22 below can be used to compute the factor of safety against rollover for a simply supported precast concrete girder (Mast, 1993)

$$1.5 \leq \frac{r(\theta'_{\max} - \alpha)}{z'_{o'}\theta'_{\max} + e_i + y\theta'_{\max}} \quad \text{Equation 7-22}$$

Where the terms are defined above.

The effects on roll stability of skewed elastomeric bearing pads may be accounted for by use of a modification factor applied to the bearing pad stiffness. The modified stiffness equals the calculated stiffness multiplied by the modification factor. Based upon the results of physical bearing pad tests (Consolazio and Hamilton, 2012), as well as computational studies (Consolazio and Hamilton, 2007), the Florida Department of Transportation developed the bearing pad stiffness modification factors shown in Table 7.7. These values are based on Florida DOT bearing dimensions so may not be applicable for other states; however the general trend should be similar. These factors include the effect of girder end rotation due to camber, as well as skew. Linear interpolation may be used for skew angles between zero and 15 degrees and between 15 degrees and 60 degrees.

Skew Angle (degrees)	0	15	30	45	60
Stiffness Modifier	1	0.40	0.32	0.26	0.21

Table 7-7 Skew Angle Bearing Pad Stiffness Modification Factors

Lateral wind load effects are added by computing equivalent eccentricities that are added to the assumed initial eccentricity, e_i , to compute a total eccentricity to use in the above equations. The eccentricities due to wind include the lateral wind deflection for the uncracked girder, plus an overturning moment eccentricity equal to the wind overturning moment computed about the bottom of the bearing pad divided by the girder self-weight.

Increasing the bottom flange width is the most effective way to increase the stability of a simply supported precast concrete girder. This will also increase the bearing pad width, which should preferably be equal to the width of the girder bottom flange, minus the chamfers, as a minimum (Mast, 1993).

In order to avoid the complexity of the rollover analysis which relies on variables such as bearing rotational stiffness and the tilt angle of the bearings, it is important to brace the precast concrete girders at the supports during erection. The girders can be braced from the top flange down to the abutment or pier seats, or multiple adjacent girders can be braced to one another. Chapter 3, Figures 3-26 through 3-32 illustrate typical bracing options for precast concrete beams.

SECTION 8. CONCENTRATED LOAD EFFECTS

7.8.1 Bearing Stiffeners

At bearing locations and at other locations on steel girders subjected to concentrated loads, where the loads are not transmitted through a deck or deck system, webs without bearing stiffeners shall be investigated for the limit states of local web yielding and local web crippling according to the provisions of Appendix D6.5 – *Concentrated Loads Applied to Webs without Bearing Stiffeners* of the AASHTO Design Specifications. Web sideway buckling, if applicable, shall be investigated according to AISC Specification J10.4.

7.8.2 Local Effects of Lifting Clamp Loads

The local effects of lifting clamp loads on the girder flange should be addressed as part of the erection evaluation. A commonly used approximate load distribution model to assess flange bending is shown in Figure 7-19. The bending moment due to the clamp load is considered distributed over an effective flange length, C_{LE} , and the bending stress in the flange is computed considering that part of the flange as a cantilever beam. The resulting stress (or applied moment) is compared to the allowable plate bending stress (or nominal plate bending capacity). While this is a simplified and conservative

approach, this conservatism helps to minimize any flange distortion. Using this method the local flange bending stress may be taken as:

$$f_{lb} = \frac{R_c k}{(b_f + C_L)(t_f)^2 / 6}$$

Equation 7-23

$$f_{lb} \leq 0.75 F_{yf}$$

Equation 7-24

Where:

- R_c = service level concentrated force at each flange edge (kip)
- F_{yf} = specified minimum flange yield stress (ksi)
- b_f = flange width (in)
- t_f = flange thickness (in)
- C_L = length of clamp along flange (in)
- k = distance from outer face of flange to web toe of fillet (in)

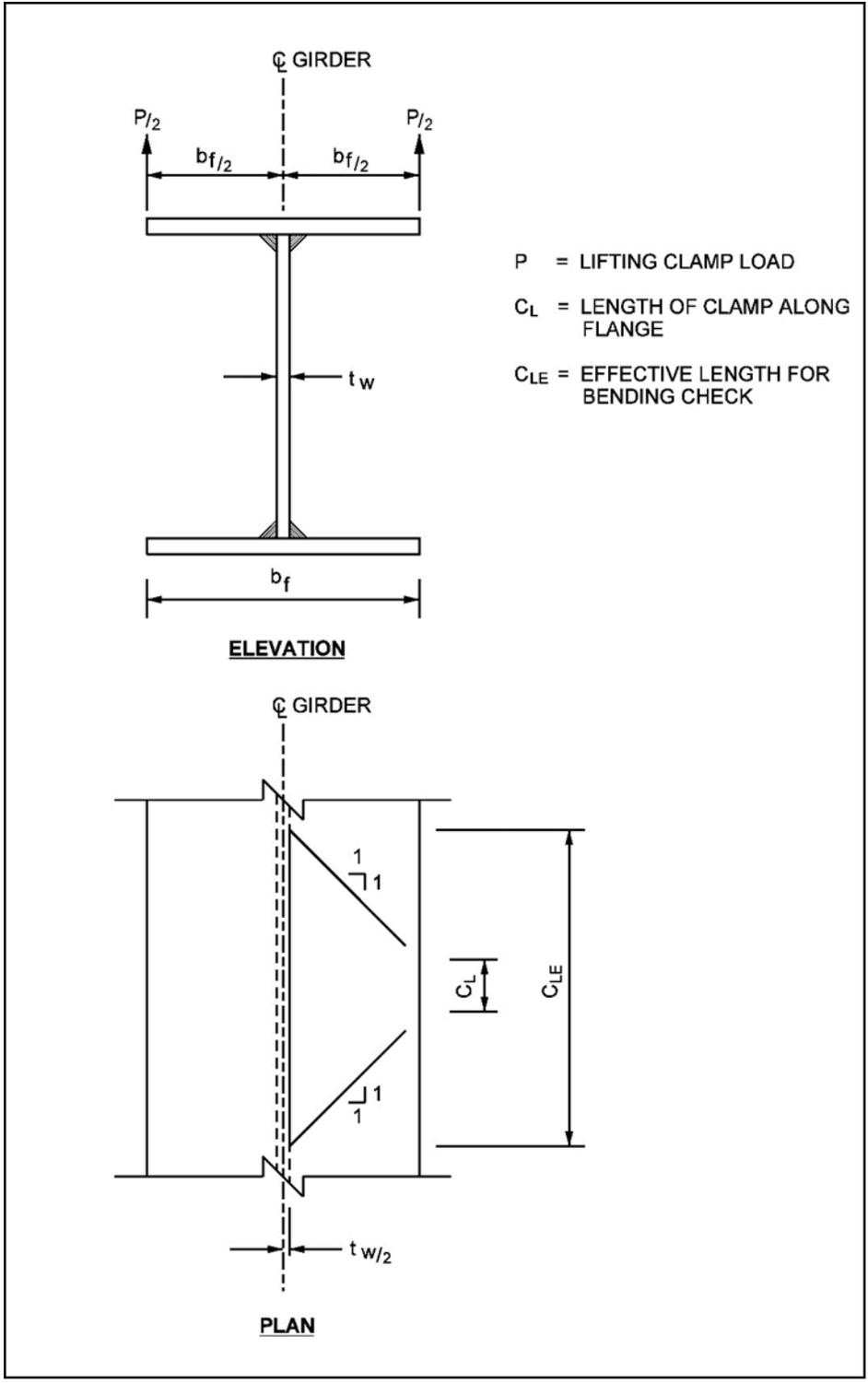


Figure 7-19 Steel Girder Lifting Clamp Load Distribution

SECTION 9. DEFLECTION CONTROL

Control of the girder vertical and horizontal deflections and girder rotations is important in assuring that erection fit-up can be achieved without applying excessive force to mating components, and that accumulated deformations are within finished structure limits. Deflections should be computed using the service load combination. Wind load is not normally included when computing deflections during deck placement.

Large lateral deflections which cause the girders to be out-of-plumb (i.e. rotate about the girder longitudinal axis) will also introduce secondary stress in girders that could be important. NCHRP Report 725 found that while a girder out-of-plumb of one percent had a negligible effect on girder or bracing stresses for curved steel I-girders, a six percent out-of-plumb increased girder stresses and lateral deflection by up to 20 percent.

Control of the rotation about the longitudinal axis of the girder during erection is particularly important in curved girder bridges, since both the supported girder and the girder being lifted for field connection will tend to rotate due to torsional effects. Rotation of the in-place girder should be minimized by installation of the permanent or temporary bracing, preferably at locations adjacent to the field splices. Limits for deflections and rotations during erection should be established by the erector, with input from the erector's engineer so that member fit-up is achieved without forcing members into position. Deflections must be kept within elastic limits such that the overall girders are in proper alignment upon completion of erection. While some variations in vertical dead load deflection can be accommodated in establishing final screed elevations, excessive girder twist makes setting formwork difficult and can effect reinforcing steel clearances.

As an example of lateral deflection limits, the Pennsylvania DOT limits the lateral deflection under service wind load to a value equal to the girder span over 150. This represents a transient load, and would not occur during active erection. Based on evaluation of data from a questionnaire sent to erectors, as well as girder analysis studies, Stith (Stith, Petruzzi, et. al. 2010) recommended limiting the girder out-of-plumb during erection for curved girders to 1.5 percent of the girder depth to facilitate fit-up of connections. Survey data indicated that a value of one percent to two percent was generally found by erectors to be a workable range.

SECTION 10. CONNECTIONS

7.10.1 Steel Connections

Connections to steel structures or components required as part of the erection operations must be designed in accordance with the applicable provisions of Article 6.13 of the AASHTO design specifications. Existing connections and splices should be evaluated for each stage of construction under consideration according to the same provisions, as applicable. Any proposed changes to the owner's standard specified

bolting procedures must be supported by engineering analysis addressing erection conditions. Reaming of bolt holes during erection should be permitted only with the approval of the engineer. Field drilling may be required in some situations.

7.10.2 Concrete Connections

Anchorage and other connections to concrete structures or components required as part of the erection operations should be designed in accordance with Appendix D of ACI 318, Building Code Requirements for Structural Concrete. Existing anchorages and connections should be evaluated for each stage of construction under consideration according to the same provisions, as applicable. When temporary connections are made to the permanent structure, provisions should be made for removal of the anchorage and repairs as may be required by the owner.

SECTION 11. TEMPORARY SUPPORTS (FALSEWORK)

Temporary supports and their associated components should be designed to carry vertical and lateral loads due to self-weight of the girders and wind and any loads that are applied to the temporary supports as the erection progresses. Temporary support design shall conform to the AASHTO Guide Specifications for Temporary Works. The effects of any longitudinal jacking during the erection should also be investigated. The elevation of the temporary supports should be such as to support the girders at their cambered no-load elevation. The use of temporary supports must not result in any overstressing of the girders. Jacks used in conjunction with the temporary supports should have a stroke adequate to permit full unloading. Unloading of temporary supports should be performed such that all temporary supports at each cross-section are unloaded uniformly. The deflections of the erected girders at the temporary supports when they are removed should be evaluated, and stability of the girders should be ensured prior to removal of the temporary supports.

Where appropriate, holding cranes may be used to temporarily support girders from above rather than use shoring towers located below the girders. Holding cranes will not provide a lateral brace. However, by altering the girder moment distribution and associated C_b value for the girder, bending stresses may be decreased.

SECTION 12. BEARINGS

Computed bearing rotations during each stage of construction under investigation should not exceed the rotational capacity of the bearing. Bearings should be installed such that, after dead load has been applied, sufficient rotation capacity should be available to accommodate rotations due to environmental loads and live loads. Expansion bearings should be installed so that they will be in the center of the permitted travel at an ambient temperature of 60°F, unless otherwise specified by the owner. Where required for erection stability, designs for blocking of the bearings or other methods of locking the structure in position should be designed and detailed based on the associated loads and local rotations.

SECTION 13. DECK PLACEMENT

Concrete-placements shall either be made in the sequence specified in the contract documents, or may be based on a sequence developed entirely by the contractor, in which case the contractor's engineer must evaluate the effects of the desired placement sequence according to the criteria specified in Article 6.10.3.4 of the AASHTO design specifications.

The duration of each placement should be specified in the construction plan. The time between placements should be determined to be such that the concrete in prior pours has reached an age or strength specified in the construction plan. Placements that include both negative and positive dead load moment regions should be placed such that the positive moment region is poured first. Any accelerating or retarding agents to be used in the concrete mix shall be specified. When changes in the deck placement sequence are made, revised deck drawings and associated fillet heights and girder elevations need to be prepared.

SECTION 14. SUMMARY

Design criteria for use in evaluation girder bridges during construction should account for the open configuration of the girders prior to deck placement and the likelihood of environmental loads that may occur over relatively short construction time durations. The criteria contained in this chapter were developed recognizing those conditions as well as AASHTO guidance on loads and load factors. The application of the criteria must achieve a structure that provides adequate strength and stability at all stages of erection.

CHAPTER 8

ERECTION PLANS AND PROCEDURES

SECTION 1. INTRODUCTION

The contractor's erection plans and procedures describe the detailed means and methods to be used by the contractor or erector in constructing the bridge girders and associated structural elements. The erection plans are developed from information in the contract drawings and specifications, the fabricator or precaster's shop drawings, and the engineering calculations prepared by (or for) the bridge erector. The erection plans must be job specific and include details of any temporary works required during erection. Complex erection projects may require input from the structural engineer of record in addition to the original design calculations such that the contractor can confirm constructability during various erection stages.

State departments of transportation and other bridge owners typically provide at least general requirements for erection plan submittal, with which the contractor must comply. Guidance on girder erection submittals is also included in the AASHTO/NSBA Steel Bridge Collaboration "Steel Bridge Erection Guide Specifications" and in Appendix B of NCHRP Report 725. This chapter provides information to supplement those requirements and address the type and level of detail that should be provided in erection plans and procedures.

Facilities owners adjacent to the bridge construction site may have erection requirements to protect their property that must also be satisfied. Railroads, in particular, may have very detailed submittal requirements, including design provisions for shoring that must be followed for work adjacent to or over tracks.

SECTION 2. PLAN PREPARATION

Erection plan submittals are prepared by the bridge erection contractor or subcontractor. The submittals should normally require at least a sketch of the work area and site logistics, as well as the sequence of erection and any supporting data, such as maximum pick weights and crane capacity charts. Submittals for larger or more complex projects may include preparation of detailed drawings and engineering calculations, piece-by-piece erection sequences, lifting diagrams and crane information, and detailed design and drawings of any temporary works.

The preparation of the submittal documents may be completed in-house by the erector, particularly for simple projects, or may involve an engineering consulting firm retained by the erector to provide support services. While erectors may have engineers as part of their staff, project commitments, as well as the extent and complexity of engineering

required for some projects, can result in retention of outside engineering services. Close collaboration between the erector and his consultant is needed to develop an erection plan that is efficient for the erector, satisfies engineering requirements for stability and strength at each erection stage and accommodates the specific site conditions.

Selection of erection methods, equipment, and temporary support systems is generally based on past erector experience and available equipment and materials, and this must be reflected in the engineering calculations.

In the preparation of the erection plans and supporting calculations, the design of temporary bracing components, shoring systems and similar components are based on specific materials' properties, and may utilize used materials. The contractor/erector must assure that the actual materials used in the field satisfy all requirements on which the design was based, and are in good condition. Where material properties cannot be documented, material testing may be required.

When the project includes work to an existing bridge, the erection plans and procedures must account for the effects of the work on the existing structure. This assessment should be based on as-built conditions of the existing structure, including any deterioration of bridge components. A complete structural assessment report for the existing structure may be required by some agencies (IDOT, 2004).

Erection plans prepared by an erector that is a subcontractor to the general contractor should be reviewed by the general contractor prior to submittal to the owner or reviewing agency. The general contractor should assure that all work activities on the project site are coordinated, work schedules are compatible, and overall site access and traffic control are accounted for.

SECTION 3. ERECTION PLANS

8.3.1 General

As noted previously, the extent of the information contained in the erection plans will vary with the bridge size and complexity and the site constraints. A sample set of erection plans for a steel girder bridge may be found in the AASHTO/NSBA Steel Bridge Collaboration, "Steel Bridge Erection Guide Specification," which may be downloaded from the AASHTO/NSBA website. Figures 8-1 through 8-6 and Figures 8-7 through 8-10 beginning on the following page show example erection plans for a skewed steel I-girder and a precast concrete girder bridge, respectively, taken from actual projects.

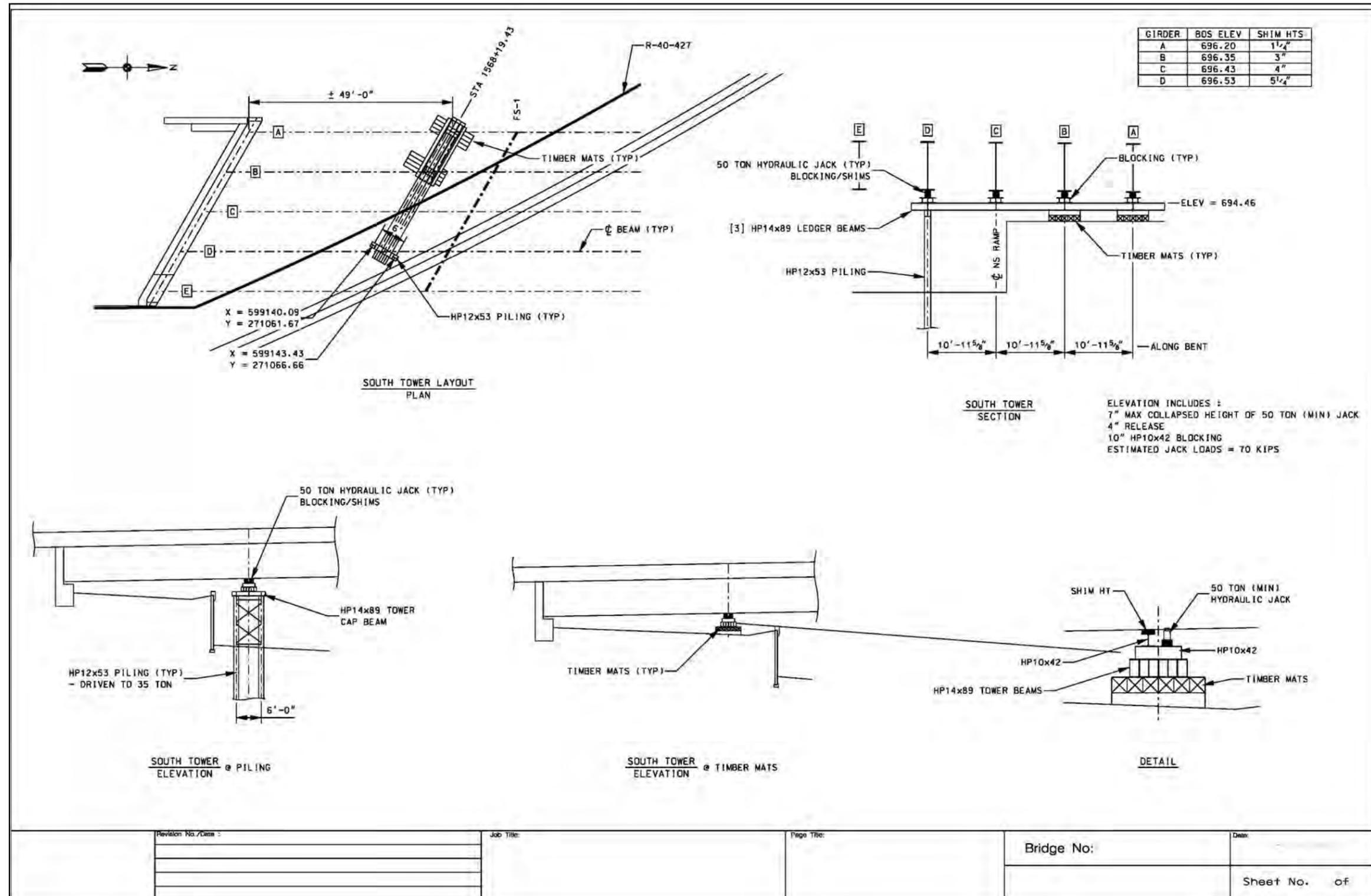


Figure 8-4 Steel Girder Erection Drawings 4 – Shoring Elevations

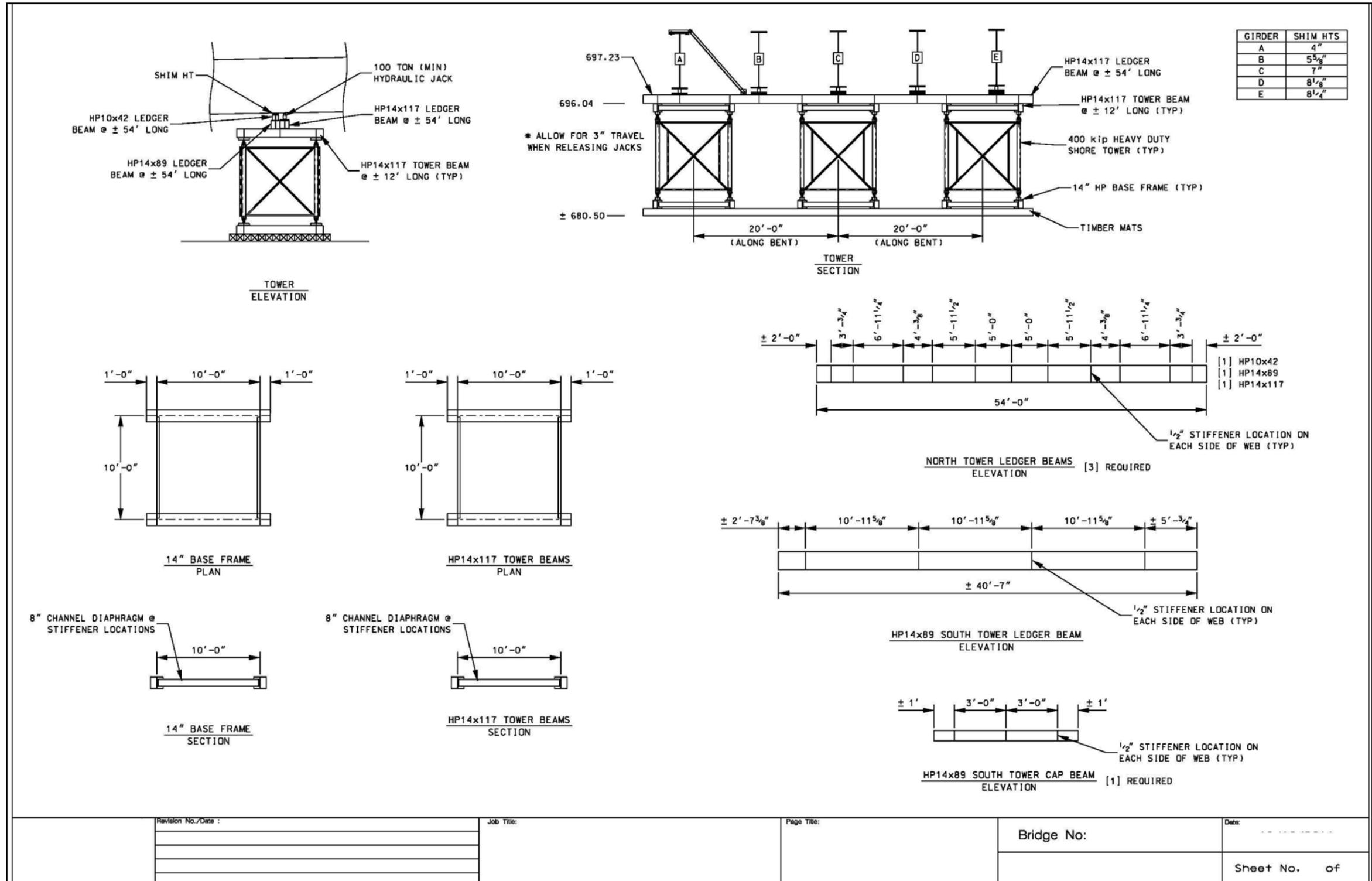


Figure 8-5 Steel Girder Erection Drawings 5 – Shoring Towers

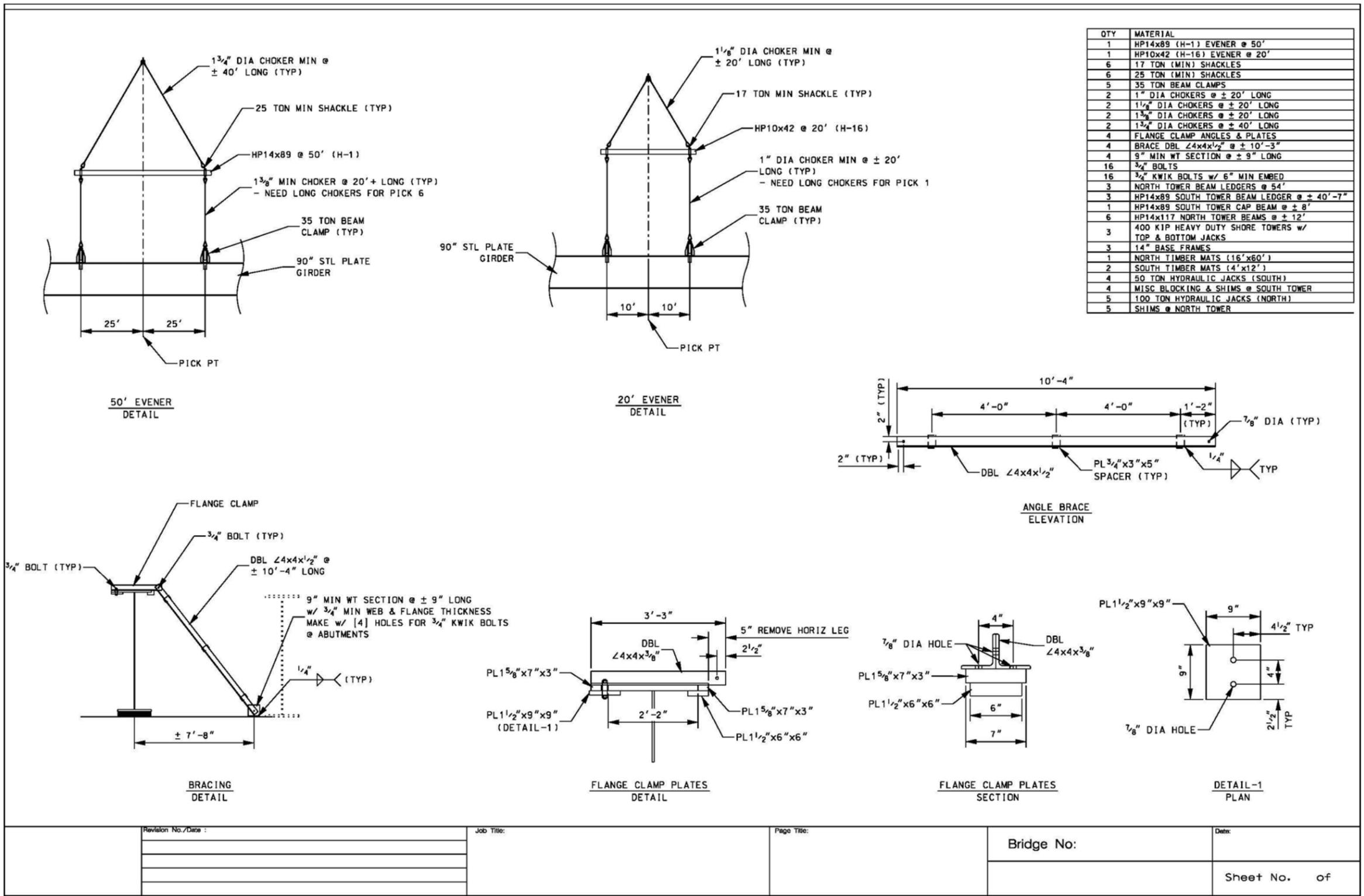


Figure 8-6 Steel Girder Erection Drawings 6 – Lifting and Local Bracing Details

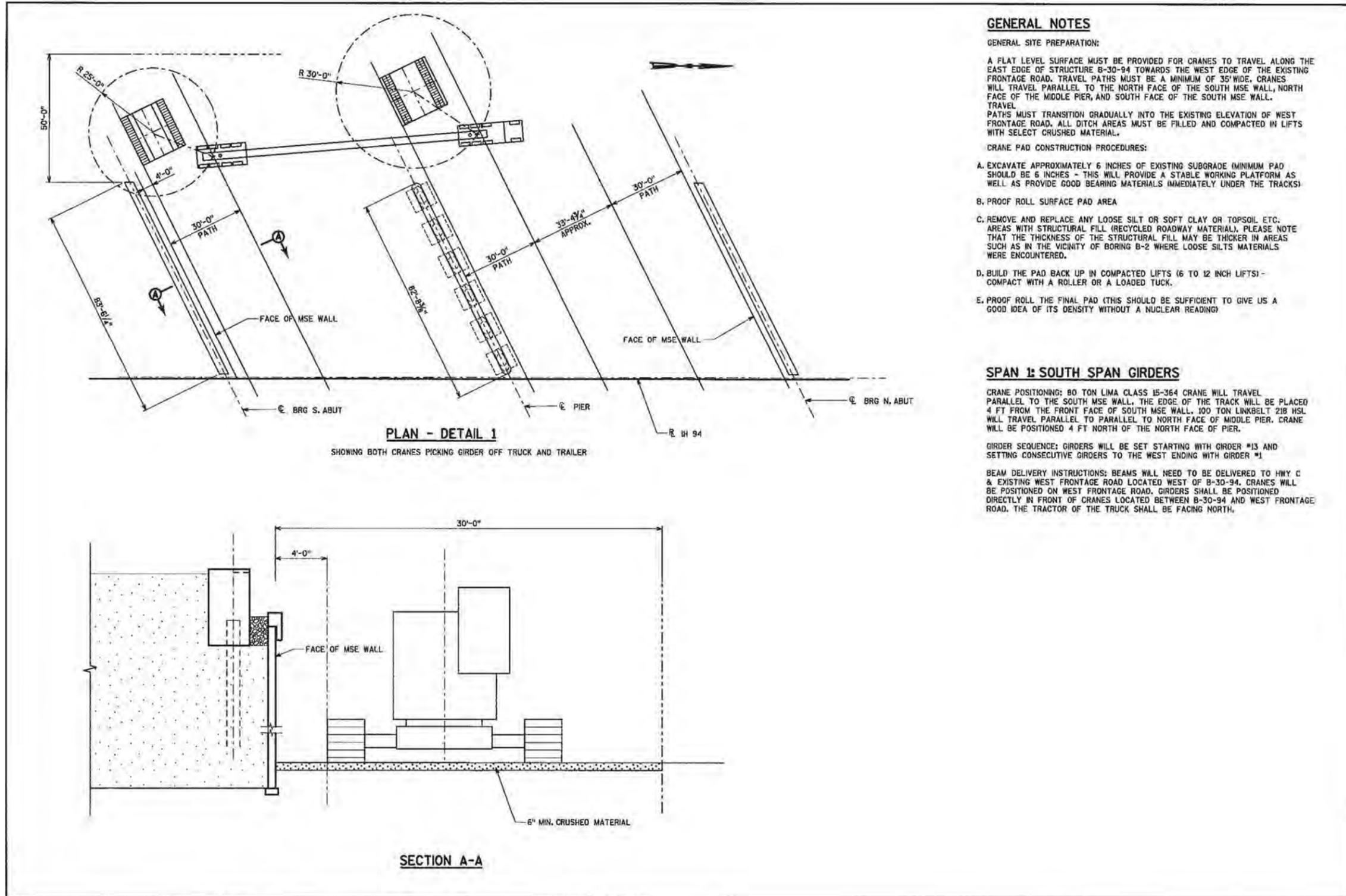


Figure 8-7 Concrete Girder Erection Plan 1 – South Span Girders Plan and Clearances

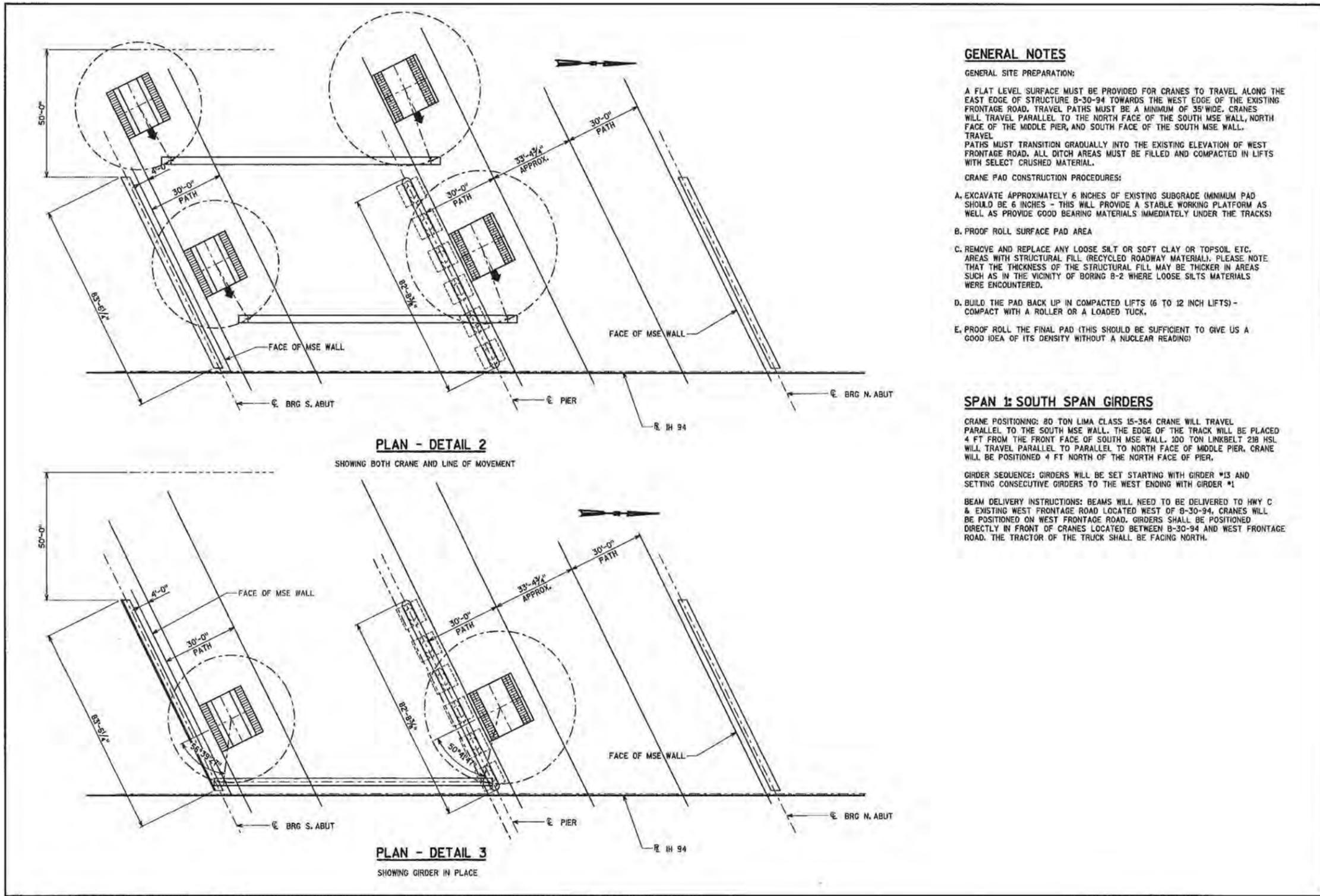


Figure 8-8 Concrete Girder Erection Plan 2 – South Span Girders Plan

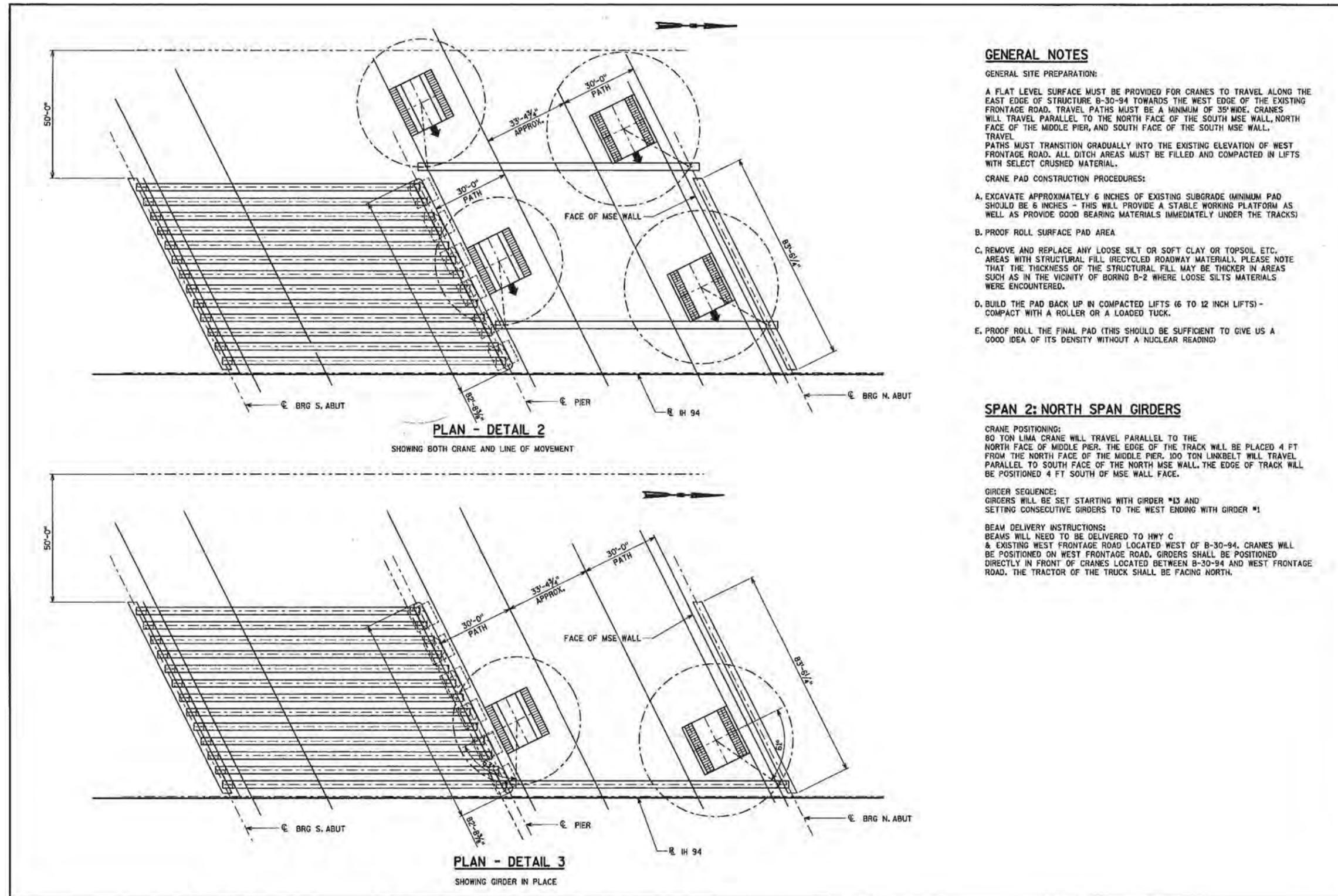


Figure 8-10 Concrete Girder Erection Plan 4 – North Span Girders Plan

8.3.2 General Plan

An overall plan of the project site including all existing conditions, topography, and the new structure should be provided (Figures 8-1 and 8-7). Any railroad tracks, waterways, and roads should be located, as well as any existing structures, and horizontal and vertical clearances shown. The general plan can be used to show girder delivery and lay-down locations. Girder delivery methods and piece weights should be coordinated with the fabricator or precaster. This drawing should also show equipment locations referenced to the bridge centerline and any substructures.

The general plan should also show the location of any buried or overhead utilities. Protection of underground utilities, if required, should be indicated. Operating clearances between crane booms, aerial lifts, and similar equipment and power/utility lines should be shown. In cases where utilities must be de-energized or temporarily relocated, the limits of the site associated with this action should be included.

General site topography should be included so that the location of site leveling for cranes or girder storage, or access road construction can be referenced to existing grades. When girder erection will be from barges operating on the waterway adjacent to the proposed bridge, the plan should include the waterway channel limits referenced to a given water elevation, waterway cross-sections showing channel profiles, and the expected water level during erection activities as well as water level variations. Barge mooring locations along with equipment locations on the barges should be provided.

8.3.3 Erection Sequencing

The erection plan should show the erection sequence for all members, both girders and bracing for the permanent structure, as well as any temporary bracing and shoring. Sequencing starts with the member delivery and is only completed for a member when it is placed and supported or braced as required to assure adequate strength and stability while controlling deformation within the limits that will allow fit-up of connections and provide a finished structure of proper alignment and elevation. The “Stages 1 and 2 Construction Plan” notes on Figure 8-1 provide an example of a step-by-step sequence from delivery to crane release.

The installation of all required temporary bracing and shoring must be specifically identified for each stage of setting girders; as may be seen in Figure 8-2. The “Stage 6 Construction Plan” notes in Figure 8-2, for instance, call for bracing of the beam to the south abutment and the temporary bent prior to releasing the crane. Member reference designations on the erection plans should be the same as those shown on the shop detail drawings. When holding cranes are used, the holding crane position, girder hold location and required holding load must be provided.

A contingency plan should be developed to supplement the erection sequence. The erection plan is based upon anticipated conditions during erection determined from local weather conditions, seasonal considerations, project staffing, material availability/supply

and other factors. Actual conditions or events, however, may differ and the contingency plan provides predetermined responses and identifies required resources to respond to unexpected events. As examples, an approaching storm may produce wind speeds in excess of those used in designing certain stage of erection, requiring installation of additional temporary bracing, or heavier than normal rains may increase river flows, increasing loads to temporary bents located in the river as well as delaying erection activities.

8.3.4 Calculations

Some bridge owners may prequalify engineers for erection engineering, require the erection engineering to be undertaken by a firm that is prequalified by the owner for design of the bridge category to be erected, or require the engineer's credentials be submitted by the erector along with his bid. These calculations are normally the general contractor's responsibility and may be prepared by someone other than the erection engineer. Computer generated calculations should be verified for selected conditions by hand methods.

Design calculations should address all critical stages of the erection sequence and substantiate the structural capacity and stability of the girders at each stage. Calculations should be referenced to the erection stage they address and use the member reference designations shown on the erection drawings. Girder capacity must be calculated using the spacing between bracing that matches the erection stage. This is particularly important where not all cross-frames or diaphragms are in place for an intermediate erection stage. Calculations must demonstrate the structural adequacy of any partially bolted primary splices after release from cranes or other external support.

The design calculations should also determine member deformations, particularly for curved or highly skewed (over 30 degrees) girders. Deformations must be limited such that connection fit-up of girder segments and bracing systems can be made without excessive forcing of members into alignment. In curved and skewed girder bridges, the differential displacements may cause difficulties with cross-frame installation. Girder elevations should be provided at splice locations and any shoring towers.

The determination of shoring loads must be based on maximum loads at the given location, which may vary for different erection stages. Tower design should conform to the AASHTO Specifications for Bridge Temporary Works. Shoring towers must be designed for wind loads as well as the girder and lateral brace reactions. When a deadman is used for guy cable or bracing anchorage, the design should consider worst case soil conditions, based on best available data in evaluating dead man resistance to uplift or sliding. In addition, provisions for cable tensioning should be determined considering cable elongation. Shoring design should consider the effect of tower shortening and settlement as well as lateral deflections on structure performance.

An assessment of the site soil's ability to support cranes and other equipment loads should be included, and when required, designs for crane pads should be provided. Crane, or other equipment, load effects on abutments, retaining walls, and underground

utilities must also be considered. This may require field investigations to verify structure or utility location, depths, and structural conditions. Site investigation data should be included with the calculations.

Calculations should also be provided to verify the capacity of contractor-fabricated rigging, such as lifting and spreader beams and lifting lugs. Catalog data should be included for manufactured items such as slings, clevises, chain-falls, concrete anchors, jacks, chain-binders, come alongs, etc. A comparison of available capacity for the manufactured item versus required capacity should be provided in the calculations.

When girders are to be erected from barge mounted cranes, calculations for barge stability and barge deck capacity should be provided. These are often prepared by a marine engineer/architect consultant. The calculations should clearly define any ballasting or deck modifications needed and define operating limits based on wind, wave, and current conditions.

If the contractor alters the bridge deck placing sequence, calculations verifying the adequacy of the bridge girders for the revised sequence must be prepared. While this work is not part of the true erection work, the associated calculations may be required along with the erection submittals. In addition to verifying the strength and stability of the bridge framing, these calculations should address any alterations to the concrete mix, such as the use of set retarding or other admixtures. Girder elevations and fillet height drawings or tables also should be prepared.

SECTION 4. GIRDER SETTING

Provisions in the erection plans for setting girders should include delivery locations and girder orientations. Crane locations for unloading, travel with load (if required), and setting are then developed and shown in the plans. Crane selection is normally performed by the erector, and while cranes must have adequate lift capacity and reach, availability and cost are often significant factors in final equipment selection.

The erection plans should include the lifting weight of each girder or girder segment and the location of lift points. Calculations for the center of gravity should be included, particularly for curved girders or if girders are to be set with cross-frames attached, and this should be clearly called out along with the resulting total assembly weight. The tabulated pick weights must include the weight of the hook block and all required below-the-hook rigging. The lifting radius and maximum radius for setting the girder should be shown in the erection drawings. This information is often presented in a table on the drawings.

Crane capacity charts for each crane to be used should be provided and clearly labeled to indicate the specific make and model of the crane, the counterweight configuration, boom length and type, and track or outrigger positions. For unusual lifting conditions, a trial lift may be made to verify equipment selection and rigging configurations. Equipment data and lifting charts are often submitted as attachments in the erection

procedure submittal and should include data for all rigging and below-the-hook devices. Crane locations on plan drawings show the center of rotation of the crane along with maximum lifting radius.

Loads on tracks or outriggers can be large. Figure 8-11 shows a typical ground pressure output for a crawler mounted crane with ground pressures exceeding 10,000 pounds per square foot for several lifting positions. Erection plans should state the required allowable soil bearing capacity and maximum applied loads. Crane mats, bearing pads under outriggers, as shown in Figure 8-12, and compacted crushed stone pads are often required to distribute crane, or other equipment, loads and must be shown on the drawings and detailed if used. Allowable soil pressures can be developed in accordance with the AASHTO Specifications, recognizing the short term nature and localized effects of the maximum crane loads.

For picks of single members using multiple cranes, the load distribution between cranes should be shown along with the method to monitor and control the distribution during lifting. The erection plans should include details of lifting and spreader beams, eveners, beam clamps, lifting slings, sling angles, and other lifting aids.

When girder erection will take place using barge mounted cranes, the erection plans should show barge/crane locations as well as any supply barges. Barge sizes, configurations and any deck modifications should be shown along with the method of maintaining barge position (spuds, anchors, combinations). If anchors are to be used, their location must also be indicated. The plans should include navigation channel limits, when applicable, and water depths. Barge stability requirements, such as which compartments are to be flooded, should be consistent with the calculations. Notes should include operational limits for wind, waves, and currents. A sample plan for a crane operating from a barge is shown in Figure 8-13, and the accompanying notes are shown in Figure 8-14. Note that in Figure 8-13 the front barge ballast tanks are required to be flooded for the crane position and usage shown. The notes in Figure 8-14 provide specific crane operating limits based on the barge stability calculations. Requirements and positions for tugs, pushers, or other support vessels should be shown along with locations of docks or moorings. Documentation that all required authorizations and permits have been obtained from the Coast Guard, US Army Corps of Engineers, and local agencies should be provided as part of the submittals.

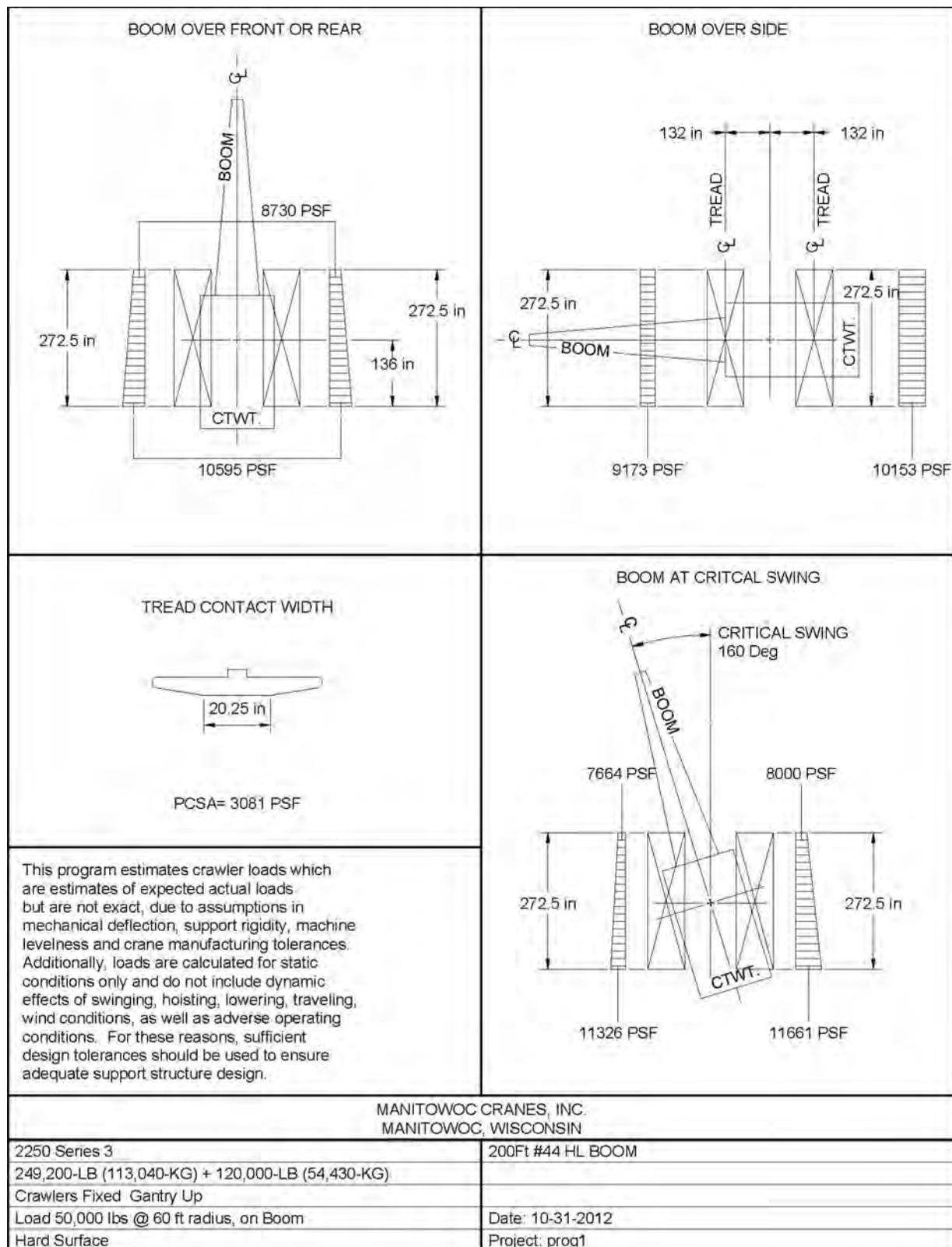


Figure 8-11 Crane Track Pressures for Crawler Crane



Figure 8-12 Truck Crane on Outriggers Supported on Timber Mats

NOTES:

1. LinkBelt 248 HYLAB with 180 foot Boom and Full Counterweight
2. Tanks 10, 11, and 12 Fully Ballasted
3. Pick Limits 360°:
 - 3.1. Min 35ft radius
 - 3.2. 50.0 kip @ 50ft
 - 3.3. 47.5 kip @ 55ft
 - 3.4. 45.0 kip @ 60ft
 - 3.5. 42.5 kip @ 65ft
 - 3.6. 40.0 kip @ 70ft
 - 3.7. 37.5 kip @ 75ft
 - 3.8. 35.0 kip @ 80ft
 - 3.9. 32.5 kip @ 85ft
 - 3.10. 30.0 kip @ 90ft
 - 3.11. 27.5 kip @ 95ft
 - 3.12. 25.0 kip @ 100ft
 - 3.13. 22.5 kip @ 105ft
 - 3.14. 20.0 kip @ 110ft
 - 3.15. 17.5 kip @ 115ft
 - 3.16. 15.0 kip @ 120ft
4. Pick Limits Over Front (Boom In—Between \odot Tracks):
 - 4.1. 65.0 kip @ 50ft
 - 4.2. 55.0 kip @ 60ft
 - 4.3. 50.0 kip @ 70ft
 - 4.4. 45.0 kip @ 80ft
 - 4.5. 35.0 kip @ 90ft
 - 4.6. 30.0 kip @ 100ft
 - 4.7. 25.0 kip @ 110ft
 - 4.8. 25.0 kip @ 120ft
5. All Materials Located in the Laydown Areas Must be Placed Symmetrically About the Longitudinal Axis of the Barge to Ensure an Overturning Moment is not Developed in the Transverse Direction of the Barge.

Figure 8-14 Plan Notes for Crane on Barge

SECTION 5. GIRDER STABILIZATION

Prior to release of the girder load by the lifting equipment, all provisions for temporary bracing and support must be in place. These stabilization provisions should be included in the erection plans, along with the sequence of installation. Figure 8-6 includes details for a temporary beam brace to restrain girder rotation, while Figure 8-5 shows a temporary shoring tower design, including support elevations.

Details of blocking required to temporarily limit movement of expansion bearings and multi-rotational bearings should be shown along with the specific bearing locations where they are to be installed. While the exact timing of this within the overall erection sequence may vary, the work must be completed prior to placing the girders.

Locations and details of all temporary support must be shown with the specific erection stage or sequence to which it applies. When holding cranes are used, the crane type, location and the required holding load need to be included, along with the lift point locations along the girder. The locations, required load capacity, and design details for any strongbacks, such as shown in Figure 8-15, should be provided, as well as the location of temporary shoring towers, as shown on the steel girder erection drawing plans, Figures 8-1 through 8-3.

Design drawings for any custom built shoring, Figure 8-5, should be included and show all details such as stiffeners, bracing connections and temporary bearings. When commercial shoring towers are used, the drawings should show the tower arrangement and overall dimensions, the configuration and details for header beams and associated framing, and required leg capacity along with manufacturer's product data. Tower foundation type (e.g., timber mats, precast concrete slabs, cast-in-place footings, and piles) along with required soil bearing capacity and foundation design details should be shown.

Figure 8-4 shows an example of steel H-piling used to construct a shoring tower. Tower heights and top of tower elevations are typically provided in tables. Methods for adjusting elevations and tower release procedures should be provided, and for commercial shoring, limits on the extension of screw height adjusters should be included as they can significantly affect tower leg capacity. The procedure for tower release must not result in uneven load distribution or deformations to the erected structure. Provisions for protecting towers from impact by construction operations or traffic should be shown on the drawings as well.



Figure 8-15 Precast Concrete Girder Supported on Strongback

Bolting requirements for girder splices and diaphragms or cross-frames are normally controlled by owner standard specifications. Any deviation from this practice (with owner approval) needs to be called out in detail, including connection location and specific bolting and pinning requirements, to include the number of bolts and/or pins in the connection as well as their location within the connection. Details must be shown for temporary girder end bracing or tie-downs. This includes product data and installation details for concrete anchorages. When these connections are made to permanent structures, methods for their removal and local repair details, if needed, should be provided.

SECTION 6. SUBMITTALS

The bridge erector or general contractor should submit the erection plans with supporting calculations and documentation to the bridge owner for review. Where third party review is required, for instance along some railroads, this review should be completed prior to submittal to the owner unless other procedures are agreed upon. Most bridges built are composed of straight girders of steel or concrete with spans

under 100 feet and on straight or slightly skewed alignments. While problems may occur with these “smaller” structures, erection submittal requirements should not be overly complex for the contractor, or require extensive review efforts by the owner. As an example, the Kansas Department of Transportation has three categories of bridge erection, based on bridge complexity, and each has an associated set of submittal requirements. Factors such as adjacent railroads, larger skew angles, or erection over in-service roadways might require more detailed submittals, even for smaller structures.

Recognizing differences in bridge complexity, two bridge classifications, Class A and Class B, each with its own submittal requirements, are discussed in this Manual. The two submittal classifications, that are presented in the following paragraphs, allow balancing submittal requirements with bridge complexity. Class A bridges are those meeting all of the following criteria:

- Span length less than 140 feet for steel girders or 120 feet for concrete girders (longest span)
- Maximum of one field splice in any span
- Girder radius of curvature is greater than 20 times the span length
- Skew angle less than 30 degrees
- Structure does not cross traffic lanes or railroad tracks
- No multi-crane lifts or erection from floating equipment (except standard two-crane PPC lifts)
- Shoring towers or strongbacks or hold cranes are not used

Bridges that do not meet the Class A criteria are considered Class B bridges. In addition, the owner may elevate a bridge to Class B based upon engineering judgment or unique girder erection conditions. While permits and approvals from railroads, Coast Guard, U.S. Army Corps of Engineers, etc. are listed under Type B bridges, all applicable permits must be provided to the owner regardless of the erection scheme, and may be provided separately from the erection procedures. Permits may have expiration dates and require periodic renewal. The minimum submittal requirements for each bridge class are as follows:

Class A bridge:

- Erection plans showing site conditions, crane positions, and girder splice locations
- Girder transportation plan
- List of all equipment to be used
- List of members/parts with weights
- Crane locations and lift weights

- Crane configuration and lifting charts
- Erection sequence
- Temporary bracing requirements (provide sketches as a minimum)
- Timing of bracing installation
- Bolting sequence and minimum number of bolts and pins for each splice
- Name and credentials of the erection superintendent who will be in charge of the field activities
- Erection schedule
- Traffic control plan
- Contingency plan

Class B bridge: All information for a Class A bridge, plus:

- Drawings of all girder or girder assembly placements with rigging requirements and lift calculations
- Drawings for all shoring towers, strongbacks and temporary bracing including, foundations, mats, etc.
- Details for any bearing restraints or blocking
- Removal procedures and any repairs for any temporary bracing, anchorage, bearing restraints, etc.
- Manufacturer's data sheets for all rigging devices, hooks, blocks, pre-engineered shoring, anchorage devices
- Methods of protecting shoring towers against accidental impact
- Copies of permits and approvals from railroads, Coast Guard, U.S. Army Corps of Engineers, property owners, etc.
- Calculations substantiating structural integrity and stability of members at each stage of erection. Compilation of all design loads and associated deflections.
- When drop-in girders are used, calculations for temperature effects for fit-up and stresses due to any required jacking forces. Include details for jacking and applied load tables.
- Calculations substantiating structural integrity of partially bolted connections
- Calculations substantiating the structural integrity of all bridge abutments, retaining walls, and underground utilities affected by crane or other equipment loads
- Calculations and revised girder elevation and fillet height drawings in the event an alternate deck placing sequence is proposed. If concrete is modified with retarders or other admixtures, mix design data should be included.

- When jacking is used, provide tables of load, jack pressure, and associated movement/jack elongations.
- Stressing procedures, loads, elongations, and sequencing for post-tensioning.
- Manufacturer's data and load calibration data for jacks used for stressing or lifting/pushing.
- Drawings should include general notes for materials and erection operations. Design wind, live, and dead loads should be listed as well as design soil bearing capacities.

Erection plans and calculations should be prepared under direction of a Licensed Professional (or Structural) Engineer and should be sealed prior to submittal. Preferably for all projects, but certainly for Class B bridges, a meeting of the parties involved should be held to review erection procedures.

Submittals of erection plans and supporting documents should be reviewed by the bridge owner or an engineering firm, generally the bridge design consultant under contract to the owner. Review requirements and schedules are normally stipulated in the standard specifications, which may sometimes be modified for larger or more complex projects by special contract provisions.

The review should be performed by staff, or owner's consultant, with experience in bridge design and construction practices. The Engineer of Record may perform the review since he/she will be aware of the design requirements of the project. Normally, the review does not include a complete check of drawings or calculations. The review may address areas affecting public safety, assure that proper Quality Assurance/Quality Control procedures have been followed, and identify conditions that have caused problems in past projects.

There may be situations where project design criteria or constraints that are not apparent to the erector or the erector's engineer may have significantly influenced project design and require modification of erection plans. All review comments should be provided to the erector/contractor for resolution. Upon resolution, a final erection submittal should be made, and serve to control erection. Any changes to the accepted erection sequence require developing supporting documentation as well as resubmittal and review prior to undertaking further erection activities.

SECTION 7. ERECTION CHECKISTS

The following checklist may be used as an aid in reviewing bridge erection submittals and assuring completeness. The applicability of the specific items will vary based on the bridge complexity and owner requirements.

Item	Included	N/A
• Topography	<input type="checkbox"/>	<input type="checkbox"/>
• Structure framing plan with members labeled	<input type="checkbox"/>	<input type="checkbox"/>
• Field splice locations	<input type="checkbox"/>	<input type="checkbox"/>
• Member delivery location and orientation	<input type="checkbox"/>	<input type="checkbox"/>
• Adjacent structures	<input type="checkbox"/>	<input type="checkbox"/>
• Underground utilities/structures	<input type="checkbox"/>	<input type="checkbox"/>
• Overhead utilities	<input type="checkbox"/>	<input type="checkbox"/>
• Roads, railroads, waterways	<input type="checkbox"/>	<input type="checkbox"/>
○ Locations	<input type="checkbox"/>	<input type="checkbox"/>
○ Clearances	<input type="checkbox"/>	<input type="checkbox"/>
○ Traffic Control	<input type="checkbox"/>	<input type="checkbox"/>
• Crane locations	<input type="checkbox"/>	<input type="checkbox"/>
○ Crane type	<input type="checkbox"/>	<input type="checkbox"/>
○ Lift radius	<input type="checkbox"/>	<input type="checkbox"/>
○ Capacity	<input type="checkbox"/>	<input type="checkbox"/>
○ Work pads, foundations	<input type="checkbox"/>	<input type="checkbox"/>
○ Travel paths	<input type="checkbox"/>	<input type="checkbox"/>
• Temporary support locations	<input type="checkbox"/>	<input type="checkbox"/>
○ Design load	<input type="checkbox"/>	<input type="checkbox"/>
○ Required capacity	<input type="checkbox"/>	<input type="checkbox"/>
○ Elevations	<input type="checkbox"/>	<input type="checkbox"/>
• Holding cranes	<input type="checkbox"/>	<input type="checkbox"/>
○ Crane type	<input type="checkbox"/>	<input type="checkbox"/>
○ Location	<input type="checkbox"/>	<input type="checkbox"/>
○ Hold load and radius	<input type="checkbox"/>	<input type="checkbox"/>

Figure 8-16 Checklist: Plan Drawings (to Scale) for Each Erection Stage

Item	Included	N/A
<ul style="list-style-type: none"> • Member rigging <ul style="list-style-type: none"> ○ Load capacity ○ Arrangement and component sizes ○ Weights ○ Center of gravity of lifted piece 	<input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/>	<input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/>
<ul style="list-style-type: none"> • Crane lifts at each stage <ul style="list-style-type: none"> ○ Lift load including rigging ○ Lift radius ○ Crane configuration/boom length 	<input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/>	<input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/>
<ul style="list-style-type: none"> • Falsework <ul style="list-style-type: none"> ○ Location ○ Applied loads/required capacity ○ Tower design details ○ Foundation requirements ○ Elevations ○ Method for release 	<input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/>	<input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/>
<ul style="list-style-type: none"> • Temporary bracing <ul style="list-style-type: none"> ○ Location ○ Member sizes ○ Connection details ○ When installed/removed ○ Bearing blocking details 	<input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/>	<input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/>

Figure 8-17 Checklist: Detail Drawings

Item	Included	N/A
<ul style="list-style-type: none"> • Erection sequence for each member <ul style="list-style-type: none"> ○ Primary members ○ Secondary members 	<input type="checkbox"/>	<input type="checkbox"/>
<ul style="list-style-type: none"> • Temporary tie-down 	<input type="checkbox"/>	<input type="checkbox"/>
<ul style="list-style-type: none"> • Time requirements for bracing installation 	<input type="checkbox"/>	<input type="checkbox"/>
<ul style="list-style-type: none"> • Temporary bracing installation/removal 	<input type="checkbox"/>	<input type="checkbox"/>
<ul style="list-style-type: none"> • Splice and connection bolting data <ul style="list-style-type: none"> ○ Number of bolts/pins ○ Location of bolts/pins 	<input type="checkbox"/>	<input type="checkbox"/>
<ul style="list-style-type: none"> • Contingency bracing/wind speed limits 	<input type="checkbox"/>	<input type="checkbox"/>
<ul style="list-style-type: none"> • Temporary bracing <ul style="list-style-type: none"> ○ Location ○ Member sizes ○ Connection details ○ When installed/removed ○ Bearing blocking details 	<input type="checkbox"/>	<input type="checkbox"/>
<ul style="list-style-type: none"> • Barges <ul style="list-style-type: none"> ○ Configuration, barge data ○ Applied loads ○ Crane locations and lift data ○ Spuds/anchoring arrangements ○ Storage areas and capacities 	<input type="checkbox"/>	<input type="checkbox"/>
<ul style="list-style-type: none"> • Temporary jacking <ul style="list-style-type: none"> ○ Location ○ Jack data ○ Maximum applied loads 	<input type="checkbox"/>	<input type="checkbox"/>

Figure 8-18 Checklist: Written Procedures and/or Drawing Notes

Item	Included	N/A
• Design criteria provided	<input type="checkbox"/>	<input type="checkbox"/>
• Level of analysis is appropriate	<input type="checkbox"/>	<input type="checkbox"/>
• Stage-by-stage analysis used	<input type="checkbox"/>	<input type="checkbox"/>
• Support conditions used in modeling match erection conditions	<input type="checkbox"/>	<input type="checkbox"/>
• Correct loads utilized	<input type="checkbox"/>	<input type="checkbox"/>
• Member strength verified for each erection stage	<input type="checkbox"/>	<input type="checkbox"/>
• Member/system stability verified for each erection stage	<input type="checkbox"/>	<input type="checkbox"/>
• Second-order amplification effects addressed	<input type="checkbox"/>	<input type="checkbox"/>
• Girder support reactions computed for worst case load configurations	<input type="checkbox"/>	<input type="checkbox"/>
• Bearings verified for load capacity and rotation	<input type="checkbox"/>	<input type="checkbox"/>
• Center of gravity and lifting loads computed for each piece/assembly	<input type="checkbox"/>	<input type="checkbox"/>
• Temporary bracing and hold down design included <ul style="list-style-type: none"> ○ Strength ○ Stiffness ○ Connection details 	<input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/>	<input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/>
• Temporary shoring system design <ul style="list-style-type: none"> ○ Loads ○ Settlement/deflection ○ Foundation 	<input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/>	<input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/>
• Calculations for integrity of existing/adjacent structures due to crane and equipment loads <ul style="list-style-type: none"> ○ Abutments ○ Wingwalls / retaining walls ○ Slopes ○ Underground utilities/structures 	<input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/>	<input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/>
• Barge stability calculations	<input type="checkbox"/>	<input type="checkbox"/>
• Calculations for the deck placement if altered from the Contract Documents <ul style="list-style-type: none"> ○ Member capacity verifications ○ Member stability verifications ○ Bearing reactions; uplift ○ Girder deflections ○ Deck fillet heights 	<input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/>	<input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/> <input type="checkbox"/>

Figure 8-19 Checklist: Calculations at all Stages of Erection

Item	Included	N/A
• Project superintendent's name and experience	<input type="checkbox"/>	<input type="checkbox"/>
• Documents sealed by Professional Engineer	<input type="checkbox"/>	<input type="checkbox"/>
• Manufacturer's data for all supplied devices and equipment	<input type="checkbox"/>	<input type="checkbox"/>
○ Crane load charts	<input type="checkbox"/>	<input type="checkbox"/>
○ Beam clamps	<input type="checkbox"/>	<input type="checkbox"/>
○ Slings, shackles, turnbuckles, chain, straps, etc.	<input type="checkbox"/>	<input type="checkbox"/>
○ Jacks	<input type="checkbox"/>	<input type="checkbox"/>
○ Spreader and lifting beams	<input type="checkbox"/>	<input type="checkbox"/>
○ Pre-engineered shoring systems	<input type="checkbox"/>	<input type="checkbox"/>
• Documentation for third party reviews / coordination	<input type="checkbox"/>	<input type="checkbox"/>
○ Railroads	<input type="checkbox"/>	<input type="checkbox"/>
○ Utilities	<input type="checkbox"/>	<input type="checkbox"/>
○ Coast Guard	<input type="checkbox"/>	<input type="checkbox"/>
○ Corp of Engineers	<input type="checkbox"/>	<input type="checkbox"/>
○ Other regulatory / governmental agencies	<input type="checkbox"/>	<input type="checkbox"/>
• Girder transportation and shipping plan and supporting calculations	<input type="checkbox"/>	<input type="checkbox"/>

Figure 8-20 Checklist: Additional Submittal Data

SECTION 8. BRIDGE REMOVAL PLANS

Plans and procedures for bridge removal projects should be generally consistent with those discussed for erection. The extent of information to be included will vary with the project type and extent. For instance, requirements for a project for concrete deck removal and widening need to assure that the existing girders, as well as the new girders, possess adequate strength and stability in the conditions with the deck removed, once compression flange lateral restraint is removed. While girder rotations at connections during erection need evaluation to assure member fit-up, this is not a concern when girders are removed.

Dividing removal projects into the same categories as shown in Section 7 is recommended, and the Erection Checklist may also be used, noting that the erection supervisor would now be the demolition supervisor. Among items that may not be included in the erection checklist that would apply to demolition are the following:

Item	Included	N/A
• Measures to protect existing facilities from falling debris	<input type="checkbox"/>	<input type="checkbox"/>
• Equipment and method of debris removal	<input type="checkbox"/>	<input type="checkbox"/>
• Location for debris disposal	<input type="checkbox"/>	<input type="checkbox"/>
• Disposal and transportation permits	<input type="checkbox"/>	<input type="checkbox"/>
• Protection of waterways	<input type="checkbox"/>	<input type="checkbox"/>
○ Pre-demolition and post-demolition surveys	<input type="checkbox"/>	<input type="checkbox"/>
○ Erosion and contamination	<input type="checkbox"/>	<input type="checkbox"/>
• Noise pollution	<input type="checkbox"/>	<input type="checkbox"/>
• Effects of equipment operating from structure during demolition	<input type="checkbox"/>	<input type="checkbox"/>
• Impact loads to the structure from removal activities	<input type="checkbox"/>	<input type="checkbox"/>
• Dust control	<input type="checkbox"/>	<input type="checkbox"/>
• Water/runoff control	<input type="checkbox"/>	<input type="checkbox"/>

Figure 8-21 Checklist: Demolition Checklist

SECTION 9. SUMMARY

The erection plans and procedures describe the means and methods to be used by the contractor in erecting or demolishing the bridge superstructure. They must assure that sufficient strength and stability is maintained at each stage of erection and also provide contingencies for unexpected events. They not only guide the contractor, but also provide information for the Owner's review. The documents must be complete and include supporting engineering calculations.

In order to match erection document requirements to bridge complexity, two bridge submittal classifications, A and B, are presented. Owners may, of course, have other requirements. Checklists are included in Chapter 8 which can assist in assuring complete documents preparation as well as their review.

CHAPTER 9

MAJOR AND UNUSUAL BRIDGE CONSTRUCTION

SECTION 1. INTRODUCTION

While girder bridges are the predominant type of bridge constructed, other bridge types are used where economic or aesthetic considerations make them advantageous. Stability during erection must be considered for all bridge types, and this chapter discusses some of the erection related stability issues for several of these. The in-depth evaluation of major and unusual bridges for erection conditions is beyond the scope of this Manual. However, many of the guidelines and practices outlined in this Manual will also be applicable to them. Loads and load combinations may follow those for girder bridges. Also, many of the other bridge types still utilize girders and bracing within the overall structure, and the stability of these members must be addressed for construction conditions. Unlike many girder bridges, provision for bridge erection is normally part of the initial bridge design for more complex structures, and for bridges such as cable stay configurations may often control bridge design.

SECTION 2. ARCH BRIDGES

Arch bridge structures have evolved greatly from the masonry construction still in existence from as early as 1300 BC to the state of the art structures seen today for many medium to long span applications. Today, arch bridges are predominantly constructed of steel, reinforced concrete and pre-stressed/post-tensioned concrete, and are erected in two primary configurations: tied arches and true arches, as shown in Figure 9-1.

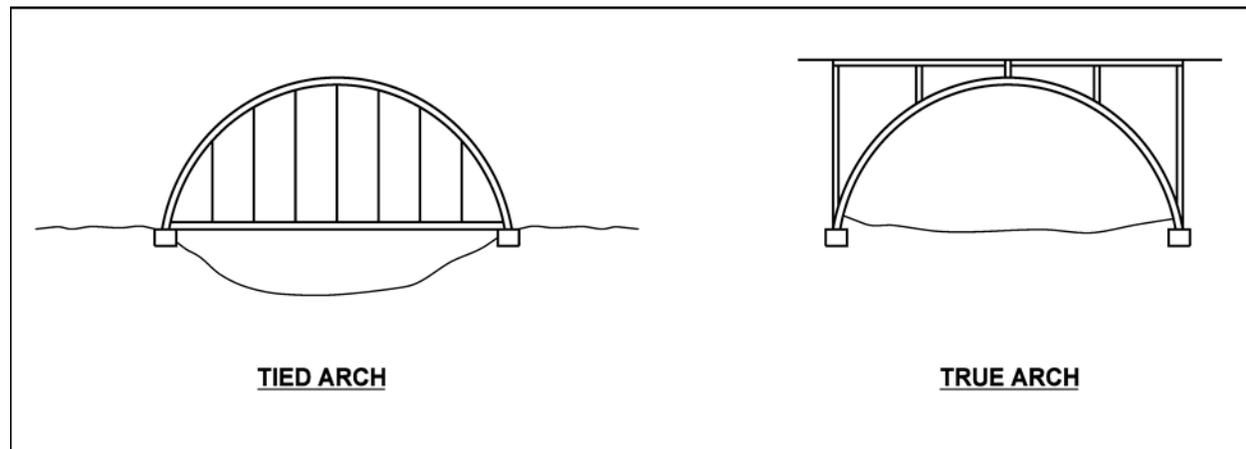


Figure 9-1 Arch Bridges – Tied and True Types

In a tied-arch bridge, the bottom chord member “ties” the two ends of the arch rib together to resist the outward thrust at the base of the arch through tension and keeps the arch element in compression. In a true arch, the thrust at the base of the arch is resisted by the mass of the bridge’s foundations at the abutments or piers to keep the arch in compression. In addition to the AASHTO Specifications, guidance on arch design is contained in publication FHWA-NHI-11-023, Design Guidelines for Arch and Cable-Supported Signature Bridges.

9.2.1 Steel

Steel is a popular and economical component for arch bridges and can be used in either a trussed or solid-ribbed arch configuration. The trussed arch uses an arch rib that is built-up in the form of a truss.

In some instances, particularly for short spans, a tied arch bridge can be erected in one piece under the proper conditions. In most instances, tied-arch bridges are constructed in place through a sequence of carefully designed and planned erection stages. Figure 9-2 illustrates the erection stages for an arch where the tie girder is supported on falsework.

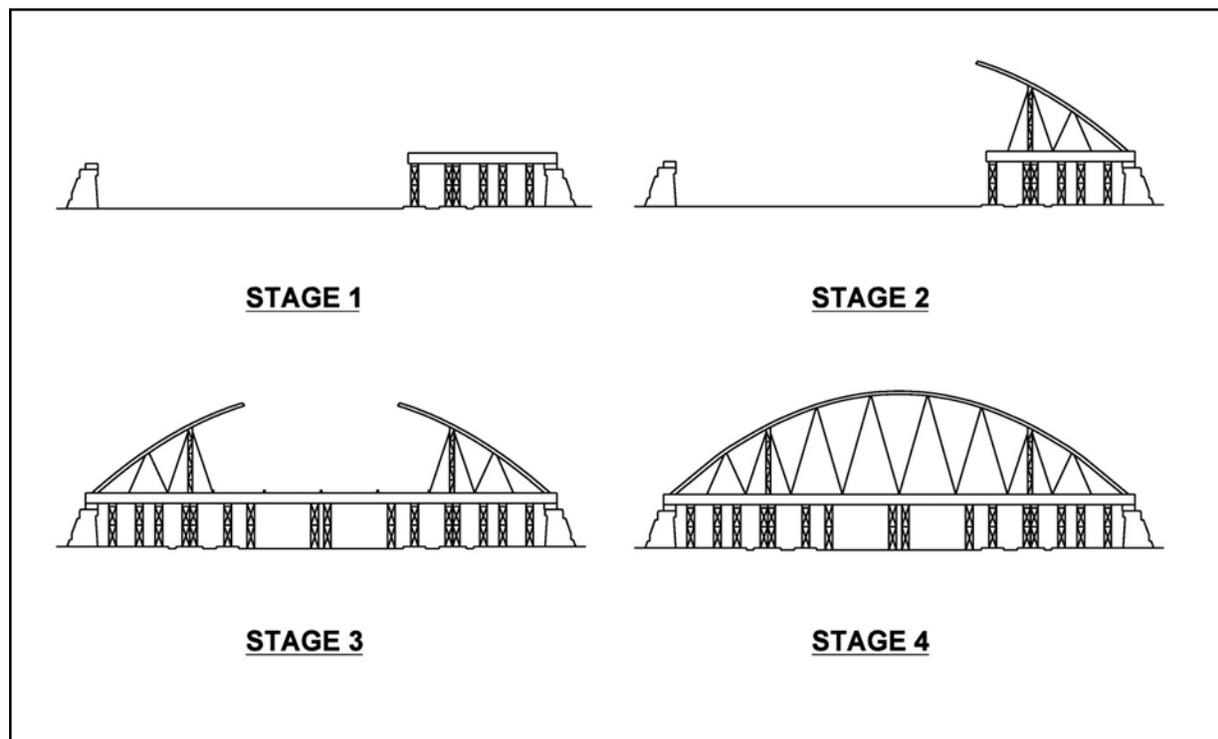


Figure 9-2 Arch Bridge Erection Staging on Falsework



Figure 9-3 Erection of Tied Arch Bridge Using Falsework

Tied arch bridge erection begins by setting the tie girder sections and end cross girder to which the arch ribs attach. The bearings must be blocked both for rotation and to provide thrust restraint, unless the tie girder is fully installed prior to starting the arch. Since the tie girder is designed primarily as a tension member, it must be temporarily supported until the hangers are installed, and must be investigated for biaxial bending due to dead loads, any construction loads, and wind.

Arch segments and tie girders can be supported from below by falsework (Figure 9-2), or suspended from cable stays running from temporary towers. In either case, once the arch section is positioned, it acts like a vertically curved beam spanning between temporary supports under both self-weight and lateral wind loads. The lateral wind loads must be transferred into the temporary support towers or through angled stay cables. Methods of adjusting the sections vertically at support towers through jacking are usually provided, and local buckling effects at support points must be examined. Figure 9-3 shows an arch being erected utilizing falsework for tie-girder and arch temporary support. Arches are built from both abutments toward the crown and a closure arch segment completes the arch. Sufficient gap must exist to allow inserting this segment, and provisions are often included to jack open the gap to aid in fit-up. Local buckling of members due to jacking should be investigated and the effect on the overall arch

considered; though this is normally not a problem as the required arch spread is very small compared to its span. For short arches though, the stresses should be investigated.

Prior to releasing any shoring, the arch lateral bracing must be in place. Floor beam camber can cause the arches to rotate slightly outward prior to the deck placement, making it necessary to pull the crown of the arches together in order to make the top strut connections.

In placing and tensioning hangers, the loads to the arch must be kept symmetrical. Arch stability is reduced when non-symmetric moments are applied. Local stresses in the arch as well as the tie girder webs and flanges due to tensioning need to be evaluated for the specific jack stands to be used as these may differ from that assumed during the original design.

A deck arch is a true arch with a deck built above the arch. Construction is carried out in much the same manner as a true arch except no tie girder is present. Setting of columns and deck framing should progress in a manner that produces symmetrical moments in the arch. Temporary bracing of the columns for wind and general stability will be required until the deck and permanent lateral bracing is in place.

Erection of precast arch bridges is performed in a manner similar to steel arches of the same configuration. It is critical that field connections achieve adequate strength at each stage of construction.

Cast-in-place concrete arches, and any cast-in-place portions of precast arch bridges, are supported by falsework designed in accordance with the AASHTO Specifications for Bridge Temporary Works.

SECTION 3. TRUSSES

Truss bridges have been used throughout history. The development of railroads increased truss construction, starting with timber truss bridges in the early 1800's and continuing with steel in the late 1800's.

A truss is a system of members comprising a series of triangles. A truss distributes the load by creating tension and compression in the members rather than bending. Trusses can be advantageous due to their high strength-to-weight ratio, ability to be prefabricated, and ability to create long spans.

Truss floor systems consist of rolled beams or built-up girders. Their stability during deck placement should be verified based on flange bracing locations as discussed elsewhere in this Manual.

There have been many types of truss configurations used and patented over the years. The most common truss type seen in modern bridge construction is the Warren Truss

patented by James Warren in 1848. The Warren Truss consists of equilateral triangles between a top and bottom chord and is often seen with and without vertical members. Other typical truss types include Howe Truss, Pratt Truss, and Lattice Truss. Figure 9-4 provides schematic representations of the Warren, Howe, Pratt, and Lattice trusses.

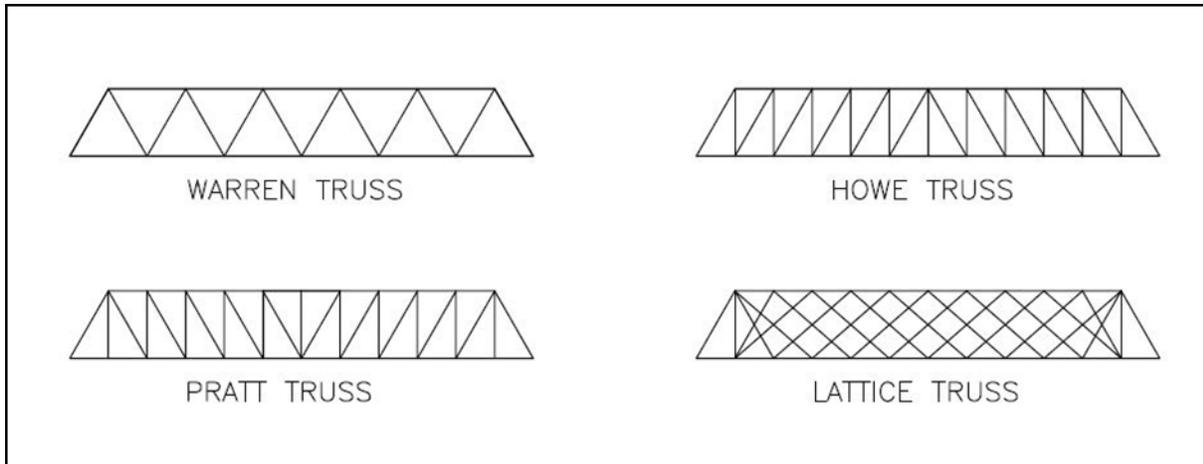


Figure 9-4 Typical Truss Configurations

The erection methods commonly used in the construction of truss bridges include ‘stick building’ the truss in its final position or pre-assembly of the truss on-site and lifting or sliding the truss into its final position. The erection method utilized is typically determined by the structure’s size, weight, and geometry, construction cost, and site conditions.

9.3.1 Stick Built Truss Erection Analysis

When a truss is erected one piece at a time, it is called stick built. Typically, lower chords are set on substructure units or falsework, and vertical, diagonal, and upper chord members are installed piece by piece to complete sections of the truss. After sections of the truss have been completed between panel points to allow for truss action within the members, piece by piece erection of the truss continues by cantilevering the truss past the substructure units and falsework. Depending on the length of the cantilever, falsework or temporary suspension cables may be required to support the truss at intermediate panel points during erection. Generally, a stick built truss will involve erecting multiple portions of the truss simultaneously until they meet and the truss can be closed.

As with any structure type, ensuring the strength and stability of the components while in an incomplete stage of erection is essential, and truss bridges require significant investigation to determine the best erection sequence. The truss structure must be analyzed during each stage of construction for the dead load of the truss members, construction live loads, and environmental loads such as wind and thermal forces.

When analyzing the structure during each erection stage, the following items should be investigated to ensure the strength and stability of the individual truss members and the structure as a whole:

- Analyze truss members and ensure demand does not exceed capacity
- Analyze the structure for lateral loading
- Analyze truss members for concentrated loads
- Analyze bearing capacity
- Verify deflection is controlled to allow proper fit-up for subsequent stages of erection

Often times, the loads induced during erection may create tension or compression forces in the members that are greater than or opposite to the forces anticipated during design for the completed structure. If a compression member is overstressed during erection, temporary bracing can be installed to reduce the member's effective length and increase capacity. For overstressed tension members or when bracing is not a viable solution for compression members, the cross-sectional properties of the member can be increased or the erection sequence can be revised. For diagonal members that cantilever from the completed portion of the truss prior to erection of the upper chord, temporary cable supports can be installed to support the cantilevered end of the diagonal member. Falsework or temporary suspension cables can also be provided during erection to support the truss at intermediate points during erection and reduce demand on the truss members (Figure 9-5).

During each stage of erection, the structure should be analyzed for lateral loading including wind, thermal and/or seismic forces. Temporary bracing may be required during the first stages of erection before permanent lateral bracing, struts, floor beams, and portals are installed and can adequately brace the structure against lateral loads. Additionally, consideration should be given to the amount of the truss structure that is erected beyond the lateral bracing, struts, floor beams, and portals that have been installed. Typical practice is to install these secondary members as soon as sections of the truss have been completed and prior to additional erection of the primary truss members.

Jacks are typically located at falsework locations to allow for vertical adjustment of the truss lower chord to ease erection fit-up. At jacking locations and at falsework bearing locations, concentrated forces are applied directly to truss members and these members must be analyzed for local buckling, yielding, and crippling. To reduce the localized loading on the truss members, it is recommended that the falsework and jacks be located directly beneath panel points of the truss, which will distribute the concentrated loads to the vertical and diagonal members framing into the panel point.



Figure 9-5 Temporary Kingpost Supporting Cantilever Construction of Bridge Truss Over the Mississippi River

Permanent truss bearings should be analyzed during each erection stage to ensure both the strength and rotational capacity of the bearings are not exceeded. Often times, vertical loads during erection are substantially less than design loading. However, lateral capacity and rotational capacity of the bearing can be critical and should be investigated. Additionally, consideration should be given to whether permanent expansion bearings need to be temporarily blocked prior to truss closure to ensure lateral stability of the structure.

As the partially completed truss structure cantilevers from the substructure units and falsework, the existing structure should be analyzed for potential uplift at the support locations. It is important to investigate the effects of wind and thermal loading during the analysis. If uplift does occur at the supports, counter-weighting of the structure or tie-downs should be designed and installed, which will also assist in the control of deflection during erection.

During all stages of erection, the deflection of the individual truss members and the structure as a whole should be investigated. It is important to minimize deflections at

each stage of construction to ease fit-up of the truss elements and reduce the time required to fit-up truss member connections. Where required, temporary bracing or cable supports can be installed to minimize deflection.

9.3.2 Pre-Assembly Erection Analysis

Depending on the size and weight of the truss and the site conditions, the entire truss structure or portions of the truss structure can be pre-assembled on-site and lifted or slid into their final position. Examples of this method include pre-assembling portions of a truss structure over a waterway on barges and lifting the truss vertically into final position and pre-assembling portions of the truss on structure approaches and launching the truss horizontally. Ensuring the strength and stability of the components while in an incomplete stage of erection is essential. The truss structure must be analyzed during each stage of construction for the dead load of the members in place, construction live loads, and environmental loads such as wind and thermal forces.

When analyzing the structure, the following items should be investigated to ensure the strength and stability of the individual truss members and the structure as a whole:

- Analyze truss members and ensure demand does not exceed capacity
- Analyze the structure for lateral strength and stability
- Analyze truss members for concentrated loads

In some cases, only a portion of the structure is pre-assembled and lifted or slid into place. Additionally, the support or falsework locations are typically not located in the same positions as the truss support in the final design. Therefore, the loads induced during erection may create tension or compression forces in the truss members that are greater than or opposite to the forces analyzed for the completed structure during design. If it is determined that truss members are overstressed during erection, temporary bracing can be installed, the cross-sectional properties of the member can be increased, or the erection plan can be revised.

During lifting or sliding of the truss, it is important to analyze the structure for lateral loading. Temporary bracing or tie-downs of the truss and falsework may be required to stabilize the structure as a whole.

At jacking locations, concentrated forces are applied directly to the truss members, which must be analyzed for local buckling, yielding, and crippling. Where possible, it is recommended that the jacks be located at panel points of the truss, which will distribute the concentrated loads to the vertical and diagonal members or floor beams framing into the panel point and significantly reduce the localized loading on the truss members.

SECTION 4. CABLE-STAYED BRIDGES

The cable-stayed bridge has emerged as one of the more dominant mid to long span bridge types used today. The structure type is defined by having at least one tower, or

pylon, extending from the bridge's foundation and from which cables are attached to support the bridge deck. They are commonly used to accommodate clear span needs of anywhere from 400 feet to 2000 feet.

Design considerations for cable-stayed bridges are included in the FHWA-NHI-11-023 publication, previously referenced. Construction practices for these bridges are presented in the Construction Practices Handbook for Concrete Segmented and Cable-Supported Bridges, which was developed by the American Segmental Bridge Institute (ASBI).

SECTION 5. BRIDGE LIFTING AND MOVING

Lifting and moving of bridge structures may involve partially, as well as fully, completed bridges. General stability concerns for girders are similar to those during normal erection, and when the deck is in place during the lifting or moving operation, a laterally stiff structure is developed.

Overall effects of lifting and moving are generally related to changes in support locations from those for the bridge in its final position. This change in support location can cause stress reversals in primary members, and as a result, distances between temporary and permanent support locations should be the minimum consistent with clearances and access for moving.

Local stresses at jacking or lifting locations must be examined, particularly for web buckling or crippling. Stiffeners are often added even when not required for member capacity in order to accommodate tolerances in jack location. Jacking systems must be designed to ensure controlled lift pressures among all locations and provide for read-out of loads and pressures during jacking. When a structure is moved after vertical jacking, blocking should be used to support the loads rather than the jacks, or locking jacks secured against lateral movement must be used. When structures are moved laterally by jacks or pullers, guides should be used to control lateral movement and provisions for lateral load distribution between components considered. The capacity of diaphragms or cross-frames may be insufficient if placed under lateral load.



Figure 9-6 New Truss on Temporary Trestle for Slide-In

Structure lifting must not induce differential vertical deflection, or overall structure misalignment, as it will cause stress redistributions. When bridges are moved longitudinally (Figure 9-6), axial loads may be induced into girders, which increase compressive stresses. Though slide plates and rollers minimize these forces, a conservative estimate of their value should be used to account for uncertainties in field operations. Temporary bents used to support structures during lifting or sliding operations should be designed for worst case vertical and lateral loads.

SECTION 6. MOVABLE BRIDGE STABILITY



Figure 9-7 Initial Balancing of the Congress Avenue Bridge over the Chicago River

Movable bridges are essentially very large machine structures that rely on balance to safely operate. The most common types of movable bridges are vertical lift bridges (bridges where the movable span moves up and down vertically much like an elevator) and bascule type bridges (bridges where the movable leaf rotates either about a fixed point or rolls back on itself). Maintenance and construction on a movable bridge can alter the balance of a movable bridge which can greatly affect the safe operation of the bridge.

Safe construction and maintenance for a movable bridge requires the contractor's construction procedures to be engineered to ensure the safety and stability of the bridge

through all steps of the construction. Even small changes in the weight of the movable leaf or counterweight can cause the movable span or leaf to become uncontrollable, resulting in serious damage to the bridge and possible injury to those working on the bridge.

Because of the complex interactions of machinery, operating systems and weight, owners often specify that the contractor hire a Movable Bridge Systems Coordinator (MBSC) to oversee the work. The MBSC's role is to ensure that all systems are fully integrated and coordinated to verify the safe performance of the work and operation of the bridge upon completion of the work. The MBSC should be an experienced Licensed Professional or Structural Engineer knowledgeable in the design and rehabilitation of movable bridges.

The individual structural components of the bridge include girders or truss members that must be analyzed to ensure strength and stability of the members before and during deck placement. In reconstruction, deck removal may produce long unbraced flange lengths. As seen in Figure 9-8, construction and rehabilitation may take place with the bridge in its "up" position, resulting in increased wind loads to the open structure.

Key elements to ensure movable bridge stability during construction include the following:

- **Bridge Lockout.** The safest means to work on a movable bridge is to lock the bridge leaf in the open position. For bascule type bridges, this is accomplished through the use of steel struts to wedge the leaf in the open position. The struts are typically designed to resist wind forces on the leaf, other construction loads, and the horizontal component of the weight of the span. For vertical lift bridges, typically additional weight is added to the counterweight block to ensure that the counterweight is always heavier than the span being worked on. Where it is not possible to lock the movable leaf or span in the open position due to traffic or other constraints, other means of stabilizing the movable span must be employed to prevent movement during the construction work. For bascule type bridges, this often means the installation of shoring towers to support the counterweight during construction. These shoring towers are designed to carry all or a portion of the weight of the counterweight. If river traffic prevents shoring of the movable leaf or span, then strictly maintaining the balance of the movable leaf is essential for the safe completion of the work. Motors and bridge controls are also positively locked out to prevent operation during conditions of unbalance or when work is occurring on the bridge. This is typically accomplished through the lock out of the knife switches or incoming power. Additional measures may also include lashing and blocking of the gear train to prevent movement.
- **Bridge Balance.** Determining the initial state of balance is a critical first step for working on a movable bridge. Often, the state of balance is unknown. Older bridges may not have had balance calculations performed. For direct current drive bridges, motor amp readings can provide a simple means of determining

balance. Motor amperage (amp) readings should be nearly the same during opening and closing when the bridge is in balance. Higher amp readings during opening indicate a span heavy condition. Higher amp readings during closing indicate a counterweight heavy condition. Amp readings for alternating current bridge drives, particularly the more sophisticated Programmable Logic Circuitry (PLC) drives, cannot be relied upon to determine balance. In these cases, strain gage testing of the bridge operating machinery is often the only reliable means of determining the state of balance of the bridge.

Regardless of the bridge type or position of the movable leaf during work, it is essential that accurate records be kept of the weight and location of the material being removed and added to the movable span and counterweight. After determining the initial balance of the bridge, a spread sheet is usually prepared that documents the weight and location of the materials removed and added to the bridge. The center of gravity of each member is determined and measured in the horizontal, vertical, and transverse direction in order to develop moments about a fixed point. A movable leaf is considered in balance when the sum of the moments of the movable leaf and counterweight is zero.

- **Structural Modeling.** Special care must be taken when performing work on the main load carrying members of a movable bridge. Main truss chord members and diagonals can have significantly different loadings depending upon whether or not the movable leaf is in the open or closed position. Developing a computer model of the bridge to allow analysis of various stress states during construction will assist in assuring the stability of the bridge leaf and avoid overstressing members. On older bridges with built-up members, the model will allow the engineer to assess how much of a member may be safely taken apart for rehabilitation or replacement. Typically, jacking struts or tension rods are used to transfer loads around a component being rehabilitated. Major repairs to principal members should be avoided under live load conditions or when the bridge is being moved since the member loads may increase significantly under these conditions.



Figure 9-8 Intermediate Balancing of the Congress Avenue Bridge over the Chicago River



Figure 9-9 Late Stage Balancing of Congress Ave Bridge over the Chicago River

- **Placing the bridge into service.** No matter how accurately records are maintained on the removal and installation of materials during rehabilitation, it is impossible to account for all of the materials or weights removed and added to the bridge. One of the safest approaches to placing a rehabilitated movable span into service is to ensure that the movable leaf or span can always be controlled. For spans or leaves rehabilitated in the open position, this typically means ensuring that the span or leaf is counterweight heavy when initially moved. For spans or leaves rehabilitated in the closed position, this typically means ensuring that the movable span or leaf is span or leaf heavy. Additional weight is added to the counterweight or leaf to create this arrangement. A procedure should be developed by the contractor's engineer to incrementally raise or lower the movable span or leaf, stopping the movement, setting the brakes and then releasing the brakes to see if the span or leaf moves on its own. Using an incremental approach, Figure 9-7 and Figure 9-8, to placing the bridge in service

means that the span or leaf is never brought fully open or closed before a thorough understanding of the balance is known. Moving the span or leaf in small increments means that there is never enough momentum to prevent the leaf from being controlled and stopped should an unbalance condition develop.

Adjustments can be made to the additional weight added to the bridge in order to maintain safe control at all times. Occasionally, an owner will specify that the span or leaf cannot be powered open or closed during the initial movement. In these cases, air winches and pulleys are required to control the leaf or span until the full movement is accomplished.

SECTION 7. SUMMARY

While this Manual focuses on stability of girder bridges, as can be seen from this chapter there are many other bridge types. Much of what has been presented in this Manual is applicable to the erection of larger and more complex bridges as well; construction loads and their effects on strength and stability of both individual members and member assemblies must be addressed for all bridges. Particularly for unusual or complex bridges, the method of erection is an integral part of the bridge design and detailed erection procedures should be included with the contract documents.

APPENDIX A REFERENCES

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APPENDIX B DESIGN EXAMPLES

This appendix contains four design examples analyzed with the UT Bridge, UT Lift, UTrAp or STAAD computer program. These four example problems are not comprehensive but serve to illustrate various important constructability checks; girder lifting, girder erection staging, buckling analysis, concrete deck pour sequence analysis, for different bridge types.

The first two examples focus on traditional steel plate girders in continuous multi-span, multi-girder systems. The first example features straight steel plate girders in a skewed 2-span bridge; the second example features curved steel plate girders in a 3-span bridge.

The third example looks at prestressed concrete bulb tee construction. This example features a concrete spliced girder bridge, in which the 3 continuous spans are composed of end girder segments, haunched girder segments at the piers, and a drop-in girder segment at the middle span; the segments are held in place during construction with temporary shoring towers and steel strongbacks prior to cast-in-place concrete pours at the splice locations.

The fourth focuses on a steel bridge similar to that used in example #2; however, instead of multiple curved I-shape plate girders, this 3-span continuous bridge features curved steel tub girders (i.e., open trapezoidal box girders). Although not a consideration during the design, this bridge would be classified as fracture-critical for inspection purposes, as there are only two tub girders.

Example Problem #1: 2-Span Multi-Girder Skewed Steel Bridge

This example problem illustrates many of the critical engineering checks associated with verification of an erection / construction plan. It is not intended to be fully comprehensive, as not all bridge elements and all limit states are being checked at all locations for all forces generated from all relevant load combinations for every stage of the steel erection and concrete deck pour. Rather, it serves to illustrate the level of engineering effort required and to show application of the guidelines and criteria defined in this Manual. In this example, a symmetric 2-span steel plate girder bridge is considered; the multi-stringer bridge has straight steel girders and skewed abutments and pier. The bridge site is located in Illinois.

For simplicity, the analysis / calculation cases in this example are numbered from 0 (crane pick of girder piece) to 5 (start of deck pour). These cases should not be confused with erection stages or deck pour sequence stages. Although the analysis / calculation cases are presented chronologically, there are many more stages than cases; only a handful of cases, representing a snapshot of the erection or deck pour completed up through a certain stage, are being shown here.

Often the unpresented intermediate stages would not govern as an analysis / calculation case (e.g. four-girder set erected up to splice #2 vs. two-girder set erected up to splice #2; the latter is more prone to a system buckling mode). And sometimes a given case, although presented and analyzed, does not need to be checked by inspection (e.g., positive span moment in Case 4 with the completely erected steel will never govern vs. Case 5 with the additional positive span moment induced by the wet concrete of the deck pour).

Some force effects are consistently ignored (e.g., shear in the steel girders) because it is obvious that, although the shear resistance will remain unchanged, the shear demand in the girders due to the final bridge condition of supporting the steel and concrete selfweight and design live load will far exceed the temporary shear demand in the girders due to the steel selfweight during erection or due to the steel selfweight and the wet concrete selfweight during the deck pour. Also, it is assumed that constructability has been verified in the design by Engineer of Record.

Some design locations are consistently ignored (e.g., the steel girders over the pier) because it is obvious that, although the negative moment resistance will remain unchanged, the negative moment demand in the girders due to the final bridge condition of supporting the steel and concrete selfweight and traffic live load will far exceed the temporary negative moment demand in the girders due to the steel selfweight during erection or due to the steel selfweight and the wet concrete selfweight during the deck pour.

Some analysis / calculation cases should be checked but are, for the interest of brevity, not checked here (e.g., the next step after Case 3, in which the last piece of girder line G1 is added, such that G1 is completely assembled and carries its steel selfweight from abutment to abutment, but it is only discretely braced by cross-frames to the adjacent girders up through splice #2, and unbraced beyond that).

Some items that this example does not cover, but which may be required in most cases for complete evaluation, are:

- Recheck of Constructability provisions of AASHTO LRFD BDS
- Deck Pour Sequence Analysis
- Displacements/Cambers/Fit-up forces
- Bearing loads/rotations and temporary blocking details
- Temporary restraints and/or bracing design

- Overturning/uplift checks
- Bolting requirements at splices and connections
- Hardware design (Lifting beams, clamps, jacks, etc.)
- Crane loads/capacity analysis during lifting.

While most of the calculations in the following sections are self-explanatory, commentary are added where appropriate regarding assumptions, alternate methodology, caveats, omitted calculations, derivations, etc.

The general plan, elevation, and cross-section of the bridge are presented in the following figures, along with the framing plan and basic detailing of plate sizes, etc. Additional figures will appear later in the example problem, as needed, just prior to where the relevant calculations are performed.

Getting started

To begin, a meeting is convened with the bridge Erector to understand the means and methods that will be utilized, and the level of experience that exists on bridges of similar proportions and slenderness. It is determined that the Erector will lift all pieces using spreader beams and that all splices and cross-frame connections will be 100% filled with bolts and tensioned “in air” prior to release from cranes. No erection or pouring of deck concrete will be done when wind speeds in excess of 20mph is forecast. Partially completed steelwork may be left overnight or over weekends, but if wind speeds in excess of 20mph are forecast, then additional bracing measures will be installed. The completed steelwork may be exposed for up to 6 months before the deck is completed. For stabilization of the first erected girder, temporary bracing by guy-cables is desired, to avoid need of a third holding crane.

The Erector has experience with erection of bridges with similar proportions: span length = 164 ft, cross-frame spacing = 25ft, flange width = 16”, skew = 20 degrees. However, there are concerns about the slenderness of the two-girder stage since the girder spacing is relatively small (4'-11”). Also, the skewed supports will increase the loads to the cross-frames, which need to be checked.

Analysis

Due to limited Erector experience with a bridge such as this, a refined staged erection analysis is conducted to better understand the behavior and the force effects in the individual components. The structure is modeled using 3D finite element analysis with frame elements. Link elements are utilized to represent the depth of the cross-frame to girder connections. Fixed bearing locations modeled with pin supports and expansion bearings are modeled as rollers. The wind, deck pour, construction dead load, and construction live load forces are simplified to linear loads acting along the centerline of the girders.

To investigate global stability for the first two stages of erection, an Eigenvalue buckling analysis is also conducted using the software tool UT Bridge.

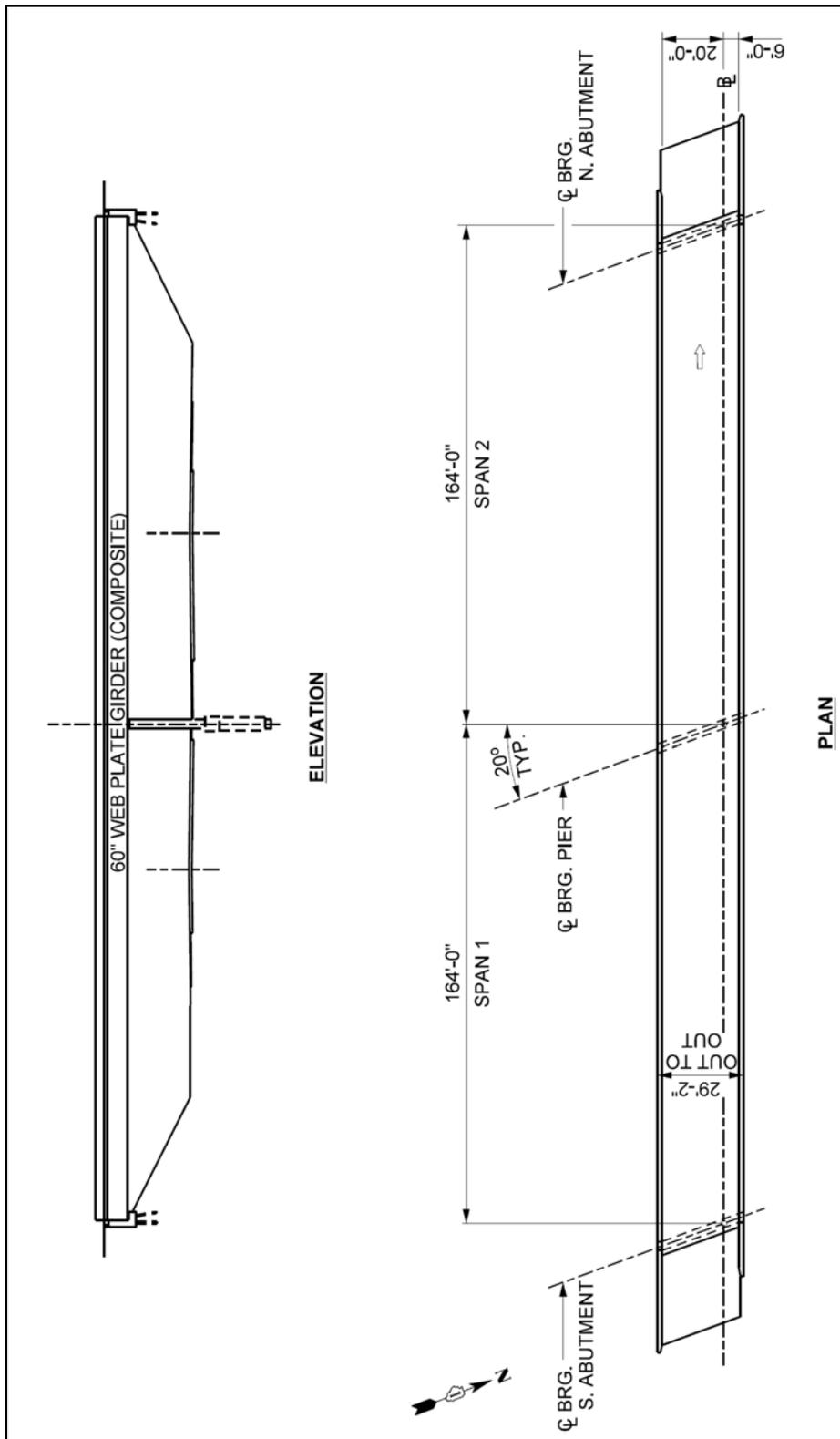


Figure B1-1: Example 1 General Plan and Elevation

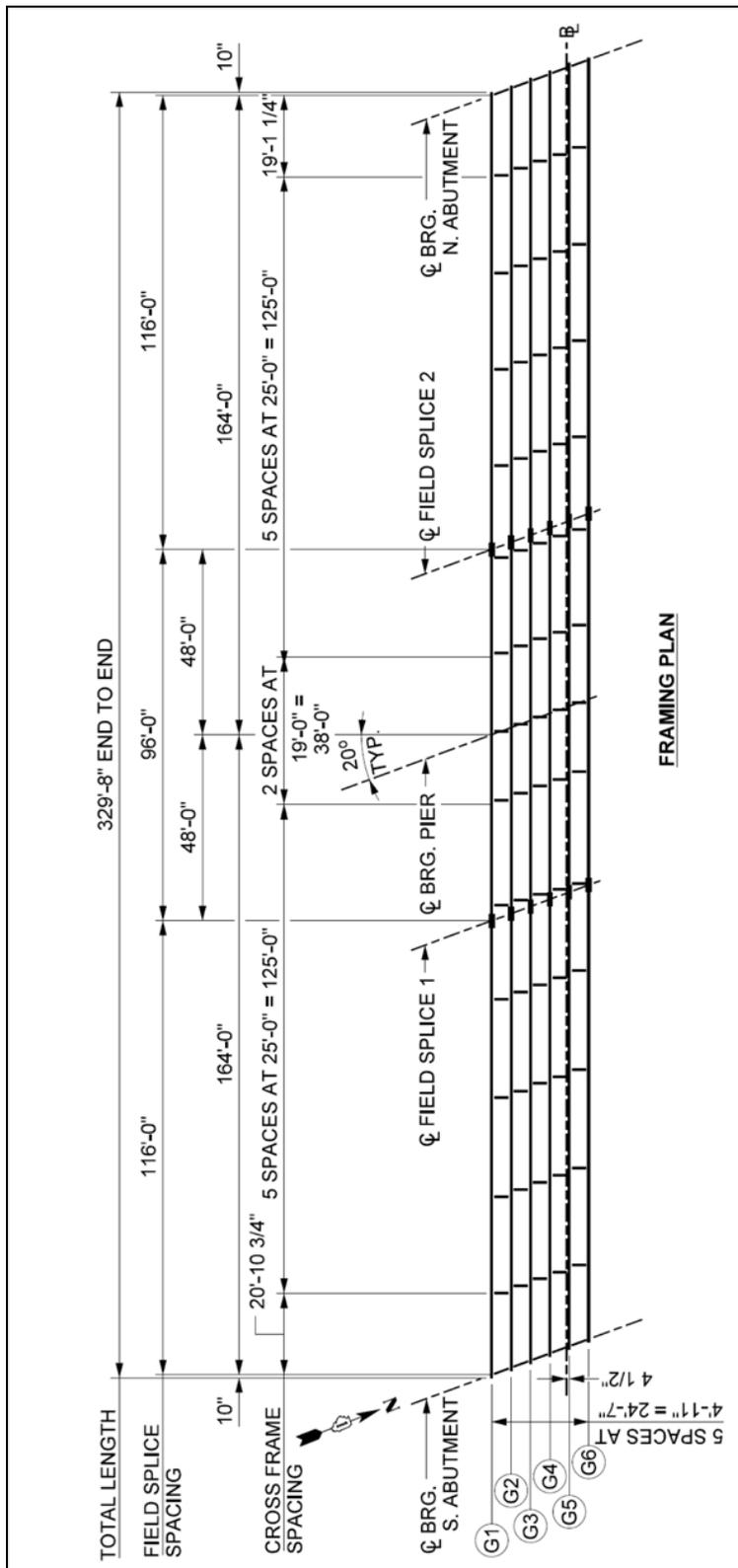


Figure B1-2: Example 1 Framing Plan

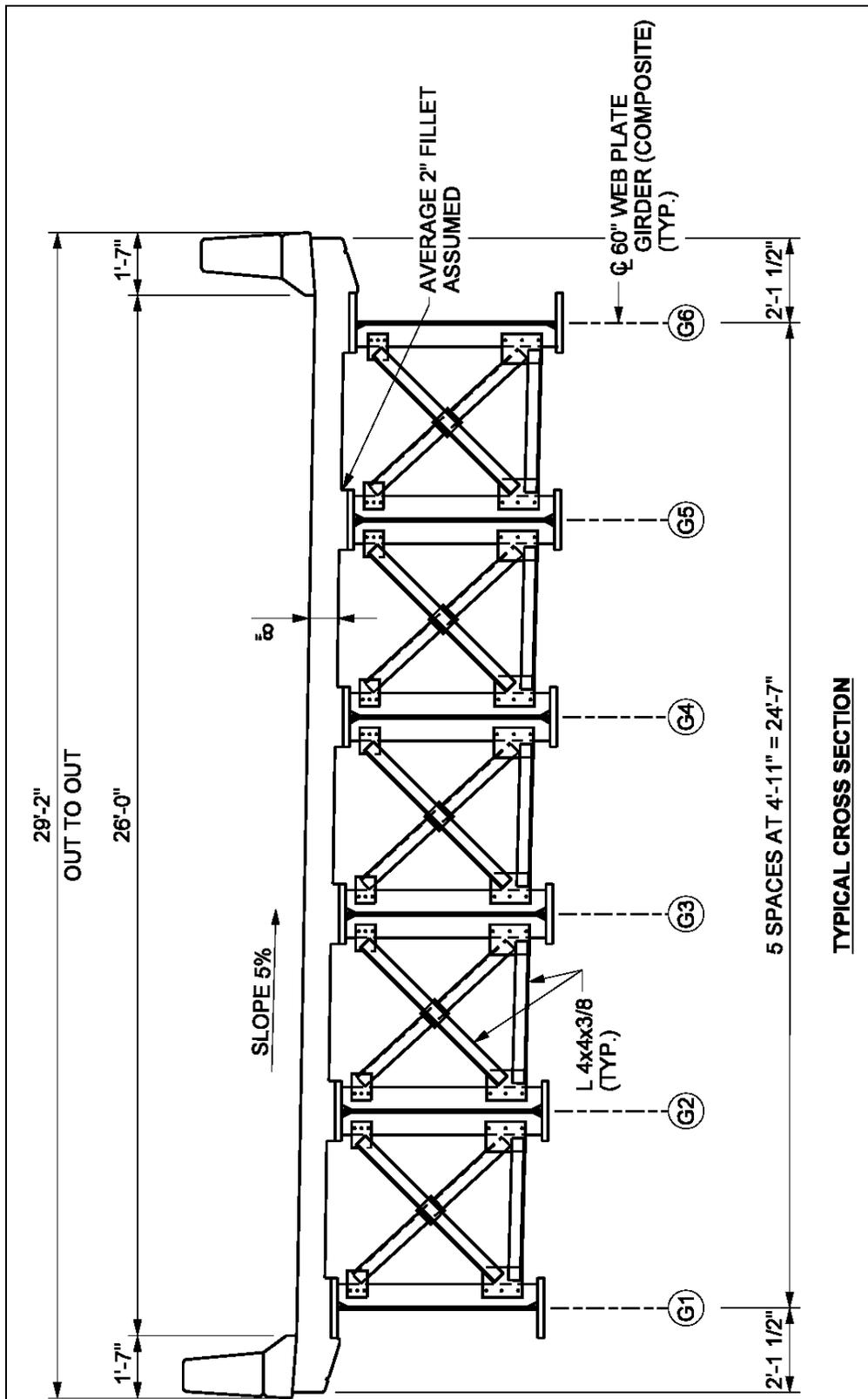


Figure B1-3: Example 1 Cross-section

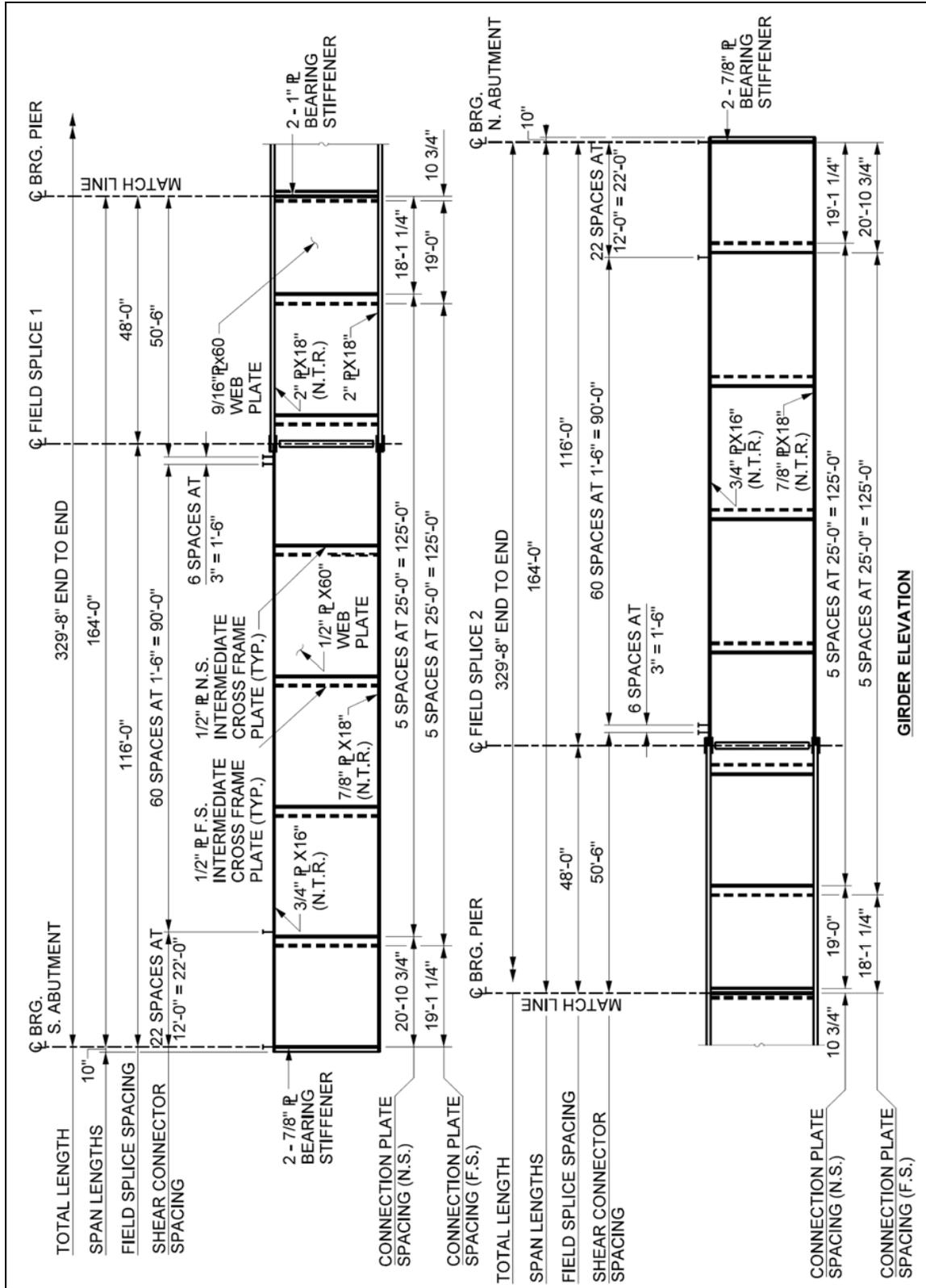


Figure B1-4: Example 1 Girder Elevation

The following table of contents illustrates the general categories into which this example problem is subdivided, and the relevant page number at which the start of each category, and its specific component analysis or calculation tasks, may be found.

<u>General Category / Description</u>	<u>Specific Analysis / Calculation Task</u>	<u>Appendix Page</u>
Table of Contents	N/A	B.7
Bridge Data	N/A	B.8
Applied Forces	Wind Load	B.8
Applied Forces	Wind Forces on Girders	B.9
Structural Analysis	Erection Analysis	B.10
Structural Analysis	Deck Pour Analysis & Evaluation of Stages	B.13
Figure	Single Girder Pick	B.14
Figure	Girder Clamp Loading	B.15
Case 0 (Girder Lift for Splicing)	Section Properties & Applied Forces	B.16
Case 0 (Girder Lift for Splicing)	Applied Stresses	B.17
Case 0 (Girder Lift for Splicing)	Local Buckling Resistance & lateral Torsional Buckling	B.18
Case 0 (Girder Lift for Splicing)	Controlling Resistance & Lateral Bending Stress	B.19
Case 0 (Girder Lift for Splicing)	Overall Flexural Check & Constructibility Check	B.20
Case 0 (Girder Lift for Splicing)	Local Flange Stresses at Girder Clamps	B.20
Case 1 (Single Girder Unbraced)	Section Properties, Applied Forces & Applied Stresses	B.21
Case 1 (Single Girder Unbraced)	Local Buckling Resistance	B.22
Case 1 (Single Girder Unbraced)	Lateral Torsional Buckling & Controlling Resistance	B.23
Case 1 (Single Girder Braced)	Lateral Torsional Buckling & Controlling Resistance	B.24
Case 1 (Single Girder Braced)	Lateral Bending Stress & Overall Flexural Check	B.25
Case 1 (Single Girder Braced)	Constructibility Check	B.26
Case 2 (Twin Girder System)	Global Buckling Resistance	B.27
Case 2 (Twin Girder System)	Lateral Bending & Interaction with Lateral Bending	B.28
Case 3 (Cross-Frame Bracing)	Axial Tension & Axial Compression Resistance	B.30
Case 3 (Cross-Frame Bracing)	Required Strength of Brace	B.32
Case 3 (Cross-Frame Bracing)	Required & Provided Bracing System Stiffness	B.33
Figure	Overhang Bracket Loading	B.34
Case 5 (Concrete Deck Pour)	Section Properties, Applied Forces & Applied Stresses	B.35
Case 5 (Concrete Deck Pour)	Local Buckling Resistance	B.36
Case 5 (Concrete Deck Pour)	Lateral Torsional Buckling & Controlling Resistance	B.37
Case 5 (Concrete Deck Pour)	Lateral Bending Stress & Overall Flexural Check	B.38
Case 5 (Concrete Deck Pour)	Constructibility Check & Overhang Bracket Forces	B.39
Case 1 (Single Girder Unbraced)	Eigenvalue Buckling Analysis in UT Bridge	B.40
Case 2 (Twin Girder System)	Eigenvalue Buckling Analysis in UT Bridge	B.41

Bridge Data:

Continuous 2-span steel plate girder bridge, 6 girders total. Out-to-out deck width is 29'-2". Girder spacing is 4'-11" for a total of 24'-7" between fascia girders. Structural deck slab thickness is 8 in. Cross-slope of deck is 5%. The two spans are symmetric at 164 ft long each, with a 96 ft long splice piece over the pier. Abutments and Pier are at 20 degree skew.

Use Illinois as example location; assume height under 30 feet.

Wind Load:

Use Figure 26.5-1A in ASCE 7-10 to determine basic wind speed for Risk Category II.

$$V := 115 \text{ miles per hour}$$

Use Manual Appendix D Table D-3.2 to determine design wind speed reduction based on construction duration.

Wind Velocity Modification Factor

$$V_m := 0.75 \text{ Assume 6 weeks to 1 year as duration for steel erection.}$$

Design Wind Speed

$$DWS := V_m \cdot V \quad DWS = 86.2 \text{ mph}$$

Incorporate modified Design Wind Speed into pressure equation

Velocity Pressure Exposure Coefficient (ASCE 7-10, Sect 26.7.3)

Assume Surface Roughness C - Open terrain with scattered obstructions having heights generally less than 30 feet. Therefore, Exposure Category C

Manual Table D-3.1, for Height = 30 feet, $K_z = 0.98$. Say $K_z = 1.0$.

Take wind directionality factor as 0.85 and topographic factor as 1.0.

$$K_z := 1 \quad K_{zt} := 1 \quad K_d := 0.85$$

Velocity Pressure (Manual Eq.D-3.4b)

$$q := 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot DWS^2 \quad q_z = 16.2 \text{ psf}$$

Gust Effect Factor $G := 0.85$

Net Force Coefficient (Assuming deck forms not in place) $C_f := 2.2$ since ratio of girder spacing to depth, $S/d < 2$

Wind Load, continued:

Net Pressure

$$Q_z := G \cdot C_f \cdot q_z \quad Q_z = 30.3 \text{ psf}$$

One Day / Girder Setting Design Wind Speed (Assuming minimum wind speed per D-3.4)

$$V := 20 \text{ mph}$$

Use 1.0 Wind Velocity Modification Factor (built in to minimum wind speed)

$$V_m := 1.0$$

Design Wind Speed:

$$DWS := V_m \cdot V \quad DWS = 20 \text{ mph}$$

One Day Girder Setting Velocity Pressure

(Manual Eq. D-3.4b)

$$q_{zset} := 0.00256 \cdot K \cdot K_d \cdot DWS^2 \quad q_{zset} = 0.9 \text{ psf}$$

One Day Girder Setting Net Pressure

$$Q_{zset} := G \cdot C_f \cdot q_{zset} \quad Q_{zset} = 1.6 \text{ psf}$$

1.6 psf is negligible and can be ignored for short-duration events like girder picks which would not be occurring unless the wind is minimal anyway. However, use 5 psf as a minimum pressure for stability checks for pieces that are already set to account for accidental loading, etc. (per D-3.3).

$$Q_{zset} := 5 \text{ psf}$$

Wind Forces on Girders:

Exposed height for girder group = 5.33 feet (worst-case) + 24'-7" bridge width * 5% cross-slope

$$h := 5.33 + 24.58 \cdot 0.05 \quad h = 6.56 \text{ ft}$$

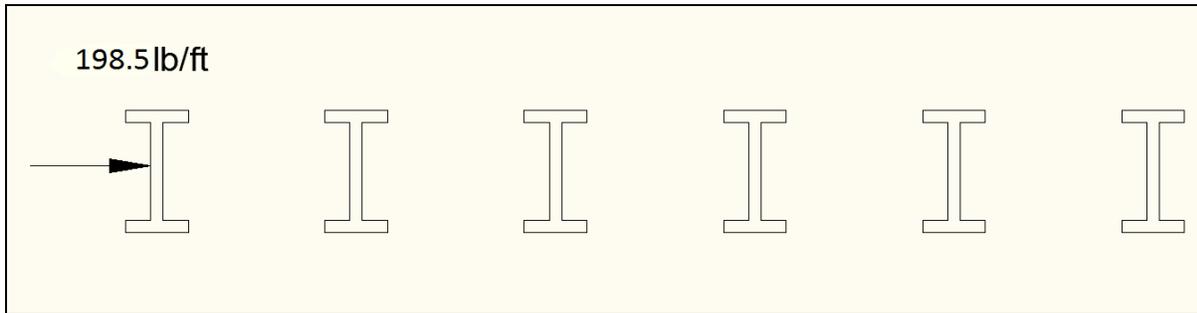
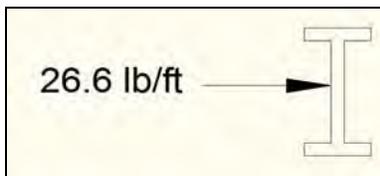
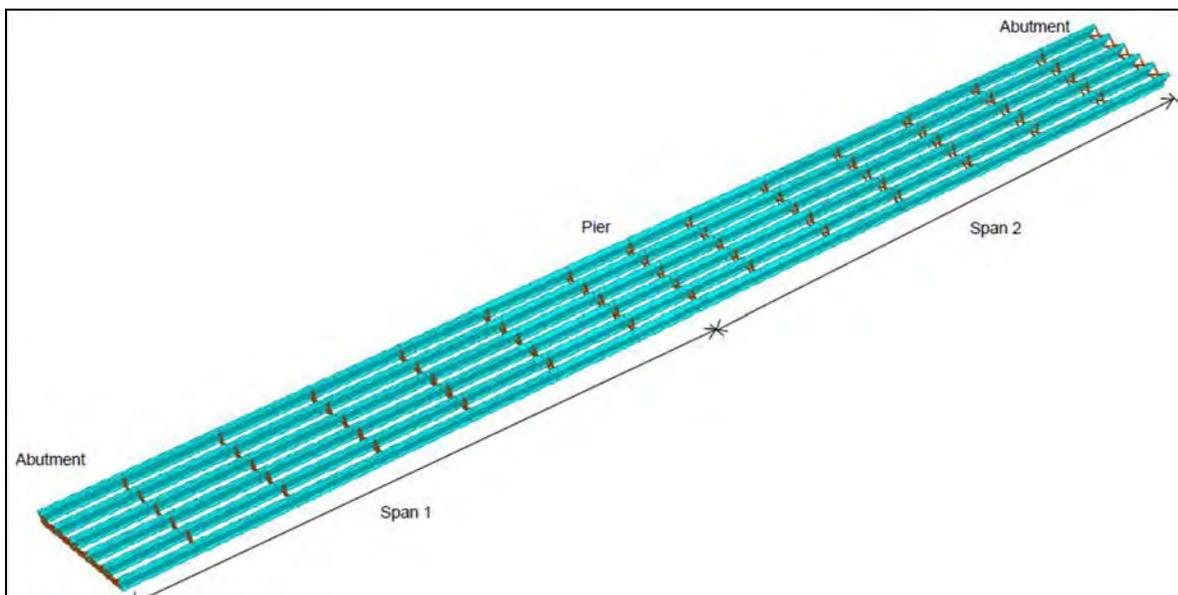
Exposed height for single girder being set = 5.33 feet (worst-case) $h_{set} := 5.33 \text{ ft}$

Force to girder group during partially-erected or fully-erected condition (6+ weeks)

$$W_1 := Q_z \cdot h \quad W_1 = 198.5 \frac{\text{lb}}{\text{ft}}$$

Force to 1st girder during its setting (one day)

$$W_{set} := Q_{zset} \cdot h_{set} \quad W_{set} = 26.6 \frac{\text{lb}}{\text{ft}}$$

Wind Forces on Girders, continued:**Figure B1-5: Wind Load to Girder Group During Partially-Erected or Fully-Erected Condition****Figure B1-6: Wind Load to 1st Girder During Its Setting (One Day)****Erection Analysis For Case 1 to Case 4:****Figure B1-7: Bridge Modeled in 3D Finite Element Analysis Software to Determine Forces to Girders (Note Skew is Present But Difficult to See Due to Perspective)**

Erection Analysis, continued:

For the purpose of this example, 4 erection stages will be analyzed for forces in the girders:

- Case 1: One girder set, Span 1 segment and Pier segment with cantilever into Span 2 up to field splice.
- Case 2: Two girder set, Span 1 segment and Pier segment with cantilever into Span 2 up to field splice.
- Case 3: Six girder set, Span 1 segment and Pier segment with cantilever into Span 2 up to field splice.
- Case 4: All Span 1, Span 2, and Pier segments set.

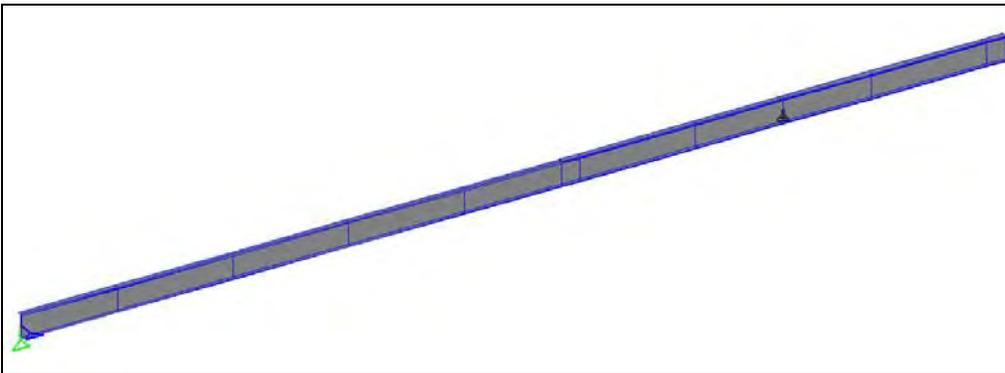


Figure B1-8: Case 1 (Dead = Selfweight, Wind = 26.6 lb/ft)

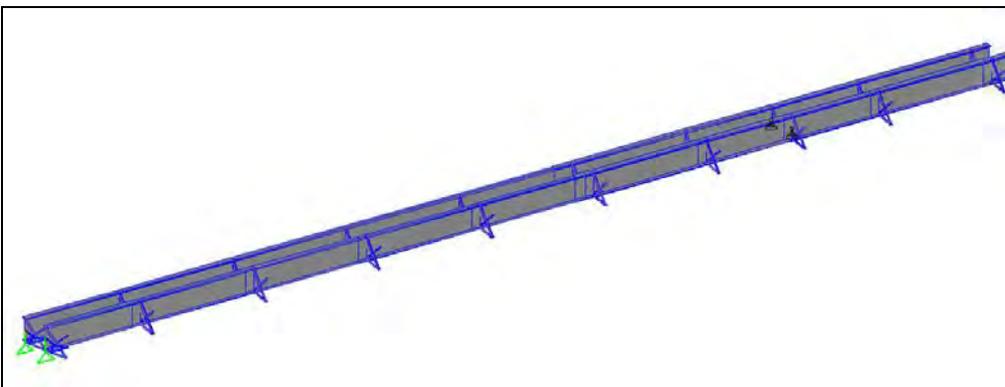
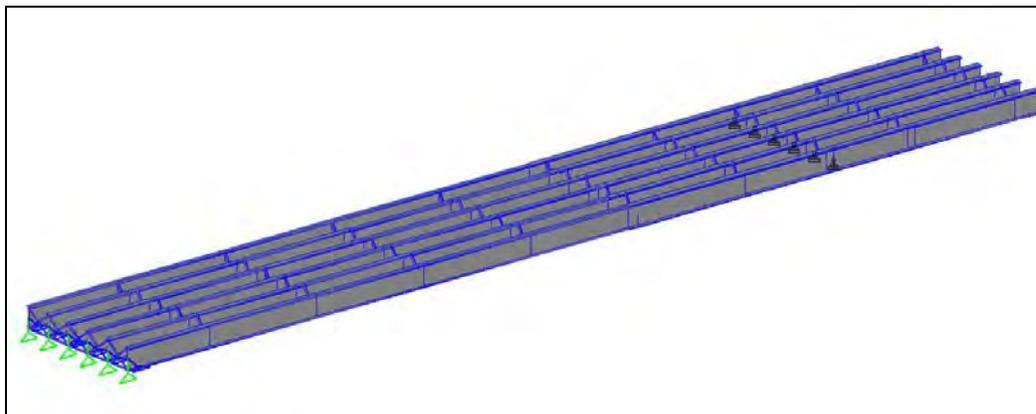
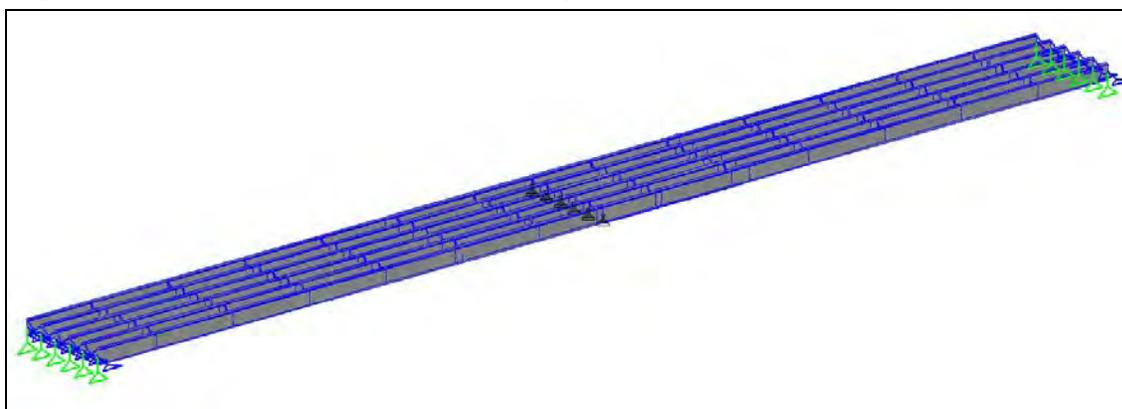


Figure B1-9: Case 2 (Dead = Selfweight, Wind = 198.5 lb/ft)

Erection Analysis, continued:**Figure B1-10: Case 3 (Dead = Selfweight, Wind = 198.5 lb/ft)****Figure B1-11: Case 4 (Dead = Selfweight, Wind = 198.5 lb/ft)**

Other erection cases should also be checked, but are omitted here for brevity. For example, between Case 3 and Case 4 is the condition with the remainder of the 1st girder line installed from the splice to abutment with the one day wind pressure of 26.6 lb/ft. And after this is another intermediate case where the remainder of the 2nd girder line is installed in Span 2 with the cross-frames connected and the 198.5 lb/ft wind load applied to the girder group.

The worst case strong axis and weak axis bending moments as well as worst case compression forces for each segment type of girder for each case is summarized in the table below. The factored results are shown for the Strength III: 1.25 DC + 1.0 CW load combination.

CASE	SEGMENT	M_{ux} (k*ft)	M_{uy} (k*ft)	P_u (k)
1	Span	762	76	0
1	Pier	-579	54	0
2	Span	900	109	64
2	Pier	-623	320	49
3	Span	805	31	14
3	Pier	-590	93	10
4	Span	466	37	16
4	Pier	-1430	114	11

Table B1-9 Erection Analysis Output Summary

Deck Pour Analysis:

For this example, it is assumed the deck pour will run continuously from abutment to abutment, based on the Contractor's requested alteration of the original deck pour sequence on the design drawings (which had the first pours in positive moment regions and then over the pier). The worst case stage for the Span girder segments is when the deck has been poured over Span 1 up to the pier. This stage will be examined for girder adequacy. The concrete weight is taken as 150 pcf and will be treated as permanent dead load. The removable formwork is taken as 10 psf and will be treated as construction dead load. The construction live load is taken as 20 psf in Span 2, but only for computing maximum negative moment at the pier. The factored results are shown for the Strength I: 1.25 DC + 1.50 CDL + 1.50 CLL load combination and the Strength VI: 1.40 DC + 1.40 CDL + 1.40 CLL load combination. No wind load combination (Strength III) is considered, since the deck pour will not be completed in winds above 20 mph.

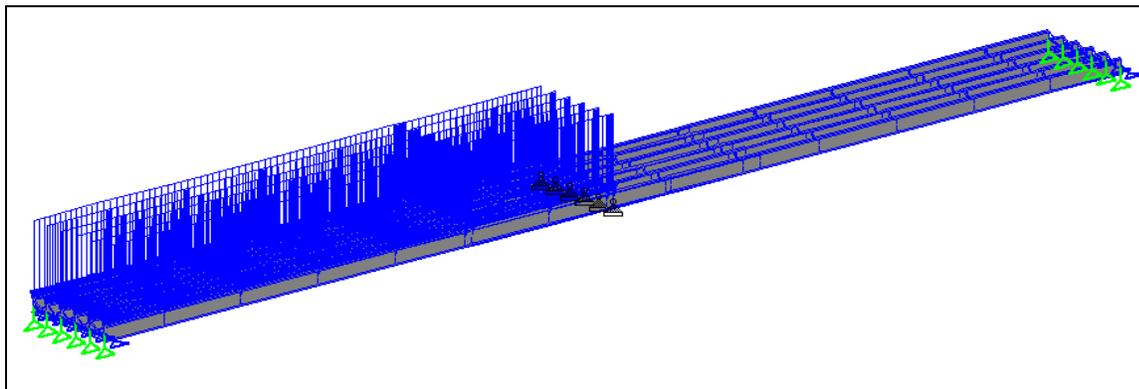


Figure B1-12: Case 5

CASE	COMB.	SEGMENT	M_{ux} (k*ft)	M_{uy} (k*ft)	P_u (k)
5	I	Span	1992	1	1
5	I	Pier	-3643	1	1
5	VI	Span	2227	1	1
5	VI	Pier	-3922	1	1

Table B1-10 Deck Pour Analysis Output Summary

Evaluation of Stages:

For the purpose of this example, calculations will be displayed for specific erection stages. The girders will be checked for adequacy in flexure, axial, and combined stresses according to AASHTO 2012 LRFD Specifications, supplemented by Appendix D of the Manual. The first girder span segment during the crane pick, prior to field splicing, will be checked (Case 0). The first girder, after field splicing, will also be checked for bending resistance (Case 1) with an unbraced length from support to support. The two-girder case (Case 2) will be analyzed for buckling of a set of girders with an unbraced length of the set from support to support. The maximum cross-frame force from the analysis of the six-girder case (Case 3) will be used to size a brace member. By inspection, Case 4 will not govern since the girder forces in this case will be exceeded by the deck pour case. Finally, the span girder (i.e., positive moment) segments will be checked for the deck pour loading (Case 5). While these cases may not represent the critical loading for each girder segment, they serve as an example to check for adequacy.

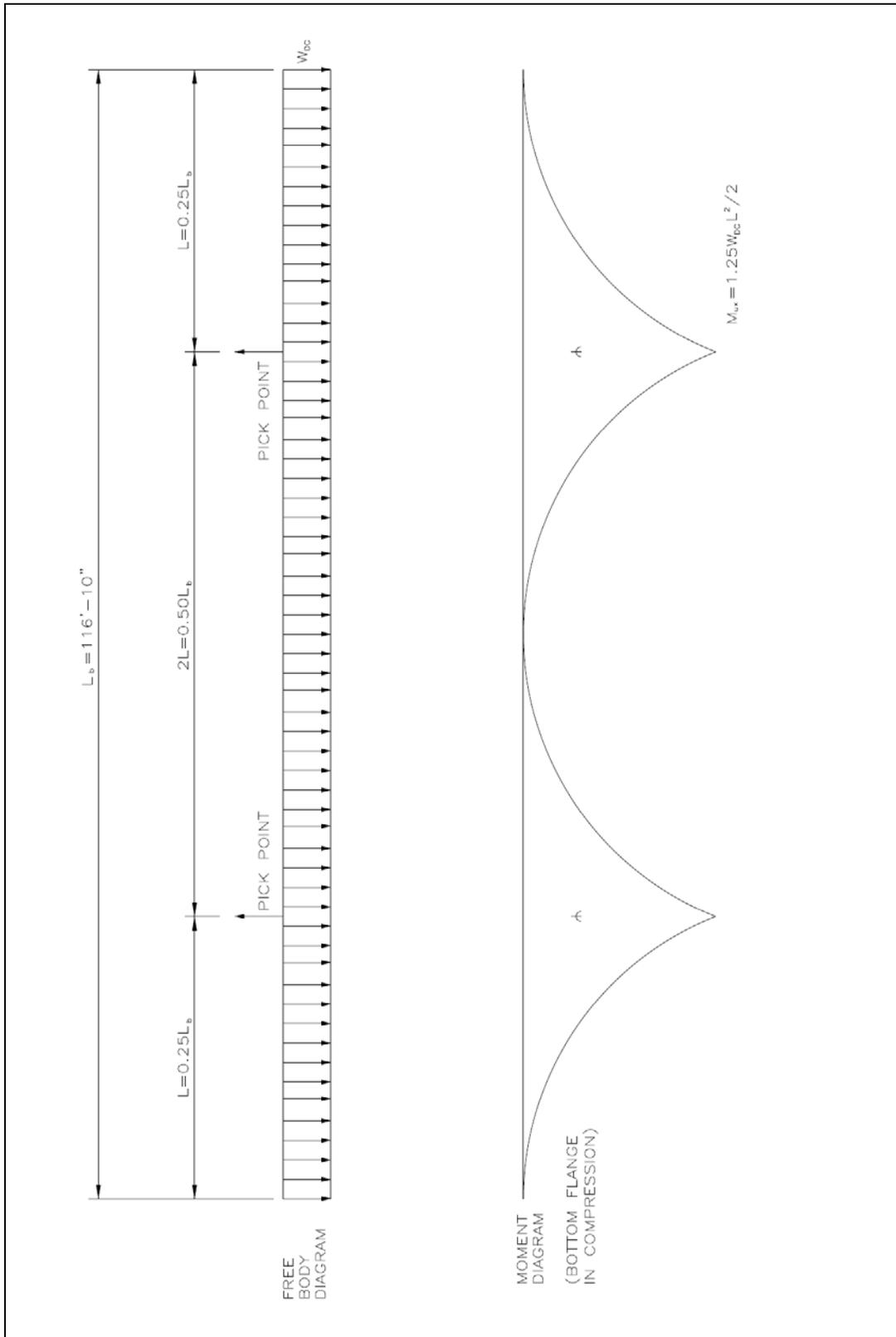


Figure B1-13: Example 1 Single Girder Pick

First Field Section During Lift (Case 0)

The first field section (span portion) of the 1st girder line must be lifted with a crane. It is possible to splice the first field section on the ground with the second field section (pier portion) of the 1st girder line, provided that a crane with adequate capacity can be found to lift the combined span / pier piece of the 1st girder line. However, it is more likely that the contractor will use two smaller cranes (as assumed here), and lift the span portion and pier portion separately, and then splice them in mid-air. The combined piece will then be set on the abutment and pier bearings.

The cranes can be released if the girder is adequate to span from abutment to pier on its own. If not, additional cranes can be used to lift the 2nd girder line so that it can be connected to the 1st with the cross-frames (to reduce the unbraced length from the span length to the cross-frame spacing). Or guy-cables can be installed at the needed intervals in the span to provide brace points, after which the cranes can release the 1st girder line. Then the 2nd girder line can be erected and cross-frames installed between the two lines.

Section Properties:

$$E_s := 29000 \text{ ksi} \quad F_y := 50 \text{ ksi} \quad L_{\text{lift}} = 116 \text{ ft} + \frac{10}{12} \text{ ft} \quad L_{\text{lift}} = 116.8 \text{ ft}$$

Flanges:

$$t_{\text{ff}} := 0.75 \text{ in} \quad b_{\text{ff}} := 16 \text{ in} \quad t_{\text{bf}} := 0.875 \text{ in} \quad b_{\text{bf}} := 18 \text{ in}$$

Web:

$$t_w := 0.5 \text{ in} \quad D := 60 \text{ in}$$

Overall Depth:

$$d := D + t_{\text{ff}} + t_{\text{bf}} \quad d = 61.6 \text{ in}$$

Calculated Properties: (Note that the steps involved in calculating these properties are omitted)

$$S_{\text{tx}} := 1051.8 \text{ in}^3 \quad S_{\text{bx}} := 1192.2 \text{ in}^3 \quad Y_t := 32.74 \text{ in} \quad Y_b := 28.89 \text{ in} \quad S_y := 75.8 \text{ in}^3 \quad Z_x := 1288.4 \text{ in}^3$$

$$A_g := 57.75 \text{ in}^2 \quad r_x := 24.42 \text{ in} \quad r_y := 3.44 \text{ in} \quad I_x := 34436.1 \text{ in}^4 \quad I_y := 681.9 \text{ in}^4 \quad Z_y := 124.5 \text{ in}^3$$

$$\text{Girder selfweight} \quad W_{\text{DC}} := A_g \cdot 0.490 \frac{\text{kip}}{\text{ft}^3} \cdot 1.1 = 0.216 \frac{\text{kip}}{\text{ft}} \text{ with 10\% weight allowance for connections, etc.}$$

Applied Factored Forces: (Assume no cross-frames are attached)

$$\text{Total piece length: } L_{\text{lift}} = 116.8 \text{ ft}$$

$$\text{Assume girder clamps are located at quarter points, where max. moment occurs: } L := \frac{L_{\text{lift}}}{4} = 29.2 \text{ ft}$$

$$\text{Lateral Bending: } M_{\text{ly}} := 0 \text{ kip ft (No appreciable wind during pick, } < 2 \text{ psf)}$$

$$\text{Major-axis Bending for Applicable Strength I \& III Load Combination: } M_{\text{ux}} := 1.25 \cdot W_{\text{DC}} \cdot \frac{L^2}{2} = 115.3 \text{ kip ft}$$

First Field Section During Lift (Case 0), continued**Flexural Resistance:**

Bottom flange is in compression: $b_{fc} := b_{bf}$ $t_{fc} := t_{bf}$

Top flange is in tension: $b_{ft} := b_{tf}$ $t_{ft} := t_{tf}$

Applied Stresses:

Bottom flange is in compression: $S_{yc} := \frac{t_{fc} \cdot b_{fc}^2}{6}$ $S_{yc} = 47.3 \text{ in}^3$

Top flange is in tension: $S_{yt} := \frac{t_{ft} \cdot b_{ft}^2}{6}$ $S_{yt} = 32 \text{ in}^3$

Stress in compression flange without consideration of lateral bending:

$$f_{buc} = \frac{M_{ux}}{S_{bx}} \quad f_{buc} = 1.2 \text{ ksi}$$

First-order stress due to lateral bending in compression flange:

$$f_{L1c} := \frac{M_{uy} \cdot (0.5b_{ft})}{I_y} \quad f_{L1c} = 0 \text{ ksi}$$

Stress in tension flange without consideration of lateral bending:

$$f_{but} = \frac{M_{ux}}{S_{bx}} \quad f_{but} = 1.3 \text{ ksi}$$

First-order stress due to lateral bending in tension flange:

$$f_{Lt} := \frac{M_{uy} \cdot (0.5b_{ft})}{I_y} \quad f_{Lt} = 0 \text{ ksi}$$

Flange Strength Reduction Factors:

Hybrid Factor (AASHTO 6.10.1.10.1)

Since the flexural member is a homogenous built-up section, the hybrid factor shall be taken as unity

$$R_h := 1$$

Web Load Shedding Factor (AASHTO 6.10.1.10.2)

Depth of web in compression $D_c := Y_b - t_{bf}$

$$\lambda_{rw} := \left(5.7 \cdot \sqrt{\frac{E_s}{F_y}} \right) \quad \lambda_{rw} = 137.3$$

$$\frac{2 \cdot D_c}{t_w} = 112.1$$

$$a_{wc} := \frac{2D_c \cdot t_w}{b_{fc} \cdot t_{fc}}$$

$$R_b := \begin{cases} 1.0 & \text{if } \frac{2 \cdot D_c}{t_w} \leq \lambda_{rw} \\ 1.0 - \left(\frac{a_{wc}}{1200 + 300 \cdot a_{wc}} \right) \cdot \left(\frac{2 \cdot D_c}{t_w} - \lambda_{rw} \right) & \text{otherwise} \end{cases}$$

Web is non-compact, so Eq. 6.10.3.2.1-3 need not be checked per AASHTO 6.10.3.2.1

First Field Section During Lift (Case 0), continued**Local Buckling Resistance (AASHTO 6.10.8.2.2):**

Slenderness ratio of the compression flange

$$L_p := r_t \cdot \sqrt{\frac{E_s}{F_y}} \quad \lambda_r = 10.3 \quad (\text{Eq. 6.10.8.2.2-3})$$

$$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E_s}{F_y}} \quad \lambda_{pf} = 9.2 \quad (\text{Eq. 6.10.8.2.2-4})$$

$$\lambda_{rf} := 0.56 \cdot \sqrt{\frac{E_s}{0.7F_y}} \quad \lambda_{rf} = 16.1 \quad (\text{Eq. 6.10.8.2.2-5})$$

Local Buckling Resistance

(Eq. 6.10.8.2.2-1)

$$F_{nc1} := \begin{cases} (R_b \cdot R_h \cdot F_y) & \text{if } \lambda_r \leq \lambda_{pf} \\ \left[1 - \left(1 - \frac{0.7F_y}{R_h \cdot F_y} \right) \cdot \frac{\lambda_r - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right] \cdot R_b \cdot R_h \cdot F_y & \text{otherwise} \end{cases} \quad (\text{Eq. 6.10.8.2.2-2})$$

$$F_{nc1} = 47.6 \text{ ksi}$$

Lateral Torsional Buckling Resistance (AASHTO 6.10.8.2.3)Unbraced length $L_b := L_{\text{lft}}$ (Entire piece length) $L_b = 116.8 \text{ ft.}$

Effective Radius

$$r_t := \frac{b_{fc}}{\sqrt{12 \cdot \left(1 + \frac{1}{3} \cdot \frac{D_c \cdot t_w}{b_{fc} \cdot t_{fc}} \right)}} \quad r_t = 4.6 \text{ in.} \quad (\text{Eq. 6.10.8.2.3-9})$$

Limiting Unbraced Length Calculations:

$$L_p := r_t \cdot \sqrt{\frac{E_s}{F_y}} \quad L_p = 9.2 \text{ ft.} \quad (\text{Eq. 6.10.8.2.3-4})$$

$$L_r := \pi r_t \cdot \sqrt{\frac{E_s}{0.7F_y}} \quad L_r = 34.4 \text{ ft.} \quad (\text{Eq. 6.10.8.2.3-5})$$

Per D-4, Moment gradient modifier

$$C_b := 6.0 \text{ for } L_{\text{lft}}/L = 0.25 \text{ (quarter points)}$$

Lateral Torsional Buckling Resistance (conservatively used in lieu of AASHTO Appendix D6.4.2)

$$F_{nc2} := \begin{cases} (R_b \cdot R_h \cdot F_y) & \text{if } L_b \leq L_p \\ \left[C_b \cdot \left[1 - \left(1 - \frac{0.7F_y}{R_h \cdot F_y} \right) \cdot \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \right] \cdot R_b \cdot R_h \cdot F_y & \text{if } L_p < L_b \leq L_r \\ \frac{C_b \cdot R_b \cdot \pi^2 \cdot E_s}{\left(\frac{L_b}{r_t} \right)^2} & \text{otherwise} \end{cases}$$

$$F_{nc2} = 18.2 \text{ ksi}$$

(Eq. 6.10.8.2.3-1, 6.10.8.2.3-2, 6.10.8.2.3-3)

First Field Section During Lift (Case 0), continued**Controlling Nominal Flexural Resistance:**

$$F_{nc} := \min(F_{nc1}, F_{nc2}) \quad F_{nc} = 18.2 \text{ ksi}$$

$$\text{Resistance factor} \quad \phi_f := 1.0 \text{ (AASHTO 6.5.4.2)}$$

$$\Phi_f \cdot F_{nc} = 18.2 \text{ ksi}$$

$$M_{rx} = \Phi_f \cdot F_{nc} \cdot S_{bx} \quad M_{rx} = 1807.7 \text{ kip ft}$$

$$\frac{M_{ux}}{M_{rx}} = 0.06 \quad \text{Adequate resistance for lateral-torsional buckling}$$

Note that no impact factor is necessary for the crane pick (20% impact only for demolition, not erection).

Determine Stress due to Lateral Bending:

First-order lateral bending stress (from previous) $f_{L1c} = 0 \text{ ksi}$

Limiting unbraced length for first-order lateral bending stress

$$1.2 \cdot L_p \cdot \sqrt{\frac{C_b \cdot R_b}{\frac{f_{buc}}{F_y}}} = 176.7 \text{ ft} \quad (\text{Eq. 6.10.1.6-2})$$

Lateral bending stress:

elastic lateral torsional buckling stress

$$F_{cr} := \frac{C_b \cdot R_b \cdot \pi^2 \cdot E_s}{\left(\frac{L_b}{r_t}\right)^2} \quad F_{cr} = 18.2 \text{ ksi} \quad (\text{Eq. 6.10.8.2.3-8})$$

Check if the first-order stress needs to be amplified: Approximated second-order lateral bending stress

$$f_{Lc} := \begin{cases} f_{Lc1} & \text{if } L_b \leq 1.2 \cdot L_p \cdot \sqrt{\frac{C_b \cdot R_b}{\frac{f_{buc}}{F_y}}} \\ \left(\frac{0.85}{1 - \frac{f_{buc}}{F_{cr}}} \right) \cdot f_{L1c} & \text{otherwise} \end{cases} \quad f_{Lc} = 0 \text{ ksi} \quad (\text{Eq. 6.10.1.6-4})$$

Lateral bending check:

$$\begin{aligned} \text{Lateral_Bending_Resistance} &:= \left\| \begin{array}{l} \text{“Lateral Bending Requirements Satisfied” if } f_{Lc} \leq 0.6F_y \\ \text{Girder is NOT Adequate in Lateral Flexure” otherwise} \end{array} \right. \\ \text{Lateral_Bending_Resistance} &:= \text{“Lateral Bending Requirements Satisfied”} \end{aligned}$$

First Field Section During Lift (Case 0), continued**Overall Flexural Resistance Check:**

The following must be satisfied:

$$f_{\text{buc}} + \frac{1}{3} \cdot f_{\text{Lc}} \leq \phi_f \cdot F_{\text{nc}} = 18.2 \text{ ksi} \quad (\text{Eq. 6.10.3.2.1-2})$$

$$f_{\text{buc}} + \frac{1}{3} \cdot f_{\text{Lc}} = 1.2 \text{ ksi} \quad \phi_f \cdot F_{\text{nc}} = 18.2 \text{ ksi}$$

$$\text{Resistance_Check} := \begin{cases} \text{"Girder Meets AASHTO Flexural Requirements"} & \text{if } f_{\text{buc}} + \frac{1}{3} \cdot f_{\text{Lc}} \leq \phi_f \cdot F_{\text{nc}} \\ \text{"Girder is NOT Adequate in Lateral Flexure"} & \text{otherwise} \end{cases}$$

$$\text{Resistance_Check} = \text{"Girder Meets AASHTO Flexural Requirements"}$$

Constructability Check:

The following must be satisfied:

$$\begin{aligned} f_{\text{buc}} + f_{\text{Lc}} &\leq \Phi_f \cdot R_h \cdot F_y && (\text{Eq. 6.10.3.2.2-1}) \\ f_{\text{buc}} + f_{\text{Lc}} &= 1.2 \text{ ksi} < \Phi_f \cdot R_h \cdot F_y = 50 \text{ ksi} \\ &\text{Adequate resistance in compression flange} \end{aligned}$$

The following must be satisfied:

$$\begin{aligned} F_{\text{but}} + f_{\text{Lt}} &\leq \Phi_f \cdot R_h \cdot F_y && (\text{Eq. 6.10.3.2.2-1}) \\ f_{\text{buc}} + f_{\text{Lt}} &= 1.3 \text{ ksi} < \Phi_f \cdot R_h \cdot F_y = 50 \text{ ksi} \\ &\text{Adequate resistance in tension flange} \end{aligned}$$

Local Flange Stresses Check:

In lieu of the simplistic method outlined in Manual Sect. D-6, use the AISC Steel Manual 13th Edition resistance equation to check for flange local ending at the girder clamps (Section J10.1).

The total clamp force at each of the two girder clamps = P

$$P := 1.25 \cdot W_{\text{DC}} \cdot \frac{L_{\text{lift}}}{2} \quad \text{for Strength Load Combination} \quad P = 15.8 \text{ kip}$$

The tensile concentrated force at each side of the top flange under the girder clamp = P/2

The following must be satisfied:

$$\begin{aligned} \frac{P}{2} &\leq \phi \cdot 6.25 \cdot t_{\text{f}}^2 \cdot F_y && (\text{AISC Eq. J10-1}) \text{ where } \Phi := 0.9 \\ \frac{P}{2} &= 7.9 \text{ kip} < \phi \cdot 6.25 \cdot t_{\text{f}}^2 \cdot F_y = 158.2 \text{ kip} \end{aligned}$$

Adequate resistance in top flange for concentrated tensile forces

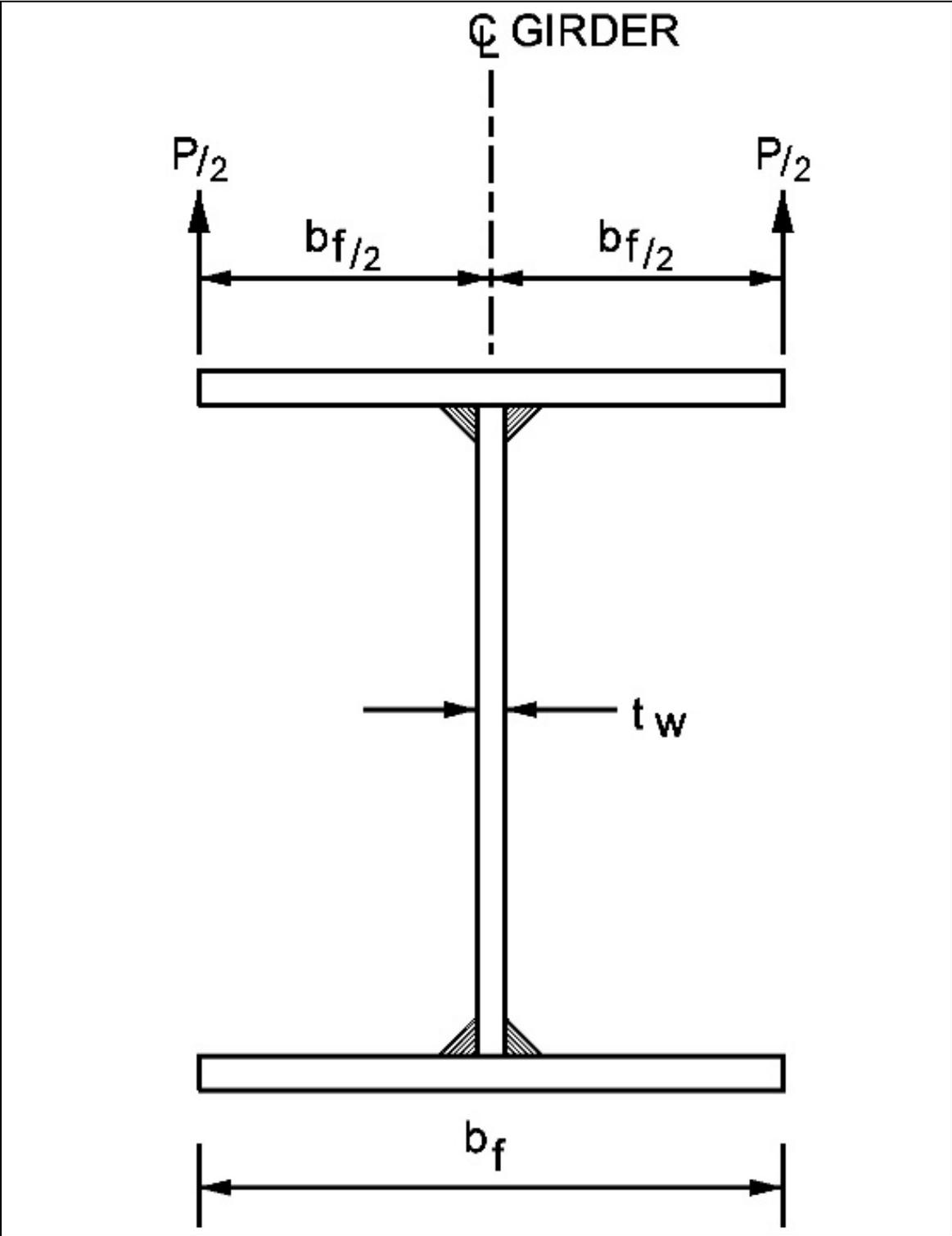


Figure B1-14: Example 1 Girder Clamp Loading

Resistance of a Single Girder for Case 1 Loading (Span 1 with Cantilever, 1st Line Erected)

Span Segment Controls Case 1 Loading

Section Properties: $E_s := 29000 \text{ ksi}$ $F_y = 50 \text{ ksi}$

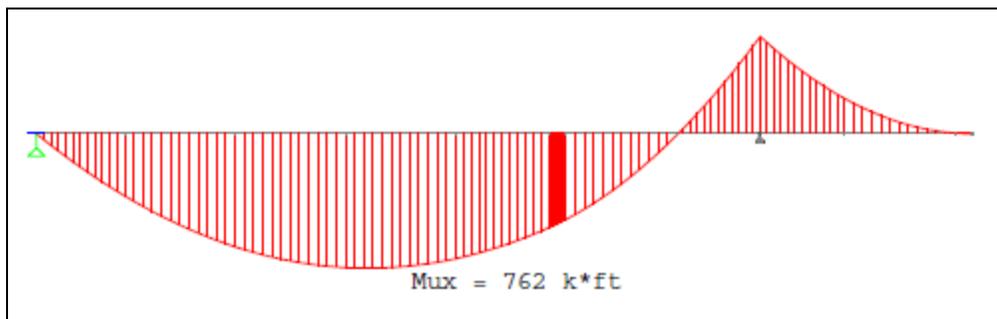
Flanges:

 $t_{tf} := 0.75 \text{ in}$ $b_{tf} := 16 \text{ in}$ $t_{bf} := 0.875 \text{ in}$ $b_{bf} := 18 \text{ in}$

Web:

 $t_w := 0.5 \text{ in}$ $D := 60 \text{ in}$

Overall Depth:

 $d := D + t_{tf} + t_{bf}$ $d = 61.6 \text{ in}$ **Calculated Properties:** (Note that the steps involved in calculating these properties are omitted, as they may be easily generated by spreadsheets, analysis software, etc.) $S_{tx} := 1051.8 \text{ in}^3$ $S_{bx} := 1192.2 \text{ in}^3$ $Y_t := 32.74 \text{ in}$ $Y_b := 28.89 \text{ in}$ $S_y := 75.8 \text{ in}^3$ $Z_x := 1288.4 \text{ in}^3$ $A_g := 57.75 \text{ in}^2$ $r_x := 24.42 \text{ in}$ $r_y := 3.44 \text{ in}$ $I_x := 34436.1 \text{ in}^4$ $I_y := 681.9 \text{ in}^4$ $Z_y := 124.5 \text{ in}^3$ **Applied Factored Forces (From Erection Analysis Output Summary):**Major-axis Bending: $M_{ux} := 762 \text{ kip}$ Lateral Bending $M_{uy} := 76 \text{ kip ft}$ **Figure B1-15: Major-Axis Bending Moment Diagram****Applied Stresses:**

Top flange is in compression:

$b_{fc} := b_{tf}$

$t_{fc} := t_{tf}$

Bottom flange is in tension:

$b_{ft} := b_{bf}$

$t_{ft} := t_{bf}$

Stress in compression flange without consideration of lateral bending:

$f_{buc} := \frac{M_{ux}}{S_{tx}}$

$f_{buc} = 8.7 \text{ ksi}$

First-order stress due to lateral bending in compression flange: $f_{L1c} := \frac{M_{uy} \cdot (0.5 \cdot b_{fc})}{I_y}$ $f_{L1c} = 10.7 \text{ ksi}$ Stress in tension flange without consideration of lateral bending: $f_{but} := \frac{M_{ux}}{S_{bx}}$ $f_{but} := 7.7 \text{ ksi}$ First-order stress due to lateral bending in tension flange: $f_{Lt} := \frac{M_{uy} \cdot (0.5 \cdot b_{ft})}{I_y}$ $f_{Lt} = 12 \text{ ksi}$

Resistance of a Single Girder for Case 1 Loading (Span 1 with Cantilever, 1st Line Erected), cont.

Flexural Resistance:

Flange Strength Reduction Factors:

Hybrid Factor (AASHTO 6.10.1.10.1)

Since the flexural member is a homogenous built-up section, the hybrid factor shall be taken as unity

$$R_h := 1$$

Web Load Shedding Factor (AASHTO 6.10.1.10.2)

$$\text{Depth of web in compression } D_c := Y_t - t_{ff}$$

$$\lambda_{rw} := \left(5.7 \cdot \sqrt{\frac{E_s}{F_y}} \right) \quad \lambda_{rw} = 137.3 \quad \frac{2 \cdot D_c}{t_w} = 128$$

$$a_{wc} := \frac{2D_c \cdot t_w}{b_{fc} \cdot t_{fc}}$$

$$R_b := \begin{cases} 1.0 & \text{if } \frac{2 \cdot D_c}{t_w} \leq \lambda_{rw} \\ 1.0 - \left(\frac{a_{wc}}{1200 + 300 \cdot a_{wc}} \right) \cdot \left(\frac{2 \cdot D_c}{t_w} - \lambda_{rw} \right) & \text{otherwise} \end{cases} \quad R_b = 1$$

Web is non-compact, so Eq. 6.10.3.2.1-3 need not be checked per AASHTO 6.10.3.2.1.

Local Buckling Resistance (AASHTO 6.10.8.2.2):

Slenderness ratio of the compression flange

$$\lambda_f := \frac{b_{fc}}{2 \cdot t_{fc}} \quad \lambda_f = 10.7 \quad (\text{Eq. 6.10.8.2.2-3})$$

$$\lambda_{pf} := 0.38 \sqrt{\frac{E_s}{F_y}} \quad \lambda_{pf} = 9.2 \quad (\text{Eq. 6.10.8.2.2-4})$$

$$\lambda_{rf} := 0.56 \cdot \sqrt{\frac{E_s}{0.7F_y}} \quad \lambda_{rf} = 16.1 \quad (\text{Eq. 6.10.8.2.2-5})$$

Local Buckling Resistance (Eq. 6.10.8.2.2-1 & Eq. 6.10.8.2.2-2)

$$F_{nc1} := \begin{cases} (R_b \cdot R_h \cdot F_y) & \text{if } \lambda_r \leq \lambda_{pf} \\ \left[1 - \left(1 - \frac{0.7F_y}{R_h \cdot F_y} \right) \cdot \frac{\lambda_r - \lambda_{pf}}{\lambda_{tr} - \lambda_{pf}} \right] \cdot R_b \cdot R_h \cdot F_y & \text{otherwise} \end{cases}$$

$F_{nc1} = 46.7 \text{ ksi}$

Resistance of a Single Girder for Case 1 Loading (Span 1 with Cantilever, 1st Line Erected), cont.**Lateral Torsional Buckling Resistance (AASHTO 6.10.8.2.3):**Unbraced length $L_b := 164$ ft Taken as span length of 164 ftEffective Radius
of Gyration

$$r_t := \frac{b_{fc}}{\sqrt{12 \cdot \left(1 + \frac{1}{3} \cdot \frac{D_c \cdot t_w}{b_{fc} \cdot t_{fc}}\right)}} \quad r_t = 3.8 \text{ in} \quad (\text{Eq. 6.10.8.2.3-9})$$

Limiting Unbraced Length Calculations:

$$L_p := r_t \cdot \sqrt{\frac{E_s}{F_y}} \quad L_p = 7.7 \text{ ft} \quad (\text{Eq. 6.10.8.2.3-4})$$

$$L_r := \pi r_t \cdot \sqrt{\frac{E_s}{0.7F_y}} \quad L_r = 29 \text{ ft} \quad (\text{Eq. 6.10.8.2.3-5})$$

Moment gradient modifier $C_b := 1$ since, $f_2 := 0$ (See 4th sample in AASHTO LRFD C6.4.10, corresponding to moment in main span of beam with cantilever overhang present)**Lateral Torsional Buckling Resistance**

$$F_{nc2} := \begin{cases} (R_b \cdot R_h \cdot F_y) & \text{if } L_b \leq L_p \\ C_b \cdot \left[1 - \left(1 - \frac{0.7F_y}{R_h \cdot F_y} \right) \cdot \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \cdot R_b \cdot R_h \cdot F_y & \text{if } L_p < L_b \leq L_r \\ \frac{C_b \cdot R_b \cdot \pi^2 \cdot E_s}{\left(\frac{L_b}{r_t} \right)^2} & \text{otherwise} \end{cases} \quad (\text{Eq. 6.10.8.2.3-1})$$

$$F_{nc2} = 1.1 \text{ ksi}$$

$$(\text{Eq. 6.10.8.2.3-2})$$

$$(\text{Eq. 6.10.8.2.3-3})$$

Resistance of a Single Girder for Case 1 Loading (Span 1 with Cantilever, 1st Line Erected), cont.**Controlling Nominal Flexural Resistance:**

$$F_{nc} := \min(F_{nc1}, F_{nc2}) \quad F_{nc} = 1.1 \text{ ksi}$$

$$\text{Resistance factor} \quad \Phi_f := 1.0 \quad (\text{AASHTO 6.5.4.2})$$

$$\Phi_f \cdot F_{nc} = 1.1 \text{ ksi}$$

$$M_{rx} := \Phi_f \cdot F_{nc} \cdot S_{tx} \quad M_{rx} = 95.7 \text{ kip ft} < \quad M_{ux} = 762 \text{ kip ft}$$

Single girder is inadequate in flexure. The girder must be held in place (supported) by crane or bracing until the second adjacent girder is set and all the diaphragm connections are installed. Guy-cables will be used at the contractor's preference to support the girder.

Lateral Torsional Buckling Resistance (AASHTO 6.10.8.2.3):

$$\text{Unbraced length} \quad L_b := 41 \text{ ft} \quad \text{Assuming tie-downs at } \frac{1}{4} \text{ points of 164 ft span length}$$

Effective Radius of Gyration

$$r_t := \frac{b_{fc}}{\sqrt{12 \cdot \left(1 + \frac{1}{3} \cdot \frac{D_c \cdot t_w}{b_{fc} \cdot t_{fc}}\right)}} \quad r_t = 3.8 \text{ in} \quad (\text{Eq. 6.10.8.2.3-9})$$

Limiting Unbraced Length Calculations:

$$L_p := r_t \cdot \sqrt{\frac{E_s}{F_y}} \quad L_p = 7.7 \text{ ft} \quad (\text{Eq. 6.10.8.2.3-4})$$

$$L_r := \pi r_t \cdot \sqrt{\frac{E_s}{0.7F_y}} \quad L_r = 29 \text{ ft} \quad (\text{Eq. 6.10.8.2.3-5})$$

$$\text{Moment gradient modifier} \quad C_b := 1 \quad \text{since, } \frac{f_{mid}}{f_2} > 1 \text{ (See 3}^{rd} \text{ sample in AASHTO LRFD C6.4.10)}$$

Lateral Torsional Buckling Resistance

$$F_{nc2} := \begin{cases} (R_b \cdot R_h \cdot F_y) & \text{if } L_b \leq L_p \\ \left[C_b \cdot \left[1 - \left(1 - \frac{0.7F_y}{R_h \cdot F_y} \right) \cdot \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \cdot R_b \cdot R_h \cdot F_y \right] & \text{if } L_p < L_b \leq L_r \\ \frac{C_b \cdot R_b \cdot \pi^2 \cdot E_s}{\left(\frac{L_b}{r_t} \right)^2} & \text{otherwise} \end{cases} \quad (\text{Eq. 6.10.8.2.3-1})$$

$$F_{nc2} = 17.5 \text{ ksi}$$

$$(\text{Eq. 6.10.8.2.3-2})$$

$$(\text{Eq. 6.10.8.2.3-3})$$

Resistance of a Single Girder for Case 1 Loading (Span 1 with Cantilever, 1st Line Erected), cont.**Controlling Nominal Flexural Resistance:**

$$F_{nc} := \min(F_{nc1}, F_{nc2}) \quad F_{nc} = 17.5 \text{ ksi}$$

$$\text{Resistance factor} \quad \Phi_f := 1.0 \quad (\text{AASHTO 6.5.4.2})$$

$$\Phi_f \cdot F_{nc} = 17.5 \text{ ksi}$$

$$M_{rx} := \Phi_f \cdot F_{nc} \cdot S_{tx} \quad M_{rx} = 1530.8 \text{ kip ft} \quad > \quad M_{ux} = 762 \text{ kip ft}$$

The temporary guy-cables selected by the contractor will be used to prevent lateral translation of the cross-section. Guy-cables are an acceptable solution here provided that the unbraced length is limited to $\frac{1}{4}$ of the span or less (say 41 ft). Now check lateral bending and flexural interaction to ensure that unbraced length is OK.

Determine Stress due to Lateral Bending:

First-order lateral bending stress (from previous): $f_{L1c} = 10.7 \text{ ksi}$

Limiting unbraced length for first-order lateral bending stress

$$1.2 \cdot L_p \cdot \sqrt{\frac{C_b \cdot R_b}{\frac{f_{buc}}{F_y}}} = 22.2 \text{ ft} \quad (\text{Eq. 6.10.1.6-2})$$

Lateral bending stress:

elastic lateral torsional buckling stress

$$F_{cr} := \frac{C_b \cdot R_b \cdot \pi^2 \cdot E_s}{\left(\frac{L_b}{r_t}\right)^2} \quad F_{cr} = 17.5 \text{ ksi} \quad (\text{Eq. 6.10.8.2.3-8})$$

Check if the first-order stress needs to be amplified:

Approximated second-order lateral bending stress:

$$f_{Lc} := \begin{cases} f_{L1c} & \text{if } L_b \leq 1.2 \cdot L_p \cdot \sqrt{\frac{C_b \cdot R_b}{\frac{f_{buc}}{F_y}}} \\ \left(\frac{0.85}{1 - \frac{f_{buc}}{F_{cr}}} \right) \cdot f_{L1c} & \text{otherwise} \end{cases} \quad f_{Lc} = 18.1 \text{ ksi}$$

(Eq. 6.10.1.6-4)

Lateral bending check:

$$\begin{aligned} \text{Lateral_Bending_Resistance} & := \left\{ \begin{array}{l} \text{“Lateral Bending Requirements Satisfied” if } f_{Lc} \leq 0.6F_y \\ \text{“Girder is NOT Adequate in Lateral Flexure” otherwise} \end{array} \right. \\ \text{Lateral_Bending_Resistance} & = \text{“Lateral Bending Requirements Satisfied”} \end{aligned}$$

Resistance of a Single Girder for Case 1 Loading (Span 1 with Cantilever, 1st Line Erected), cont.

Overall Flexural Resistance Check:

The following must be satisfied:

$$f_{buc} + \frac{1}{3} \cdot f_{Lc} \leq \phi_f \cdot F_{nc} \quad (\text{Eq. 6.10.3.2.1-2})$$

$$f_{buc} + \frac{1}{3} \cdot f_{Lc} = 14.7 \text{ ksi} \quad \phi_f \cdot F_{nc} = 17.5 \text{ ksi}$$

$$\text{Resistance_Check} := \begin{cases} \text{"Girder Meets AASHTO Flexural Requirements"} & \text{if } f_{buc} + \frac{1}{3} \cdot f_{Lc} \leq \phi_f \cdot F_{nc} \\ \text{"Girder is NOT Adequate in Lateral Flexure"} & \text{otherwise} \end{cases}$$

Resistance_Check = "Girder Meets AASHTO Flexural Requirements"

Constructability Check:

The following must be satisfied:

$$f_{buc} + f_{Lc} \leq \Phi_f \cdot R_h \cdot F_y \quad (\text{Eq. 6.10.3.2.1-1})$$

$$f_{buc} + f_{Lc} = 26.8 \text{ ksi} < \Phi_f \cdot R_h \cdot F_y = 50 \text{ ksi}$$

Adequate resistance in compression flange

The following must be satisfied:

$$f_{but} + f_{Lt} \leq \Phi_f \cdot R_h \cdot F_y \quad (\text{Eq. 6.10.3.2.2-1})$$

$$f_{but} + f_{Lt} = 19.7 \text{ ksi} < \Phi_f \cdot R_h \cdot F_y = 50 \text{ ksi}$$

Adequate resistance in tension flange

Also note the following: among other assumptions, the above Case 1 calculations were predicted on the supports (pier and abutment) acting as brace points. For a single girder line, cross-frames are not yet present, so this bracing assumption is only valid if the detailing is done properly to prevent flange rotation / translation (i.e., tie-downs or other temporary girder end bracing).

Two Girders After Release (Case 2)

As explained for Cases 0 and 1, the 1st girder line combined segment (Span 1 piece spliced to pier piece) must be temporarily supported or braced so that the 2nd girder line segment (Span 1 piece spliced to pier piece) can be erected and then attached via the cross-frames to the 1st girder line. This could be accomplished via the use of the original picking cranes, or the installation of shoring towers in Span 1, or the installation of cable tie-downs in Span 1. It will be required to connect all cross-frames (100% bolts) between the two girder lines before releasing the temporary braces or removing the temporary supports. At this point, the unbraced length for both girder lines will be equal to the cross-frame spacing. By inspection, lateral-torsional buckling between cross-frames will not control the resistance. However, global buckling of the two-girder system is possible and must be checked.

$L_s := 164$ ft Length of main span, not cantilever span

$S := 4 \frac{11}{12}$ ft Spacing between girders

Recall that steel elastic modulus $E_s = 29000000$ psi

Recall from Case 1 that strong-axis moment of inertia

$$I_x = 34436.1 \text{ in}^4$$

Recall from Case 1 that weak-axis moment of inertia

$$I_y = 681.9 \text{ in}^4$$

Recall from Case 1 that centroidal distance to top fiber

$$Y_t = 32.74 \text{ in}$$

Recall from Case 1 that centroidal distance to bottom fiber

$$Y_b = 28.89 \text{ in}$$

Recall from Case 1 that top flange dimensions are

$$b_{tf} = 16 \text{ in and } t_{tf} = 0.75 \text{ in}$$

Recall from Case 1 that bottom flange dimensions are

$$b_{bf} = 18 \text{ in and } t_{bf} = 0.875 \text{ in}$$

Thus centroidal distance to extreme tension fiber

$$t := Y_b = 28.89 \text{ in}$$

Thus centroidal distance to extreme compression fiber

$$c := Y_t = 32.74 \text{ in}$$

Thus weak-axis moment of inertia about tension flange

$$I_{yt} := \frac{t_{bf} \cdot b_{bf}^3}{12} = 425.3 \text{ in}^4$$

Thus weak-axis moment of inertia about compression flange

$$I_{yc} := \frac{t_{tf} \cdot b_{tf}^3}{12} = 256 \text{ in}^4$$

Thus the effective weak-axis moment of inertia

$$I_{\text{eff}} := I_{yc} + \left(\frac{t}{c}\right) \cdot I_{yt} = 631.2 \text{ in}^4$$

Global Buckling Resistance:

$$M_{gs} := \frac{\pi^2 \cdot S \cdot E_s}{L_s^2} \cdot \sqrt{I_{\text{eff}} \cdot I_x}$$

$$M_{gs} = 1694 \text{ kip ft} \quad \text{for the system}$$

$$M_{go} := \frac{M_{gs}}{2} = 874 \text{ kip ft}$$

for each girder in the twin-girder system

Recall, the maximum factored moment in Case 2 is $M_{ux} := 900$ kip ft for one girder.

$$\Phi_{ix} \cdot M_{go} = 762.3 \text{ kip ft} < M_{ux} = 900 \text{ kip ft} \quad \text{Flexural stability of two girders is not adequate.}$$

At the contractor's preference, top and bottom flange lateral bracing will be installed in the two panels at each end of Span 1. If the owner allows it, this lateral flange bracing can be permanent (i.e., drill holes in flanges and install angle iron). Or, if the owner prefers, this lateral flange bracing can be temporary and mechanically connected (beam clamps, cables and turnbuckles, adjustable straps, etc.)

Two girders After Release (Case 2), continued

The effect of the top and bottom flange lateral bracing will be to stiffen the ends of the span. Ultimately this changes the buckled shape K factor so that the twin girders buckle more like a column with fixed-ends rather than a pin-ended column. Conservatively this can be modeled with a global buckling mode effective span length L_{eff} which is equal to the clear span distance of the unstiffened panels. The new global buckling mode resistance $M_{\text{gs_eff}}$ will be computed based on this effective span.

$L_{\text{eff}} := 3 \cdot 25 \text{ ft} = 75 \text{ ft}$ with 2 end panels stiffened at the abutment and at the pier (out of 7 panels total)

$$M_{\text{gs_eff}} := \frac{\pi^2 \cdot S \cdot E_s}{L_{\text{eff}}^2} \cdot \sqrt{I_{\text{eff}} \cdot I_x} \quad M_{\text{gs_eff}} = 8100.1 \text{ kip ft for the system}$$

$$M_{\text{go_eff}} := \frac{M_{\text{gs_eff}}}{2} = 4050 \text{ kip ft for each girder in the twin-girder system}$$

$\Phi_{\text{fix}} \cdot M_{\text{go_eff}} = 3645 \text{ kip ft} > < M_{\text{ux}} = 900 \text{ kip ft}$ *Flexural stability of two stiffened girders is adequate.*

Lateral Bending from wind:

Recall, the maximum factored moment for Case 2 is $M_{\text{uy}} := 109 \text{ kip ft}$ for one girder at midspan.

Recall, the maximum factored axial load for Case 2 is $P_u := 64 \text{ kip}$ for one girder.

$A_g = 57.8 \text{ in}^2$ so axial stress $f := \frac{P_u}{A_g} = 1108.2 \text{ psi}$

By inspection, the axial stress is small enough that it can be ignored, so the moment-only interaction or strong-axis bending and weak-axis bending will be computed.

Interaction of strong-axis bending (system buckling) and weak-axis bending (yielding):

Unbraced length $L_b := 25 \text{ ft}$ with cross-frame spacing of 25 ft

Recall from Case 1 that web load shedding factor $R_b = 1$

Recall from Case 1 that effective radius of gyration $r_t = 3.8 \text{ in}$

Recall from Case 1 that limiting unbraced length $L_p = 7.7 \text{ ft}$

Recall from Case 1 that moment gradient modifier $C_b = 1$

Recall from Case 1 that compression flange width $b_{\text{tf}} = 16 \text{ in}$

Recall from Case 1 that weak-axis moment of inertia $I_y = 681.9 \text{ in}^4$

Recall from Case 1 strong-axis section modulus to compression flange $S_{\text{xc}} := S_{\text{tx}} = 1051.8 \text{ in}^3$

Recall that weak-axis factored moment $M_{\text{uy}} := 109 \text{ kip ft}$

Recall that strong-axis factored moment $M_{\text{ux}} := 900 \text{ kip ft}$

Stress in compression flange without consideration of lateral bending $f_{\text{buc}} := \frac{M_{\text{ux}}}{S_{\text{xc}}} = 10.3 \text{ ksi}$

First-order lateral bending stress in compression flange $f_{\text{L1c}} := \frac{M_{\text{uy}} \cdot \left(\frac{b_{\text{tf}}}{2}\right)}{I_y} = 15.3 \text{ ksi}$

Two Girders After Release (Case 2), continued

Limiting unbraced length for first-order lateral bending stress

$$1.2 \cdot L_p \cdot \sqrt{\frac{C_b \cdot R_b}{F_y}} = 20.4 \text{ ft} \quad (\text{Eq. 6.10.1.6-2})$$

Lateral bending stress:

elastic lateral torsional buckling stress

$$F_{cr} := \frac{C_b \cdot R_b \cdot \pi^2 \cdot E_s}{\left(\frac{L_b}{r_t}\right)^2} \quad F_{cr} = 47 \text{ ksi} \quad (\text{Eq. 6.10.8.2.3-8})$$

Check if first-order stress needs to be amplified: Approximated second-order lateral bending stress

$$f_{Lc} := \begin{cases} f_{Lc} & \text{if } L_b \leq 1.2 \cdot L_p \cdot \sqrt{\frac{C_b \cdot R_b}{F_y}} \\ \left(\frac{0.85 \cdot f_{Lc}}{1 - \frac{f_{Lc}}{F_{cr}}} \right) & \text{otherwise} \end{cases} \quad f_{Lc} = 16.7 \text{ ksi}$$

(Eq. 6.10.1.6-4)

Lateral bending check:

$$\text{Lateral_Bending_Resistance} := \begin{cases} \text{"Lateral Bending Requirements Satisfied"} & \text{if } f_{Lc} \leq 0.6F_y \\ \text{"Girder is NOT Adequate in Lateral Flexure"} & \text{otherwise} \end{cases}$$

$$\text{Lateral_Bending_Resistance} = \text{"Lateral Bending Requirements Satisfied"}$$

The following must be satisfied: $M_{ux} + \frac{1}{3} \cdot f_{Lc} \cdot S_{xc} \leq \phi_{fx} \cdot M_{gs}$ (Modified Eq. A6.1.1-1)

$$M_{ux} + \frac{1}{3} \cdot f_{Lc} \cdot S_{xc} = 1387.7 \text{ kip ft} < \phi_{fx} \cdot M_{go_eff} = 3645 \text{ kip ft}$$

Therefore interaction OK.

As an additional check an Eigenvalue analysis is included on Page B-41 of this example. This can utilized to provide a more detailed buckling analysis.

Note that if the designer lacked sufficient engineering judgement to recognize that the negative moment utilization would not govern for the Case 2 cantilever (because, with the same unbraced length of 19 ft, the negative moment, both in Case 5 and under traffic, is much higher), then the negative moment resistance could be calculated in the same fashion as the positive moment resistance was in Case 1 (but using the section properties of the pier girder segment). Also note that the moment gradient modifier C_b would be calculated per C6.4.10 as follows:

$$f_0 := 259 \text{ kip ft} \quad \text{at brace}$$

$$f_2 := 623 \text{ kip ft} \quad \text{at pier}$$

$$f_1 := f_0 \quad (\text{concave moment})$$

$$C_b := \min \left[1.75 - 1.05 \cdot \left(\frac{f_1}{f_2} \right) + 0.3 \cdot \left(\frac{f_1}{f_2} \right)^2, 2.3 \right] = 1.37 \quad \text{where } L_b := 19 \text{ ft and } M_{ux} := f_2 = 623 \text{ kip ft}$$

Adequacy of the Cross-Frame Bracing Member for Case 3 Loading

Cross-frame members act to transfer lateral winds to adjacent girders and their bearings during steel erection stages. The cross-frames also provide stability of the girder flanges during erection and placement of the deck. In skewed structures the cross-frames experience greater loading than non-skewed structures because the load in a girder will transfer through the cross-frame to an adjacent girder due to the closer proximity of the support on that adjacent girder. While deck pour loads may control the cross-frame design, for this example Case 3 will be examined with wind load acting on the fully erected steel superstructure.

Axial Tension Resistance of the Brace L4x4x3/8 (AASHTO 6.8.2):

$P_{ut} := 18.2$ kip Maximum tension force in any brace member from Case 3 analysis model.

Material / Section Properties (Taken from AISC manual) $F_y := 36$ ksi $F_u := 58$ ksi $A_g := 2.86$ in²

Note that A36 is still the preferred material specification for angles. Per AISC, availability of angles in other grades should be confirmed prior to their specification.

From Figure B1-3, it appears that the brace is welded rather than bolted to the conn. plate, so $A_n := A_g$

For shear lag reduction factor, lacking weld details, assume $U := 0.60$ (0.5 is worst case per 6.8.2.2)

Resistance factors $\phi_y := 0.95$ $\phi_u := 0.80$ (AASHTO 6.5.4.2)

Tension resistance for yielding $P_{ry} := \phi_y \cdot F_y \cdot A_g = 97.8$ kip (Eq. 6.8.2.1-1)

Tension resistance for fracture $P_{ru} := \phi_u \cdot F_u \cdot A_n \cdot U = 79.6$ kip (Eq. 6.8.2.1-2)

Axial Resistance Check $P_{ut} = 18.2$ kip $<$ $P_{rt} := \min(P_{ry}, P_{ru}) = 79.6$ kip Tension resistance is adequate

Axial Compression Resistance of the Brace L4x4x3/8 (AASHTO 6.9.4):

$P_{uc} := 4.4$ kip Maximum compression force in any brace member from Case 3 analysis model.

Material / Section Properties (Taken from AISC manual) $E_s := 29000$ ksi $F_y := 36$ ksi $A_g := 2.86$ in²

Check Slenderness of the Member (Sect. 6.9.4.2)

Following requirement needs to be satisfied for the element to qualify as nonslender:

$$\frac{b}{t} \leq k \cdot \sqrt{\frac{E}{F_y}} \quad (\text{Eq. 6.9.4.2.1-1}) \quad k := 0.45 \quad \text{from Table 6.9.4.2.1-1}$$

angle leg:

$$b := 4 \text{ in}$$

$$t := 0.375 \text{ in}$$

$$\frac{b}{t} = 10.7$$

$$k \cdot \sqrt{\frac{E_s}{F_y}} = 12.8$$

leg check:

Find Q if element is slender:

$$Q_s := \begin{cases} 1.34 - 0.76 \left(\frac{b}{t} \right) \cdot \sqrt{\frac{F_y}{E_s}} & \text{if } \frac{b}{t} \leq 0.91 \cdot \sqrt{\frac{E_s}{F_y}} \\ \frac{0.53 \cdot E_s}{F_y \cdot \left(\frac{b}{t} \right)^2} & \text{otherwise} \end{cases} \quad (\text{Eq. 6.9.4.2.2-5})$$

$$(\text{Eq. 6.9.4.2.2-6})$$

Adequacy of the Cross-Frame Bracing Member for Case 3 Loading, continued

$$Q_s = 1.1$$

$$Q_s := \begin{cases} 1.0 & \text{if } \frac{b}{t} \leq 0.45 \cdot \sqrt{\frac{E_s}{F_y}} \\ Q_s & \text{otherwise} \end{cases}$$

$$Q = 1$$

Determine Effective Slenderness Ratio $(KL/r)_{\text{eff}} = \lambda_{\text{eff}}$ (Sect. 6.9.4.4)

Recall $S = 59$ in $L := \max\left(S, \frac{S \cdot \sqrt{2}}{2}\right)$ S governs the brace length, so $L = 59$ in

$$r_x := 1.23 \text{ in so } \frac{L}{r_x} = 48$$

$$\lambda_{\text{eff}} := \begin{cases} \left(72 + 0.75 \frac{L}{r_x}\right) & \text{if } \frac{L}{r_x} \leq 80 \\ \left(32 + 1.25 \frac{L}{r_x}\right) & \text{otherwise} \end{cases} \quad (\text{Eq. 6.9.4.4-1})$$

$$\lambda_{\text{eff}} = 108$$

(Eq. 6.9.4.4-2)

Limiting KL/r for secondary compression members $\lambda_{\text{limit}} := 140$ (Sect. 6.9.3)

Maximum actual slenderness corresponds to minor principal axis buckling $r_z := 0.779$ in $K := 1$

$\frac{K \cdot L}{r_z} = 75.7 < \lambda_{\text{limit}} = 140$ Therefore, actual maximum slenderness ratio is adequate

Flexural Buckling Resistance

$$P_e := \frac{\pi^2 \cdot E_s}{(\lambda_{\text{eff}})^2} \cdot A_g \quad P_e = 70.2 \text{ kip} \quad (\text{Eq. 6.9.4.1.2-1})$$

Since the various conditions for single-angle members are satisfied as enumerated in AASHTO LRFD Sect. 6.9.4.4, the effective slenderness ratio can be calculated per that section; therefore, only flexural buckling resistance will be used to determine nominal compressive resistance of the brace. The effect of the eccentricities can be neglected when evaluated in this manner.

Equivalent Nominal Yield Resistance

$$P_o := Q \cdot F_y \cdot A_g \quad P_o = 103 \text{ kip} \quad (\text{Sect. 6.9.4.1.1})$$

$$\frac{P_e}{P_o} = 0.7$$

Nominal Compressive Resistance

$$P_n := \begin{cases} \left[0.658^{\left(\frac{P_o}{P_e}\right)} \cdot P_o\right] & \text{if } \frac{P_e}{P_o} \geq 0.44 \\ (0.877 P_e) & \text{otherwise} \end{cases} \quad (\text{Eq. 6.9.4.1.1-1})$$

$$P_n = 55.7 \text{ kip}$$

(Eq. 6.9.4.1.1-2)

Adequacy of the Cross-Frame Bracing Member for Case 3 Loading, continuedResistance factor $\phi_c := 0.9$ (AASHTO 6.5.4.2)

Factored Axial Resistance

$$P_{rc} := \phi_c \cdot P_n \quad P_{rc} = 50.2 \text{ kip} \quad (\text{Eq. 6.9.2.1-1})$$

Axial Resistance Check

$$P_{uc} = 4.4 \text{ kip} < P_{rc} = 50.2 \text{ kip} \quad \text{Compression resistance is adequate}$$

Verify Bracing Strength to Provide Girder StabilityUnbraced length $L_b := 25 \text{ ft} = 300 \text{ in}$ with cross-frame spacing of 25 ftSpan length $L := 164 \text{ ft} = 1968 \text{ in}$ Maximum moment within span $M_f := 805 \text{ kip ft} = 9660 \text{ kip in}$ Height of cross-frame $h_b := S = 59 \text{ in}$ Number of braces in span, excluding supports $n := 6$ Modulus of elasticity $E_s = 29000 \text{ ksi}$ Moment modification factor $C_b := 1$ Distance between flange centroids $h_o := d - \frac{t_{ff}}{2} - \frac{t_{bf}}{2} = 60.8 \text{ in}$ Recall girder cross-sectional properties $Y_b = 28.89 \text{ in}$ $Y_t = 32.74 \text{ in}$ $t_{ff} = 0.75 \text{ in}$ $t_{bf} = 0.875 \text{ in}$

Calculate effective minor axis moment of inertia

$$t := Y_b = 28.39 \text{ in} \quad c := Y_t = 32.74 \text{ in} \quad I_{yc} := \frac{t_{ff} \cdot b_{ff}^3}{12} = 256 \text{ in}^4 \quad I_{yt} := \frac{t_{bf} \cdot b_{bf}^3}{12} = 425.3 \text{ in}^4$$

$$I_{eff} := I_{yc} + \left(\frac{t}{c}\right) \cdot I_{yt} = 631.2 \text{ in}^4 \quad (\text{Manual Eq. D-5.2j})$$

$$\text{Recall } I_y = 681.9 \text{ in}^4 \quad \text{Therefore } \frac{I_{eff}}{I_y} = 0.926$$

$$\text{Required strength } F_{br} := \frac{0.005 L_b \cdot L \cdot M_f^2}{h_b \cdot n \cdot E_s \cdot I_{eff} C_b^2 \cdot h_o} = 0.7 \text{ kip} \quad (\text{Manual Eq. D-5.2a})$$

$$\text{Length of diagonal member } L_c := S \cdot \sqrt{2} = 83.44 \text{ in}$$

Required strength of compression brace $F_{brc} := F_{br} = 0.7 \text{ kip}$ (Manual Fig. 5-7, Tension System)Required strength of tension brace $F_{brt} := \frac{2F_{br} \cdot L_c}{S} = 1.98 \text{ kip}$ (Manual Fig 5-7, Tension System)Available strength of tension brace to resist stability force $P_{rt} - P_{ut} = 61.42 > F_{brt} = 1.98 \text{ kip}$ Available strength of compression brace to resist stability force $P_{rc} - P_{uc} = 45.76 \text{ kip} > F_{brc} = 0.7 \text{ kip}$

Therefore, bracing is adequate for strength

Adequacy of the Cross-Frame Bracing Member for Case 3 Loading, continued**Check Required Stiffness of Bracing System $\beta_{T,reqd}$** Resistance factor $\phi_{br} := 0.75$

$$\text{Required stiffness } \beta_{T,reqd} := \frac{2.4 \cdot L \cdot M_f^2}{\phi_{br} \cdot n \cdot E_s \cdot I_{eff} \cdot C_b^2} = 5350.4 \text{ kip in} \quad (\text{Manual Eq. D-5.2b})$$

Calculate Attached Brace Stiffness β_b Area of diagonal member $A_c := A_g = 2.86 \text{ in}^2$ Area of horizontal member $A_h := A_g = 2.86 \text{ in}^2$

$$\text{Attached brace stiffness } \beta_b := \frac{E_s \cdot S^2 \cdot h_b^2}{\left(\frac{2 \cdot L_c^3}{A_c}\right) + \frac{S^3}{A_h}} = 735101 \text{ kip in} \quad (\text{Manual Fig. 5-7, Eq. for Tension System})$$

Calculate Web Distortional Stiffness β_{sec} Recall web thickness $t_w = 0.5 \text{ in}$ Intermediate stiffener plate thickness $t_s := 0.5 \text{ in}$ Intermediate stiffener plate width $b_s := 7 \text{ in}$

$$\text{Web distortional stiffness } \beta_{sec} := 3.3 \cdot \frac{E_s}{h_o} \left[\frac{(1.5 \cdot h_o) \cdot t_w^3}{12} + \frac{t_s \cdot b_s^3}{12} \right] = 23986 \text{ kip in} \quad (\text{Manual Eq. D5.2h})$$

However, for a full depth cross-frame, $\beta_{sec} := 999999999999 \text{ kip in}$ (infinity)**Calculate In-Plane Girder System Stiffness β_g** Number of girders $n_g := 6$ Recall major axis moment of inertia $I_x = 34436.1 \text{ in}^4$

$$\text{Girder system stiffness } \beta_g := \frac{24 \cdot (n_g - 1)^2 \cdot S^2 \cdot E_s \cdot I_x}{n_g \cdot L^3} = 45608 \text{ kip in} \quad (\text{Manual Eq. D5.2i})$$

Calculate Total Provided System Stiffness β_T and Compare with Required System Stiffness $\beta_{T,reqd}$

$$\text{Total provided system stiffness } \beta_T := \frac{1}{\left(\frac{1}{\beta_b} + \frac{1}{\beta_{sec}} + \frac{1}{\beta_g}\right)} = 42943.6 \text{ kip in} \quad (\text{Rearranged Manual Eq. D5.2g})$$

$B_T = 42943.6 \text{ kip in} > \beta_{T,reqd} = 5350.4 \text{ kip in}$ Therefore, stiffness of bracing system is adequate.

Note that the controlling case would be the deck pour (Case 5) when moment in the girder is greatest.

$$\text{Thus } \beta_{T,reqd5} := \left(\frac{2227 \text{ kip ft}}{805 \text{ kip ft}}\right)^2 \cdot \beta_{T,reqd} = 40947.9 \text{ kip in} < \beta_T \text{ Therefore, still OK.}$$

Fascia Girder During Deck Pour (Case 5)

Note: Case 4 was analyzed but does not govern, so capacity calculations for that case are not performed in this example. For Case 5 loading, the girder span segment controls, rather than the pier segment.

Section Properties:

$$E_s := 29000 \text{ ksi} \quad F_y := 50 \text{ ksi}$$

$$\text{Flanges:} \quad t_{ff} := 0.75 \text{ in} \quad b_{ff} = 16 \text{ in} \quad t_{bf} := 0.875 \text{ in} \quad b_{bf} := 18 \text{ in}$$

$$\text{Web:} \quad t_w := 0.5 \text{ in} \quad D := 60 \text{ in} \quad \text{Overall Depth: } d := D + t_{ff} + t_{bf} \quad d = 61.6 \text{ in}$$

Calculated Properties: (Note that the steps involved in calculating these properties are omitted)

$$S_{lx} := 1051.8 \text{ in}^3 \quad S_{bx} := 1192.2 \text{ in}^3 \quad Y_t := 32.74 \text{ in} \quad Y_b := 28.89 \text{ in} \quad S_y := 75.8 \text{ in}^3 \quad Z_x := 1288.4 \text{ in}^3$$

$$A_g := 57.75 \text{ in}^2 \quad r_x := 24.42 \text{ in} \quad r_y := 3.44 \text{ in} \quad I_x := 34436.1 \text{ in}^4 \quad I_y := 681.9 \text{ in}^4 \quad Z_x := 124.5 \text{ in}^3$$

Applied Factored Forces (From Deck Pour Analysis Output Summary):

$$\text{Major-axis Bending} \quad M_{ux} := 2227 \text{ kip ft} \quad (\text{Strength VI})$$

$$\text{Lateral Bending:} \quad M_{uy} := 1 \text{ kip ft} \quad (\text{Strength VI})$$

$$\text{Unbraced Length:} \quad L_b := 25 \text{ ft} \quad \text{with cross-frame spacing of 25 ft}$$

Lateral Force on Ea. Flange from Overhang Bracket at Fascia Girder: $F_L := 0.150 \text{ klf}$ (Assumed, see Page B-39 for verification)

Lateral Moment in Each Flange from Overhang Bracket at Fascia Girder:

$$M_L := \frac{F_L \cdot L_b^2}{12} \quad (\text{Eq. C6.10.3.4-2})$$

Factored Lat. Moment in Ea. Flange from Overhang Bracket at Fascia Girder: $M_{Lu} := 1.50 M_L = 11.7 \text{ kip ft}$ (conservatively assume highest load factor corresponding to all construction dead load)

Flexural Resistance

$$\text{Top flange in compression:} \quad b_{fc} := b_{ff} \quad t_{fc} := t_{ff}$$

$$\text{Bottom flange is in tension:} \quad b_{ft} := b_{bf} \quad t_{ft} := t_{bf}$$

Applied Stresses:

$$\text{Top flange is in compression:} \quad S_{yc} := \frac{t_{fc} \cdot b_{fc}^2}{6} \quad S_{yc} := 32 \text{ in}^3$$

$$\text{Bottom flange is in tension:} \quad S_{yt} := \frac{t_{ft} \cdot b_{ft}^2}{6} \quad S_{yt} := 47.3 \text{ in}^3$$

$$\text{Stress in compression flange without consideration of lateral bending:} \quad f_{buc} := \frac{M_{ux}}{S_{tx}} \quad f_{buc} = 25.4 \text{ ksi}$$

$$\text{First-order-stress due to lateral bending in compression flange:} \quad f_{L1c} := \frac{M_{uy} \cdot (0.5 \cdot b_{fc})}{I_y} + \frac{M_{Lu}}{S_{yc}} \quad f_{L1c} = 4.5 \text{ ksi}$$

$$\text{Stress in tension flange without consideration of lateral bending:} \quad f_{but} := \frac{M_{ux}}{S_{bx}} \quad f_{but} = 22.4 \text{ ksi}$$

First-order stress due to lateral bending
in tension flange:

$$f_{Lt} := \frac{M_{ly} \cdot (0.5 \cdot b_{ft})}{I_y} + \frac{M_{Lu}}{S_{yt}} \quad f_{Lt} = 3.1 \text{ ksi}$$

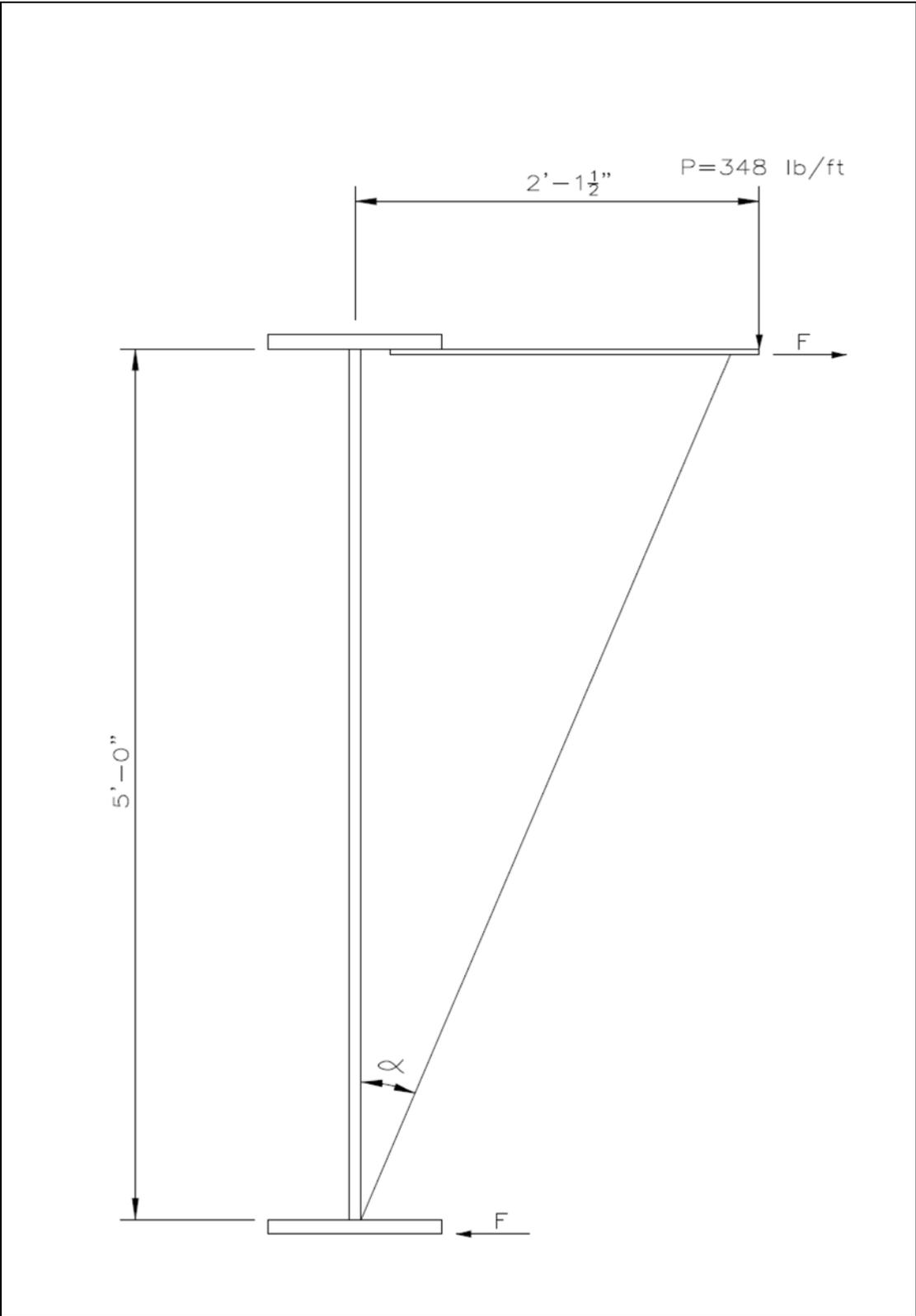


Figure B1-16: Example 1 Overhang Bracket Loading

Fascia Girder During Deck Pour (Case 5), continued*Flange Strength Reduction Factors:*

Hybrid Factor (AASHTO 6.10.1.10.1)

Since the flexural member is a homogenous built-up section, the hybrid factor shall be taken as unity

$$R_h := 1$$

Web Load Shedding Factor (AASHTO 6.10.1.10.2)

Depth of web in compression $D_c := Y_t - t_{ff}$

$$\lambda_{rw} := \left(5.7 \cdot \sqrt{\frac{E_s}{F_y}} \right) \quad \lambda_{rw} = 137.3 \quad \frac{2 \cdot D_c}{t_w} = 128$$

$$a_{wc} := \frac{2D_c \cdot t_w}{b_{fc} \cdot t_{fc}}$$

$$R_b := \begin{cases} 1.0 & \text{if } \frac{2 \cdot D_c}{t_w} \leq \lambda_{rw} \\ \left[1.0 - \left(\frac{a_{wc}}{1200 + 300 \cdot a_{wc}} \right) \cdot \left(\frac{2 \cdot D_c}{t_w} - \lambda_{rw} \right) \right] & \text{otherwise} \end{cases} \quad R_b = 1$$

Web is non-compact, so Eq. 6.10.3.2.1-3 need not be checked per AASHTO 6.10.3.2.1.

Local Buckling Resistance (AASHTO 6.10.8.2.2):

Slenderness ratio of the compression flange

$$\lambda_f := \frac{b_{fc}}{2 \cdot t_{fc}} \quad \lambda_f = 10.7 \quad (\text{Eq. 6.10.8.2.2-3})$$

$$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E_s}{F_y}} \quad \lambda_{pf} = 9.2 \quad (\text{Eq. 6.10.8.2.2-4})$$

$$\lambda_{rf} := 0.56 \cdot \sqrt{\frac{E_s}{0.7F_y}} \quad \lambda_{rf} = 16.1 \quad (\text{Eq. 6.10.8.2.2-5})$$

Local Buckling Resistance

$$F_{nc1} := \begin{cases} (R_b \cdot R_h \cdot F_y) & \text{if } \lambda_f \leq \lambda_{pf} \\ \left[\left[1 - \left(1 - \frac{0.7F_y}{R_h \cdot F_y} \right) \cdot \frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right] \cdot R_b \cdot R_h \cdot F_y \right] & \text{otherwise} \end{cases} \quad (\text{Eq. 6.10.8.2.2-1})$$

$$F_{nc1} = 46.7 \text{ ksi}$$

$$(\text{Eq. 6.10.8.2.2-2})$$

Fascia Girder During Deck Pour (Case 5), continued**Lateral Torsional Buckling Resistance (AASHTO 6.10.8.2.3):**Unbraced length $L_b := 25$ ft with cross-frame spacing of 25 ftEffective Radius
of Gyration

$$r_t := \frac{b_{fc}}{\sqrt{12 \cdot \left(1 + \frac{1}{3} \cdot \frac{D_c \cdot t_w}{b_{fc} \cdot t_{fc}}\right)}} \quad r_t = 3.8 \quad (\text{Eq. 6.10.8.2.3-9})$$

Limiting Unbraced Length Calculations:

$$L_p := r_t \cdot \sqrt{\frac{E_s}{F_y}} \quad L_p = 7.7 \text{ ft} \quad (\text{Eq. 6.10.8.2.3-4})$$

$$L_r := \pi r_t \cdot \sqrt{\frac{E_s}{0.7F_y}} \quad L_r = 29 \text{ ft} \quad (\text{Eq. 6.10.8.2.3-5})$$

$$\text{Moment gradient modifier} \quad C_b := 1 \quad \text{since, } \frac{f_{mid}}{f_2} > 1$$

Lateral Torsional Buckling Resistance

$$F_{nc2} := \begin{cases} (R_b \cdot R_h \cdot F_y) & \text{if } L_b \leq L_p \\ C_b \cdot \left[1 - \left(1 - \frac{0.7F_y}{R_h \cdot F_y}\right) \cdot \left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \cdot R_b \cdot R_h \cdot F_y & \text{if } L_p < L_b \leq L_r \\ \frac{C_b \cdot R_b \cdot \pi^2 \cdot E_s}{\left(\frac{L_b}{r_t}\right)^2} & \text{otherwise} \end{cases} \quad (\text{Eq. 6.10.8.2.3-1})$$

$$F_{nc2} = 37.8 \text{ ksi}$$

(Eq. 6.10.8.2.3-2)

(Eq. 6.10.8.2.3-3)

Controlling Nominal Flexural Resistance:

$$F_{nc} := \min(F_{nc1}, F_{nc2}) \quad F_{nc} = 37.8 \text{ ksi}$$

$$\text{Resistance factor} \quad \Phi_f := 1.0 \quad (\text{AASHTO 6.5.4.2})$$

$$\Phi_f \cdot F_{nc} = 37.8 \text{ ksi}$$

Fascia Girder During Deck Pour (Case 5), continued**Determine Stress due to Lateral Bending:**

First-order lateral bending stress (from previous): $f_{L1c} = 4.5$ ksi

Limiting unbraced length for first-order lateral bending stress

$$1.2 \cdot L_p \cdot \sqrt{\frac{C_b \cdot R_b}{F_y} \cdot \frac{f_{buc}}{F_y}} = 13 \text{ ft} \quad (\text{Eq. 6.10.1.6-2})$$

Lateral bending stress:

elastic lateral torsional buckling stress

$$F_{cr} := \frac{C_b \cdot R_b \cdot \pi^2 \cdot E_s}{\left(\frac{L_b}{r_t}\right)^2} \quad F_{cr} = 47 \text{ ksi} \quad (\text{Eq. 6.10.8.2.3-8})$$

Check if the first-order stress needs to be amplified:

Approximated second-order lateral bending stress:

$$f_{Lc} := \begin{cases} f_{L1c} & \text{if } L_b \leq 1.2 \cdot L_p \cdot \sqrt{\frac{C_b \cdot R_b}{F_y} \cdot \frac{f_{buc}}{F_y}} \\ \left(\frac{0.85}{1 - \frac{f_{buc}}{F_{cr}}} \right) \cdot f_{L1c} & \text{otherwise} \end{cases} \quad f_{Lc} = 8.4 \text{ ksi}$$

(Eq. 6.10.1.6-4)

Lateral bending check:

$$\begin{array}{l} \text{Lateral_Bending_Resistance} \\ \text{Lateral_Bending_Resistance} \end{array} \quad \left\| \begin{array}{l} \text{:= "Lateral Bending Requirements Satisfied" if } f_{Lc} \leq 0.6F_y \\ \text{"Girder is NOT Adequate in Lateral Flexure" otherwise} \\ \text{= "Lateral Bending Requirements Satisfied"} \end{array} \right.$$

Overall Flexural Resistance Check:

The following must be satisfied:

$$f_{buc} + \frac{1}{3} \cdot f_{Lc} \leq \phi_f \cdot F_{nc} \quad (\text{Eq. 6.10.3.2.1-2})$$

$$f_{buc} + \frac{1}{3} \cdot f_{Lc} = 28.2 \text{ ksi} \quad \phi_f \cdot F_{nc} = 37.8 \text{ ksi}$$

$$\text{Resistance_Check} := \left\| \begin{array}{l} \text{"Girder Meets AASHTO Flexural Requirements" if } f_{buc} + \frac{1}{3} \cdot f_{Lc} \leq \phi_f \cdot F_{nc} \\ \text{"Girder is NOT Adequate in Lateral Flexure" otherwise} \end{array} \right.$$

$$\text{Resistance_Check} = \text{"Girder Meets AASHTO Flexural Requirements"}$$

Fascia Girder During Deck Pour (Case 5), continued**Constructability Check:**

The following must be satisfied:

$$\begin{aligned} f_{\text{buc}} + f_{\text{Lc}} &\leq \Phi_f \cdot R_h \cdot F_y && \text{(Eq. 6.10.3.2.1-1)} \\ f_{\text{buc}} + f_{\text{Lc}} = 33.8 \text{ ksi} &< \Phi_f \cdot R_h \cdot F_y = 50 \text{ ksi} \\ &\text{Adequate resistance in compression flange} \end{aligned}$$

The following must be satisfied:

$$\begin{aligned} f_{\text{but}} + f_{\text{Lt}} &\leq \Phi_f \cdot R_h \cdot F_y && \text{(Eq. 6.10.3.2.2-1)} \\ f_{\text{but}} + f_{\text{Lt}} = 25.6 \text{ ksi} &< \Phi_f \cdot R_h \cdot F_y = 50 \text{ ksi} \\ &\text{Adequate resistance in tension flange} \end{aligned}$$

Overhang Bracket Forces:

Assume 2" edge distance at parapets. Therefore $\text{deck_width} := \left[\left(29 + \frac{2}{12} \right) \text{ft} - \frac{2}{12} \text{ft} - \frac{2}{12} \text{ft} \right]$

$$\text{overhang} := \frac{\text{deck_width} - \left(24 + \frac{7}{12} \right) \text{ft}}{2} \quad \text{overhang} = 2.125 \text{ ft} \quad \text{Recall } D = 60 \text{ in}$$

Assume 10" thick deck at fascia overhang; ½ of weight goes to bracket and ½ directly to girder

$$W_{\text{fascia}} := 0.5 \text{ overhang } 10 \text{ in } 150 \text{ pcf} \quad W_{\text{fascia}} = 132.8 \text{ plf} \quad \text{Concrete weight to bracket}$$

Assume 200 plf for screed rail at fascia overhang $W_{\text{rail}} := 200 \text{ plf}$ Rail weight to bracket

Assume 10 psf for forms and another 5 psf for bracket components and miscellaneous at fascia overhang; ½ of weight goes to bracket and ½ directly to girder

$$W_{\text{forms}} := 0.5 \text{ overhang } (10 \text{ psf} + 5 \text{ psf}) \quad W_{\text{forms}} = 15.9 \text{ plf} \quad \text{Form weight to bracket}$$

$$W_{\text{bracket}} := W_{\text{fascia}} + W_{\text{rail}} + W_{\text{forms}} \quad W_{\text{bracket}} = 348.7 \text{ plf} \quad \text{Total uniform load to bracket}$$

$$\alpha := \text{atan} \left(\frac{\text{overhang}}{D} \right) \quad \alpha = 23.03 \text{ deg} \quad \text{Angle of bracket relative to fascia girder web}$$

$$F_L := W_{\text{bracket}} \cdot \tan(\alpha) \quad F_L = 148.2 \text{ plf} \quad \text{Lateral force on flange from bracket}$$

Recall that 150 plf was assumed for lateral force on each flange from bracket in computing lateral moments and flange stresses. Therefore, previous calculations are valid.

Comparison of Results for Eigenvalue Buckling Analysis from UT Bridge (Case 1)

Recall, for a single girder (Case 1), the factored moment demand $M_{ux} := 762$ kip ft

This included a load factor of 1.25 for Dead Load.

So, the applied unfactored moment $M_x := \frac{M_{ux}}{1.25} = 609.6$ kip ft

Per the UT Bridge analysis (unfactored), the Eigenvalue for Case 1 is 0.8941.

This is equivalent to a load factor, meaning that the ultimate resistance $\phi M_n := 0.8941 M_x = 545$ kip ft

Recall, for a single girder (Case 1), the lateral-torsional buckling resistance $M_{rx} := 95.7$ kip ft

$$\frac{\phi M_n}{M_{rx}} = 5.695 \text{ (off by 470\%)}$$

It appears that the calculated design strength does not reasonably approximate the Eigenvalue analysis shown below for this buckling mode. For the reasons behind this, see the discussion in Manual Section 6.6. Note that the Eigenvalue is less than 1, which indicates that the single girder case is unstable under its own selfweight. The computed AASHTO design strength for lateral-torsional buckling also leads to this conclusion since $M_{rx} \ll M_{ux}$.

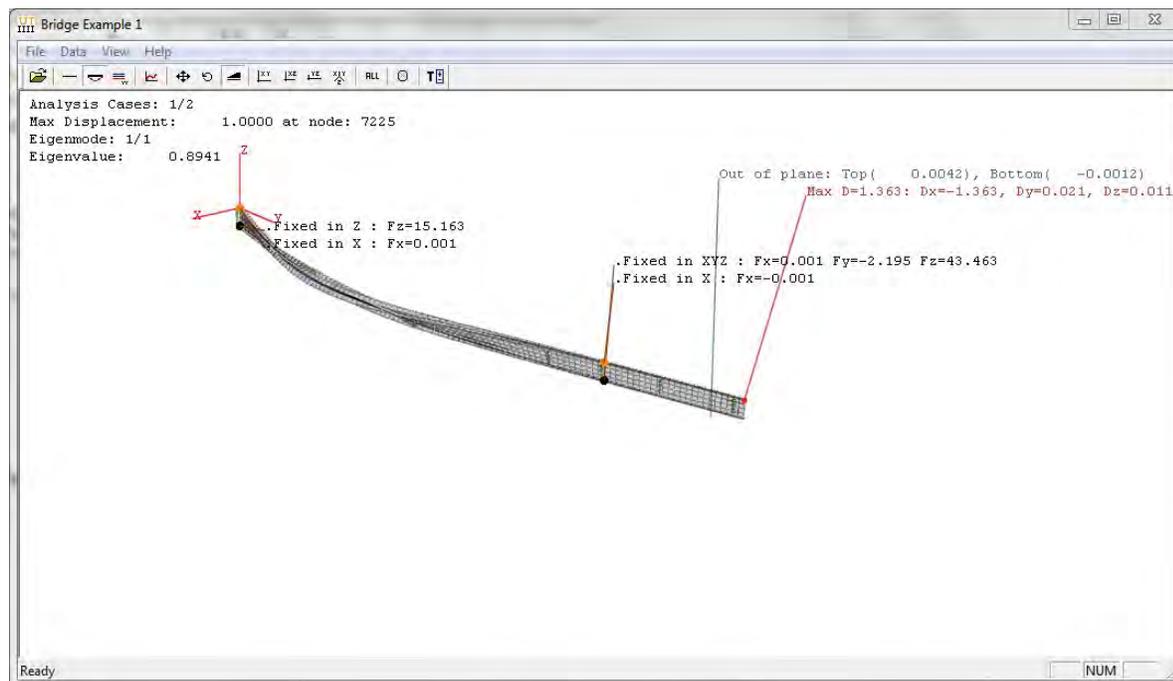


Figure B1-17: Case 1 Eigenvalue Analysis

Comparison of Results for Eigenvalue Buckling Analysis from UT Bridge (Case 2)

Recall, for a two-girder system (Case 2), the factored moment demand $M_{ux} := 900 \text{ kip ft}$

This included a load factor of 1.25 for Dead Load.

So, the applied unfactored moment $M_x := \frac{M_{ux}}{1.25} = 720 \text{ kip ft}$

Per the UT Bridge analysis (unfactored), the Eigenvalue for Case 2 is 2.1489.

This is equivalent to a load factor, meaning that the ultimate resistance:

$\Phi M_n := 2.1489 M_x = 1547.2 \text{ kip ft per girder}$

Recall, for a two-girder system (Case 2), the system buckling resistance:

$M_{rx} := 762.3 \text{ kip ft}$

$\frac{\Phi M_n}{M_{rx}} = 2.03$ (off by 103%)

It appears that the calculated design strength does not reasonably approximate the Eigenvalue analysis shown below for this buckling mode. This is likely due to the omission of the C_b factor in the calculation for global buckling resistance (i.e., implied $C_b = 1.0$). By contrast, the Eigenvalue analysis automatically takes into account the moment gradient. Note that the Eigenvalue here is greater than 1.75, which should still provide an adequate margin of safety against buckling. However, as discussed previously, based on the calculated system buckling resistance, the contractor chose to install top and bottom flange lateral bracing in the end panels of the span to avoid any potential problems.

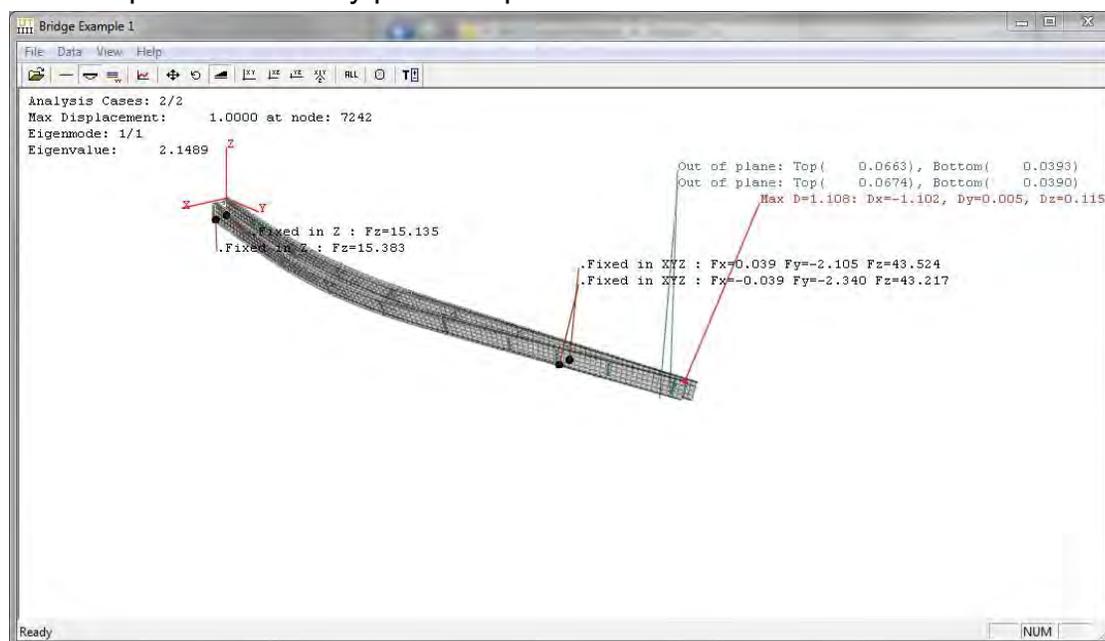


Figure B1-18: Case 2 Eigenvalue Analysis

Summary

This example has illustrated many of the required checks to verify the adequacy of the erection and construction plan. While this example selected specific cases as a summary of required checks, the erection engineer will be required to provide the necessary checks are all critical stages of the construction. As in this example, the first girder is often inadequate to support itself on bearings without additional temporary supports. In some cases, two girders set on bearings do not meet buckling criteria and it is important to always include the two girder buckling analysis.

The construction contractor and resident engineer are responsible to observe the structure during erection. If any unusual deflections in the field are observed, work should stop immediately and the erection engineer should be consulted.

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Notes

Reader Notes

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Reader Notes

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EXAMPLE PROBLEM #2: 3-SPAN MULTI-GIRDER CURVED STEEL STRUCTURE

This example problem expands upon Example Problem #1 to examine the effects of the curvature in the plate girders during steel bridge construction. While Example Problem #1 incorporated the use of a commercial 3D finite element analysis program which are typically utilized in engineering offices, this problem focuses on analysis using UT Lift and UT Bridge. These free bridge erection analysis programs were developed by the University of Texas. This problem is not intended to be comprehensive, but instead a look at the analysis and output capabilities of UT Lift and UT Bridge focusing on the effects of curvature.

The following table of contents illustrates the general categories into which this example problem is subdivided, and the relevant page number at which the start of each category, and its specific component analysis or calculation tasks, may be found.

General Category / Description	Specific Analysis / Calculation Task	Appendix Page
Table of Contents	N/A	B.51
Bridge Data	N/A	B.52
Framing Plan Details	N/A	B.54
Applied Forces	Wind Load	B.59
Lifting Analysis	Single Girder Pick with Interior Cross-frames	B.61
Figure	UT Lift Output	B.62
Lifting Analysis	Single Girder Pick without Cross-frames	B.68
Figure	UT Lift Output	B.69
Lifting Analysis	Single Girder Pick with Exterior Cross-frames	B.75
Figure	UT Lift Output	B.76
Lifting Analysis	Single Girder Pick without Cross-frames	B.82
Figure	UT Lift Output	B.83
Lifting Analysis	Single Girder Pick without Cross-frames	B.89
Figure	UT Lift Output	B.90
Erection Analysis	Two Girder System	B.96
Erection Analysis	Four Girder System	B.101
Erection Analysis	Multi-Span Girder System	B.104
Erection Analysis	Bearing Capacity	B.105
Deck Pour Analysis	UT Bridge Output	B.106
Deck Pour Analysis	AASHTO Resistance Calculations	B.109
Deck Pour Analysis	Cross-Frame Capacity	B.117
Erection Analysis	Splice Capacity with 50% Bolts	B.121
Erection Analysis	Holding Crane	B.125

Bridge Data:

Continuous 3 span curved steel plate girder bridge, 4 girders total. Radius of curvature is 700 ft at centerline of roadway. Out-to-out deck width is 40.5 ft. Girder spacing is 11 ft for a total of 33 ft between fascia girders. Structural deck slab thickness is 9 in. with 0.5 in. integral wearing surface.

The spans are symmetric, with Spans 1 and 3 at 160 ft long and Span 2 at 210 ft long measured along the centerline of structure.

The field splices are also symmetric, with Field Section 1 at 120 ft long in Spans 1 and 3 (at Abutments 1 and 2), Field Section 2 at approximately 78.15 ft long (centered on Piers 1 and 2), and Field Section 3 at approximately 133.65 ft long in the middle of Span 2. Since these measurements are taken at the centerline of structure, where there is no physical girder, the actual field section lengths vary depending on which girder is considered.

Erection Analysis:

A single curved girder pick is modeled with UT Lift 1.3 to determine lift point locations and resulting forces in girder. For this example, Field Section 3 (136.8 ft long) of the exterior girder G4 is chosen for the lifting analysis. 10% additional weight is assumed to account for connection material, stiffeners, etc. In computing the weight of the attached cross-frames, an L8x6x9/16 is assumed.

Therefore,

$$A_g := 7.56 \text{ in}^2 \text{ for L8x6x9/16}$$

$$W_x := 2 \cdot 490 \frac{\text{lb}}{\text{ft}^3} A_g \cdot \left[11 \text{ ft} + \sqrt{(11 \text{ ft})^2 + (7 \text{ ft})^2} \right] = 1236.8 \text{ lb}$$

per cross-frame (2 horizontals & 2 diagonals)

Several stages of the bridge erection are examined utilizing UT Bridge to determine the adequacy of the steel system under self weight and wind loads. A two girder, four girder, and multi-span system are analyzed during different stages of the erection sequence.

The capacity of the splice with 50% bolts is examined during steel erection. Bearing capacities during erection and deck pour in cases of high rotation is also discussed, however specific bearing details are not included in this example.

Deck Pour Analysis:

For this example, it is assumed the deck pour will run in four casts. The first cast is in Span 3 starting at Abutment 2 and ending at the field splice. The second cast is in Span 2 between field splices. The third cast is in Span 1 from the field splice to Abutment 1. The fourth cast is over Piers 1 and 2. The worst case stage for the Span 3 girder segment is when the deck has been poured over Span 3 up to the splice (first cast). This stage will be examined for girder adequacy. The concrete weight should be taken as 150 pcf and treated as permanent dead load, while construction dead load, the weight of the forms, should be taken at 10 psf. Construction live load should be taken as 20 psf in Spans 2 and 3.

The pour sequence is modeled in UT Bridge, which does not allow for the form weight or construction live load, so those are ignored. Wind load is assumed to be negligible during the pour and is also ignored. The factored results used in the calculations are from the Strength I: 1.25 DC + 1.50 CDL + 1.50 CLL load combination. The Strength VI: 1.40 DC + 1.40 CDL + 1.40 CLL load combination should also be analyzed; however, it can be seen from the Strength I results that the design will still be adequate for Strength VI, since at most there will be a 12% increase in applied stresses (1.40/1.25), and all the allowable stresses are significantly higher than the applied stresses.

Evaluation of Stages:

The girders will be checked for adequacy according to AASHTO 2012 LRFD Specifications. The G4 girder (Field Section 3) will be checked for stability as well as bending capacity during the pick. The G4 girder (Field Section 1) will be checked for the deck pour loading. While these cases may not represent the critical loading for each girder segment, they serve as an example to check for adequacy. The erection stages will be checked for displacements and buckling eigenvalues utilizing UT Bridge. Note that many necessary checks are not performed here as they are conceptually similar to the calculations shown in Example 1 for straight plate girders.

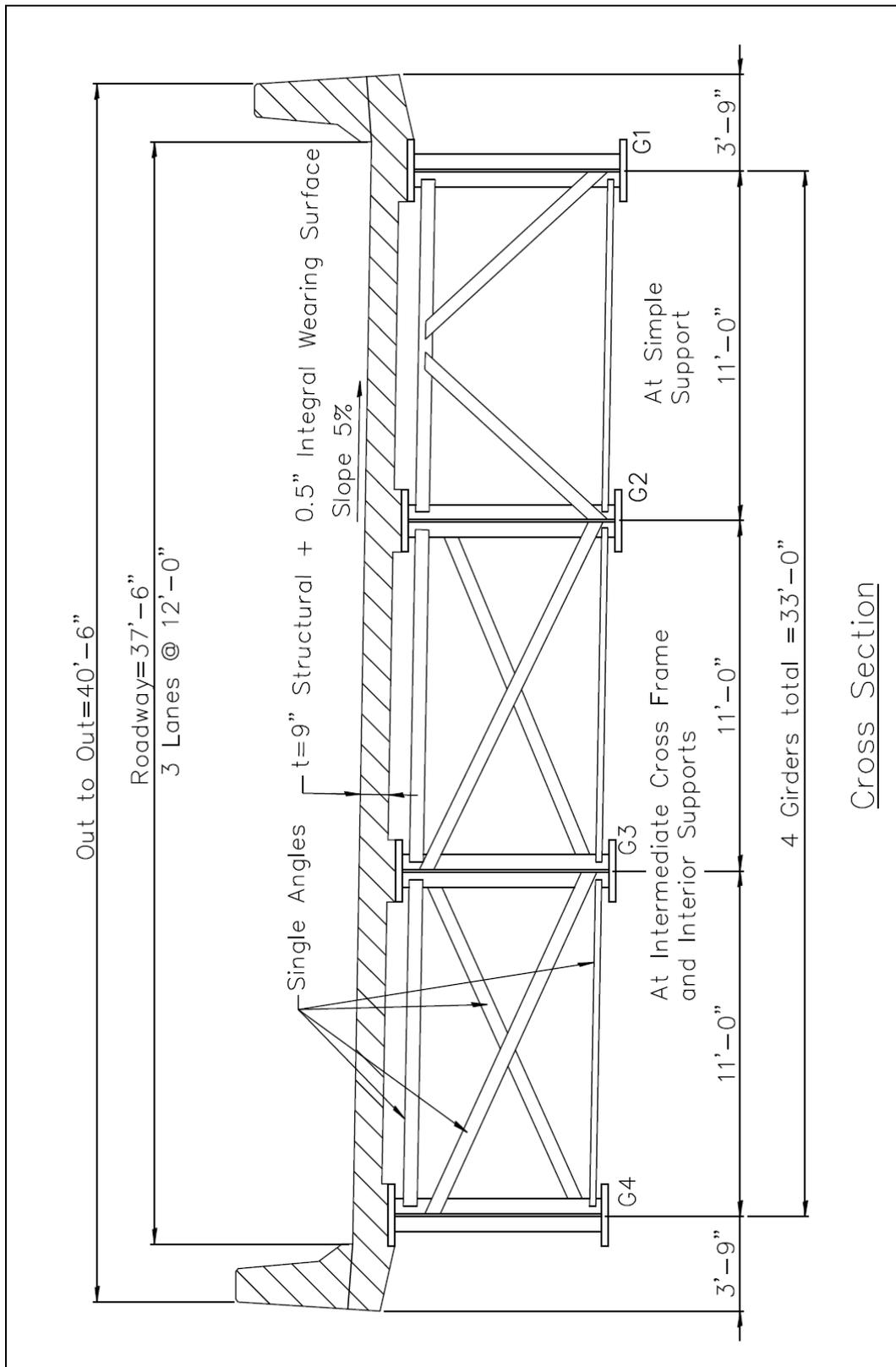


Figure B2-1: Example 2 Cross-section

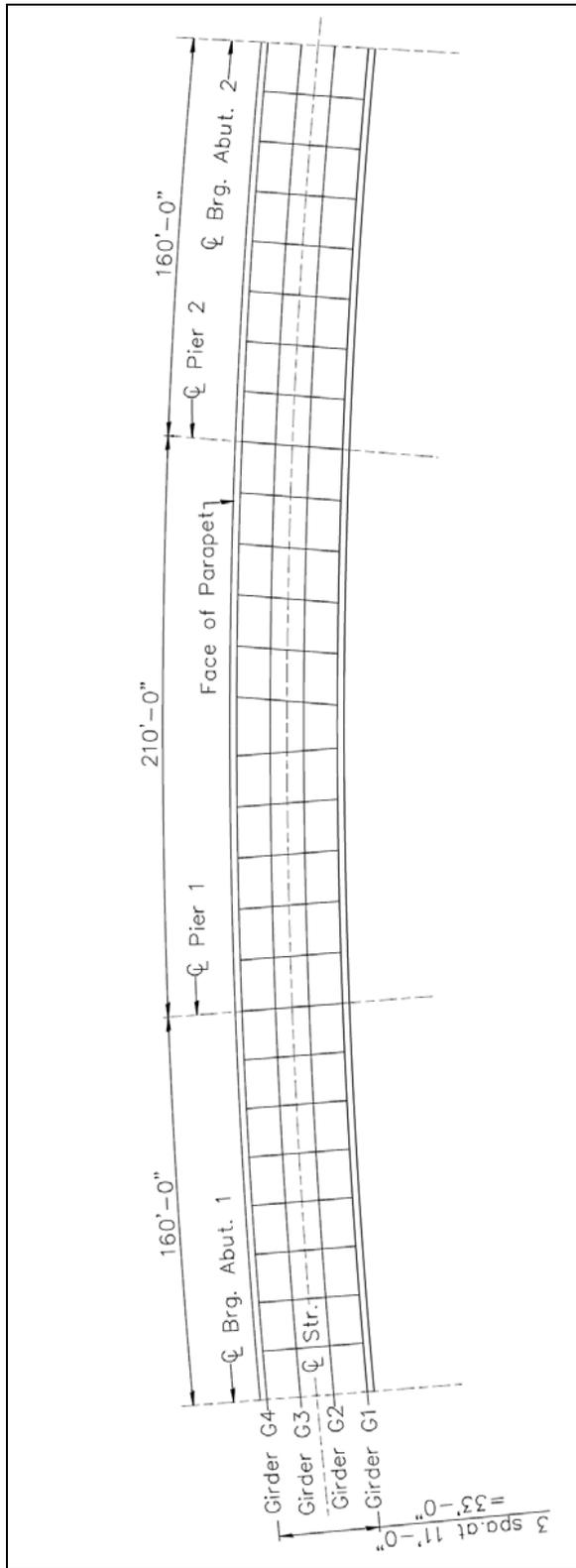


Figure B2-2: Example 2 Framing Plan

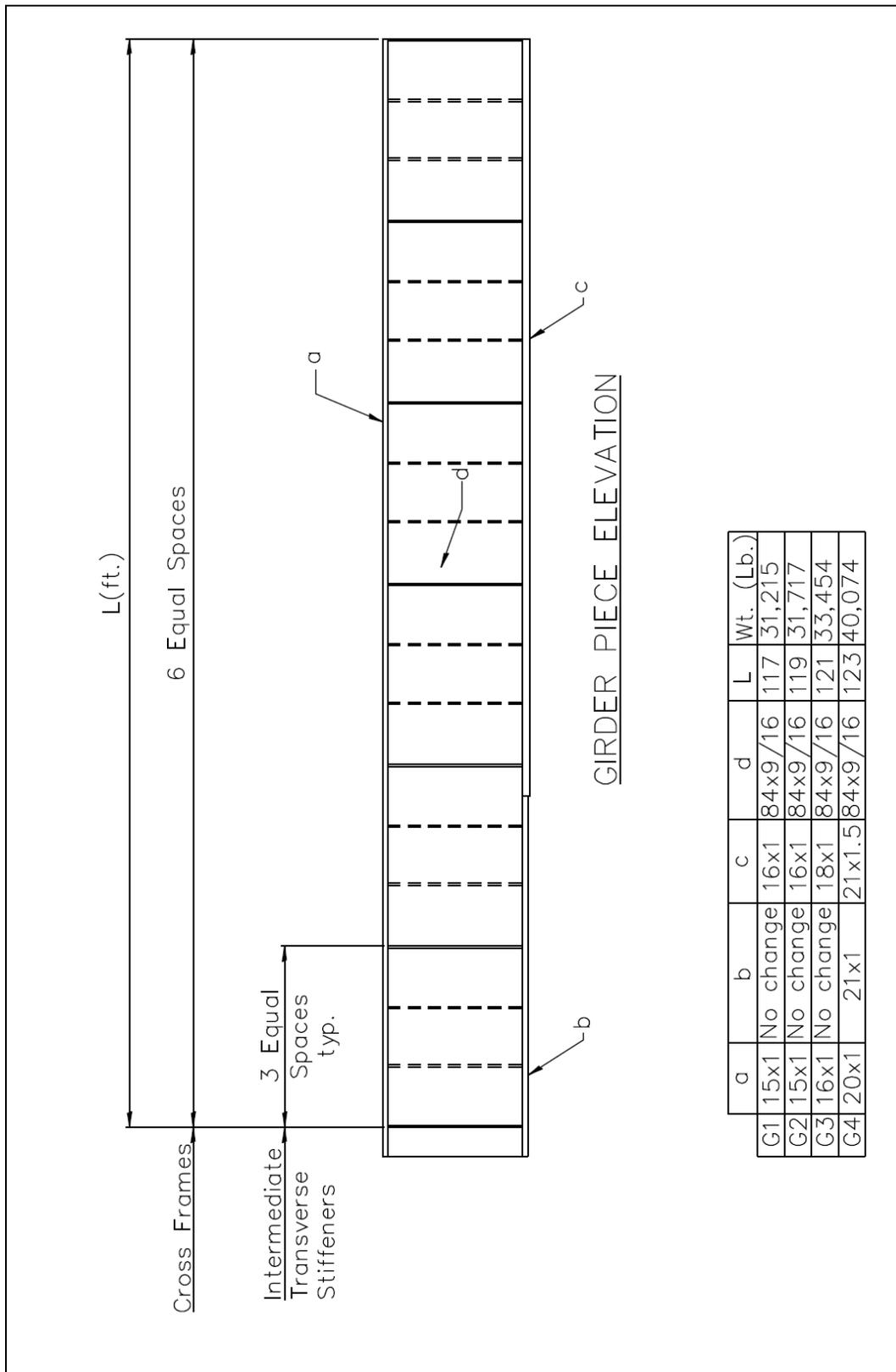


Figure B2-3: Example 2 Field Section 1

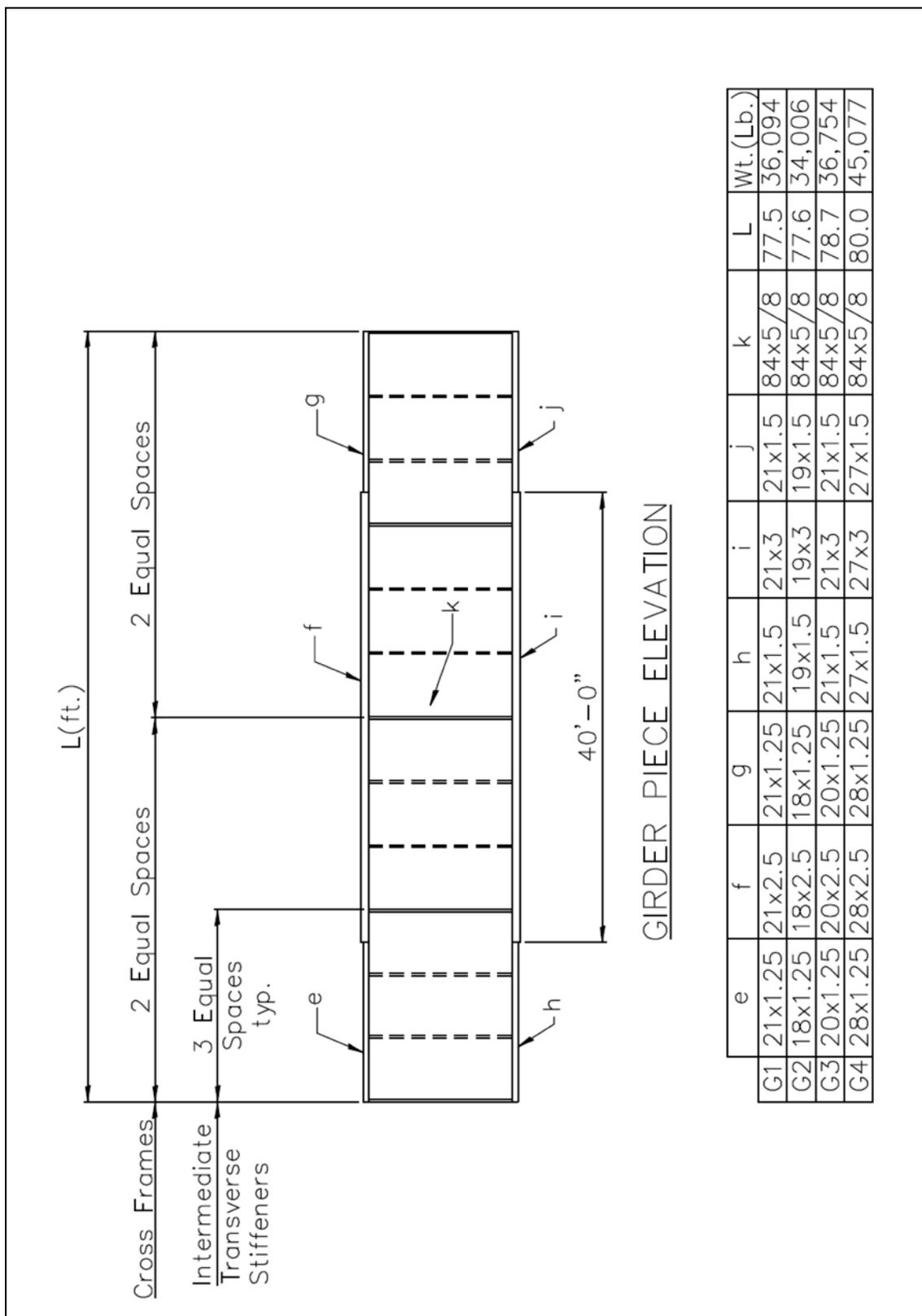


Figure B2-4: Example 2 Field Section 2

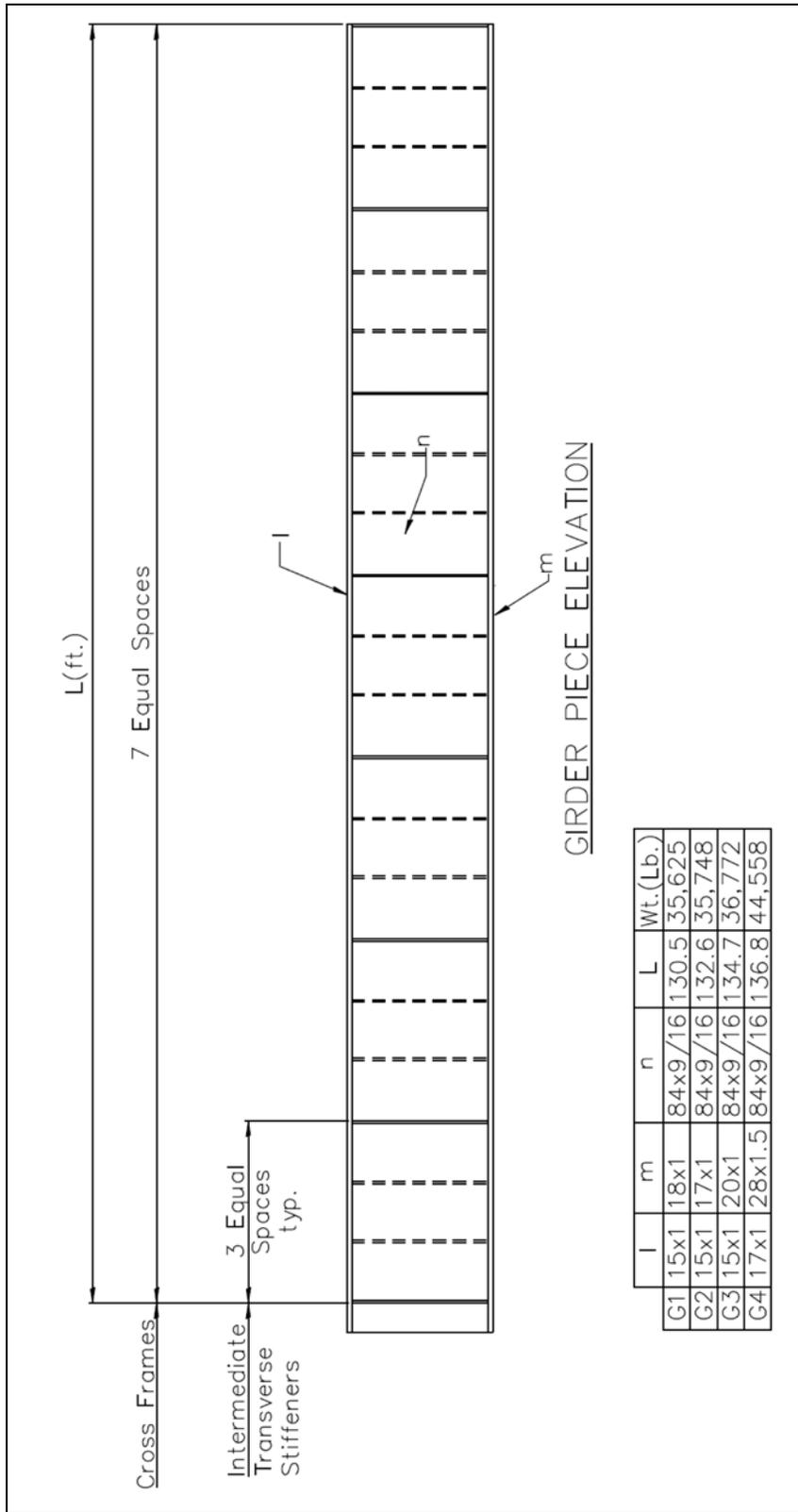


Figure B2-5: Example 2 Field Section 3

Wind Load:

Use Figure 26.5-1A in ASCE 7-10 to determine basic wind speed for Risk Category II.

$$V := 115 \text{ miles per hour}$$

Use Manual Appendix D Table D-3.2 to determine design wind speed reduction based on construction duration.

Wind Velocity Modification Factor

$$V_m := 0.75 \quad \text{Assume 6 weeks to 1 year as duration for steel erection.}$$

Design Wind Speed

$$DWS := V_m \cdot V \qquad DWS = 86.3 \text{ mph}$$

Incorporate modified Design Wind Speed into pressure equation

Velocity Pressure Exposure Coefficient (ASCE 7-10, Sect 26.7.3)

Assume Surface Roughness C - Open terrain with scattered obstructions having heights generally less than 30 feet. Therefore, Exposure Category C

Manual Table D-3-1, for Height = 30 feet, $K_z = 0.98$. Say $K_z = 1.0$.

Take wind directionality factor as 0.85 and topographic factor as 1.0.

$$K_z := 1 \qquad K_{zt} := 1 \qquad K_d := 0.85$$

Velocity Pressure (Manual Eq. D-3.4b)

$$q_z := 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot DWS^2 \qquad q_z = 16.2 \text{ psf}$$

Gust Effect Factor $G := 0.85$

Net Force Coefficient (Assuming deck forms not in place) $C_f := 2.2$ since ratio of girder spacing to depth, $S/d < 2$

Net Pressure

$$Q_z := G \cdot C_f \cdot q_z \qquad Q_z := 30.3 \text{ psf}$$

Wind Load, continued:

One Day / Girder Setting Design Wind Speed (Assuming minimum wind speed per D-3.4)

$$V := 20 \text{ mph}$$

Use 1.0 Wind Velocity Modification Factor (built in to minimum wind speed)

$$V_m := 1.0$$

Design Wind Speed:

$$DWS := V_m \cdot V \quad DWS = 20 \text{ mph}$$

One Day Girder Setting Velocity Pressure

(Manual Eq. D-3.4b)

$$q_{zset} := 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot DWS^2 \quad q_{zset} = 0.9 \text{ psf}$$

One Day Girder Setting Net Pressure

$$Q_{zset} := G \cdot C_f \cdot q_{zset} \quad Q_{zset} = 1.6 \text{ psf}$$

1.6 psf is negligible and can be ignored for short-duration events like girder picks which would not be occurring unless the wind is minimal anyway. However, use 5 psf as a minimum pressure for stability checks for pieces that are already set to account for accidental loading, etc. (per D-3.3).

$$Q_{zset} := 5 \text{ psf}$$

Wind Forces on Girders:

Exposed height for girder group = 7.5 feet (worst-case) + 33'-0" bridge width * 5% cross-slope

$$h := 7.5 + 33.0 \cdot 0.05 \quad h = 9.15 \text{ ft}$$

Exposed height for single girder being set = 7.5 feet (worst case) $h_{set} := 7.5 \text{ ft}$

Force to girder group during partially-erected or fully-erected condition (6+ weeks)

$$W_1 := Q_z \cdot h \quad W_1 = 277 \frac{\text{lb}}{\text{ft}}$$

Force to 1st girder during its setting (one day)

$$W_{set} := Q_{zset} \cdot h_{set} \quad W_{set} = 37.5 \frac{\text{lb}}{\text{ft}}$$

Single Girder Pick of G4, Field Section 3 (with inside cross-frames attached)

See the following pages for the UT Lift 1.3 output (Excel spreadsheet). The chosen lift point is 15'-10 1/4" from the end of the girder with a chord length of 105'-0" between lift points. The reaction at each point is approximately 33 kips, which must not exceed the allowable capacity of the lift clamps.

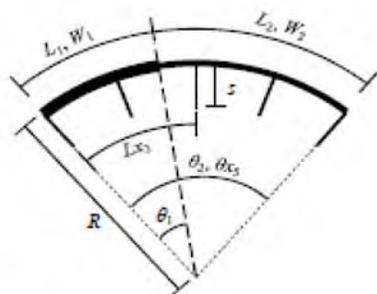
Also note that the calculated moment in the lifted curved girder is less than the buckling resistance (793.21 kip-ft vs. 2370.09 kip-ft).

The maximum twist at the end of the girder $\theta_{total} := 3.023$ deg

Since the twist is much more than 1.5 degrees, an alternate lift scheme should be considered. Try a lift of the girder G4 without the cross-frames attached. This will induce much less torsion of the lifted piece. The remaining Field Section 3 girders will also have to be analyzed (G3, G2, and G1) in order to determine a viable lift sequence that includes the placement of the cross-frames.

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Behavior of Curved Girder During Lifting



- L_1 : Length of Section 1
- L_2 : Length of Section 2
- W_1 : Weight per Unit Length of Section 1
- W_2 : Weight per Unit Length of Section 2
- $\theta_0 = 0$
- θ_1 : Internal Angle from the Beginning of the Girder to the End of Section 1
- θ_2 : Internal Angle from the Beginning of the Girder to the End of Section 2
- s : Cross Frame Width
- L_{x_1} : Length along the Girder to X-Frame 3
- L_{x_3} : Internal Angle from the Beginning of the Girder to X-Frame 3
- R : Radius of Curvature of the Girder

Girder Input:

Project : FHWA Appendix B Example Problems
 Girder # : Example 2 - Girder G4 Field Section 3

Clear Input User Input
 Notable Results

Number of Cross Sections:	
NUMSECTIONS =	1
Radius of Curvature (ft): R	
R =	716.5 ft

Material Constants:	
Girder Scale Factor:	E = 29000 ksi
S.F. _{girder} =	G = 11154 ksi
	$\rho = 490 \text{ lbs/ft}^3$

	Section 1						
TFLW	17						
TFLT	1						

DEPTH	84						
WEBT	0.5625						

BFLW	28						
BFLT	1.5						

	Section 1						
$L_1 =$	136.8						
$W_1 =$	397.7						
$\theta_1 =$	10.94						
$J_1 =$	42.15						
$C_{wt} =$	2589149						

- Length of Cross Sections (ft): L_1
- Weight per Unit Length (lb/ft): W_1
- Internal Angle from Beginning (deg.): θ_1
- Top Flange Width & Thickness (in): TFLW, TFLT
- Web Thickness & Depth (in): WEBT DEPTH
- Bottom Flange Width & Thickness (in): BFLW, BFLT
- Polar Moment of Inertia (in⁴): J_1
- Warping Constant (in⁶): C_{wt}

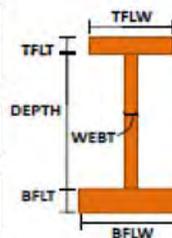


Figure B2-6: Page 1 of UT Lift Output for Girder G4 Field Section 3, with Cross-Frames

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Cross Frame Input:

Number of Cross Frame Locations:	
NUMXFRAMES =	8
Cross Frame Width: s	
$s =$	11.00 ft

Uniform Cross Frame Weights:	Constant X-Frame Weight
Weight =	

Uniformly Spaced Cross Frames:		Constant X-Frame Spacing	All Cross Frames on Inside of Curve, Outside of Curve, or Both		
Spacing =	19.542857 ft		All I	All O	All I/O
Location of the 1 st Cross Frame:					
1st X-Frame Loc. =	0 ft				

	X-Frame 1	X-Frame 2	X-Frame 3	X-Frame 4	X-Frame 5	X-Frame 6	X-Frame 7	X-Frame 8
$Lx_j =$	0	19.54286	39.085714	58.62857	78.17143	97.71429	117.2571	136.8
$Wx_j =$	1236.776	1236.776	1236.7759	1236.776	1236.776	1236.776	1236.776	1236.776
I/O	I	I	I	I	I	I	I	I
$\theta x_j =$	0.000	1.563	3.126	4.688	6.251	7.814	9.377	10.939

$Lx_j =$								
$Wx_j =$								
I/O								
$\theta x_j =$								

Cross Frame Location along centerline (CL) of the Girder (ft): Lx_j
 Weight of One Cross Frame (lbs): Wx_j
 Inside of Curve, Outside of Curve or Both (I, O, or I/O): I/O
 Internal Angle from Beginning (deg.): θx_j

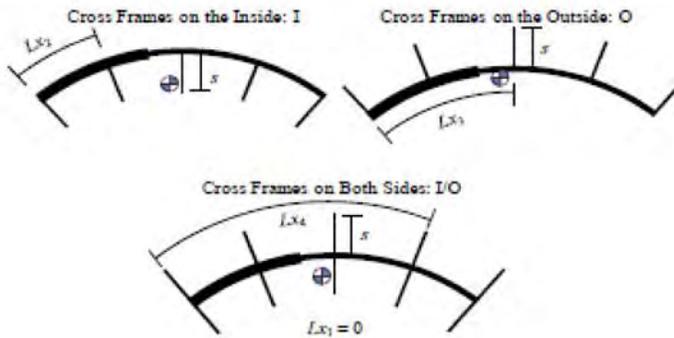
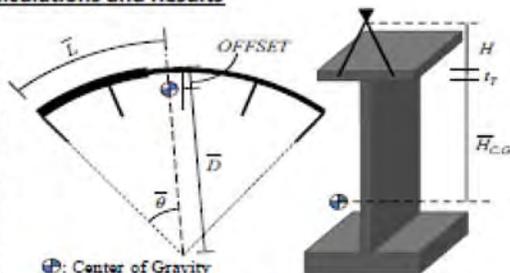


Figure B2-7: Page 2 of UT Lift Output for Girder G4 Field Section 3, with Cross-Frames

10/31/2013

Girder Segment Center of Gravity Calculations and Results

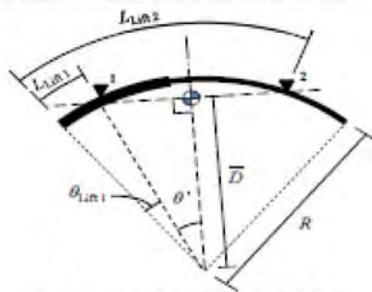
Total Length of Girder	
TOTAL_LENGTH =	136.80 ft
Total Girder Weight	
WEIGHT =	64300 lbs
	64.300 kips
Angular Distance to Center of Gravity	
$\bar{\theta}$ =	5.470 degrees
Distance along CL to Center of Gravity	
\bar{L} =	68.400 ft
Radial Distance to Center of Gravity	
\bar{D} =	714.520 ft
Offset of Center of Gravity from the CL	
OFFSET =	1.980 ft
Depth to Center of Gravity	
\bar{H}_{CG} =	50.545 in



- \oplus : Center of Gravity
- $\bar{\theta}$: Angular Distance to Center of Gravity
- \bar{L} : Length along Girder to Center of Gravity
- \bar{D} : Radial Distance to Center of Gravity
- H: Height of Axis of Rotation above the Top of Girder
- t_f : Top Flange Thickness
- \bar{H}_{CG} : Depth to Center of Gravity
- OFFSET: Radial Distance of Center of Gravity from \bar{C} of Girder

Zero Twist/Equal Force Lift Locations of a Nonprismatic Curved Girder

Angular Distance from Lift Pts. to Center of Gravity	
θ' =	4.261 degrees
Distance along CL to Lift Pt. 1	
L_{LN1} =	15.117 ft
Distance along CL to Lift Pt. 2	
L_{LN2} =	121.683 ft
Spreader Bar Needed	
ΔL =	106.467 ft
Lift Load (kips)	
LIFT_LOAD =	32.15 kips

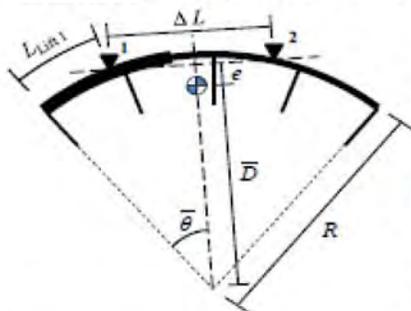


- L_{LN1} : Length along Girder to Lift Pt. 1
- L_{LN2} : Length along Girder to Lift Pt. 2
- θ_{LN1} : Angular Distance to Lift Pt. 1
- θ_{LN2} : Angular Distance to Lift Pt. 2
- \oplus : Center of Gravity
- θ' : Angular Distance from Lift Pts. to Center of Gravity
- \bar{D} : Radial Distance to Center of Gravity
- R: Radius of Curvature of the Girder

Figure B2-8: Page 3 of UT Lift Output for Girder G4 Field Section 3, with Cross-Frames

10/31/2013

Predicted Rigid Body Twist and Lift Reactions of a Nonprismatic Curved Girder



- $L_{Lift 1}$: Length along Girder to Lift Pt. 1
- ΔL : Chord Length to Lift Pt. 2 (Spreader Bar)
- e : Eccentricity Between the Line of Support and the Center of Mass
- \oplus : Center of Gravity
- $\bar{\theta}$: Angular Distance to Center of Gravity
- \bar{D} : Radial Distance to Center of Gravity
- R : Radius of Curvature of the Girder

Distance along CL to Lift Pt. 1	$L_{Lift 1} = 15.854$ ft
---------------------------------	--------------------------

Chord Length Between Lift Pts.	$\Delta L = 105.000$ ft
--------------------------------	-------------------------

Axis of Rotation above Top of Girder	$H = 30.000$ in
--------------------------------------	-----------------

Eccentricity	$e = 0.054$ ft 0.651 in
--------------	------------------------------

Twist	$\theta_{Rigid} = 0.008$ rad 0.458 degrees
-------	---

Reactions	Lift Point 1 = 32.15 kips Lift Point 2 = 32.15 kips
-----------	--

Calculate Twist & Stress

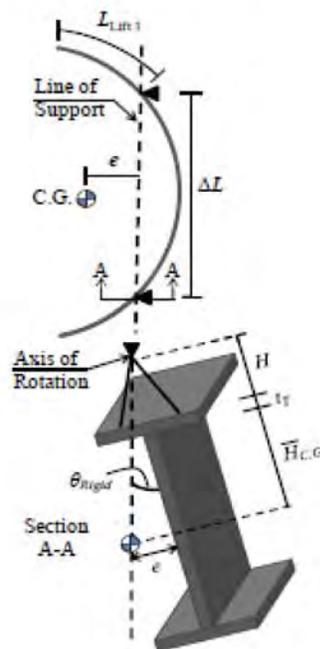


Figure B2-9: Page 4 of UT Lift Output for Girder G4 Field Section 3, with Cross-Frames

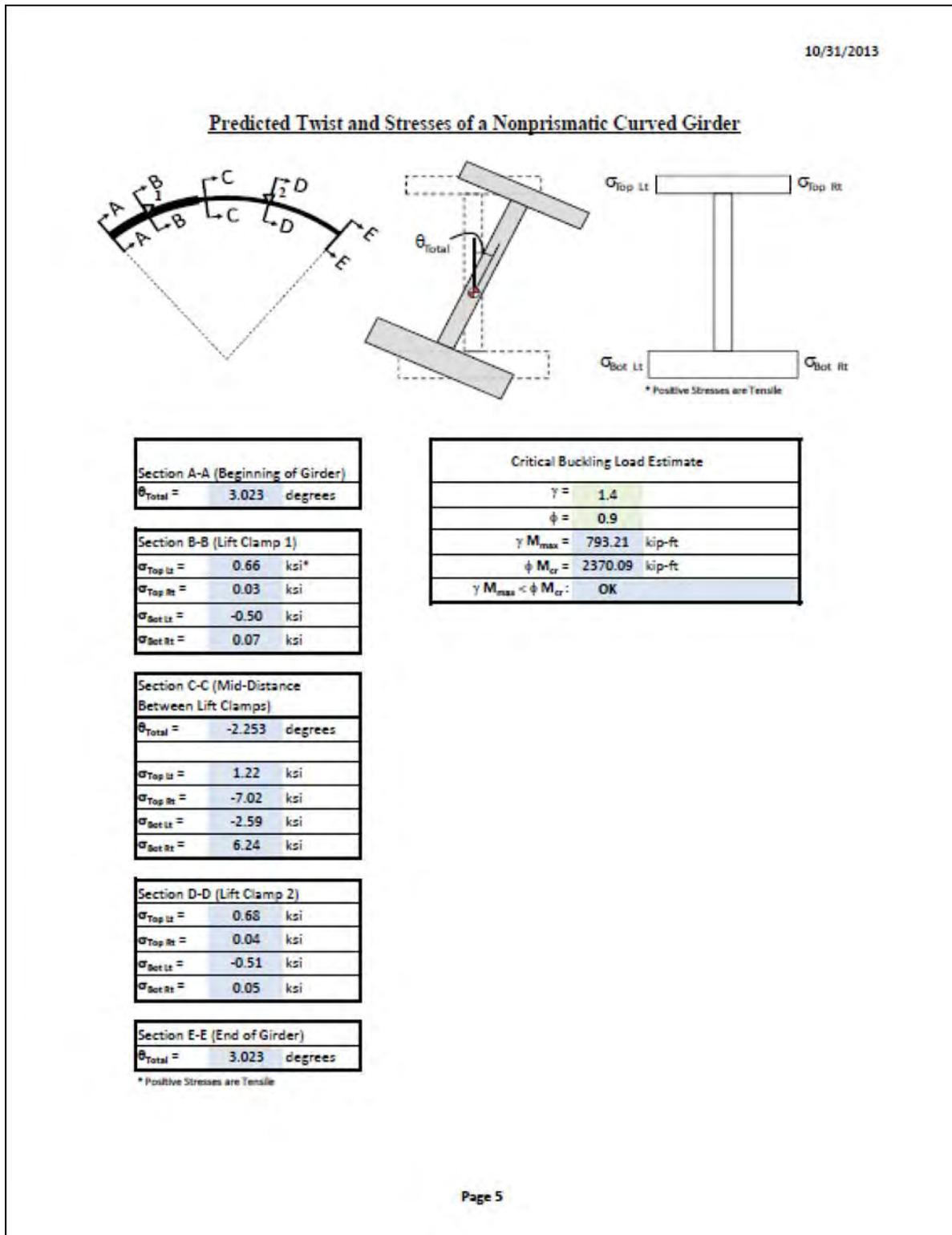


Figure B2-10: Page 5 of UT Lift Output for Girder G4 Field Section 3, with Cross-Frames

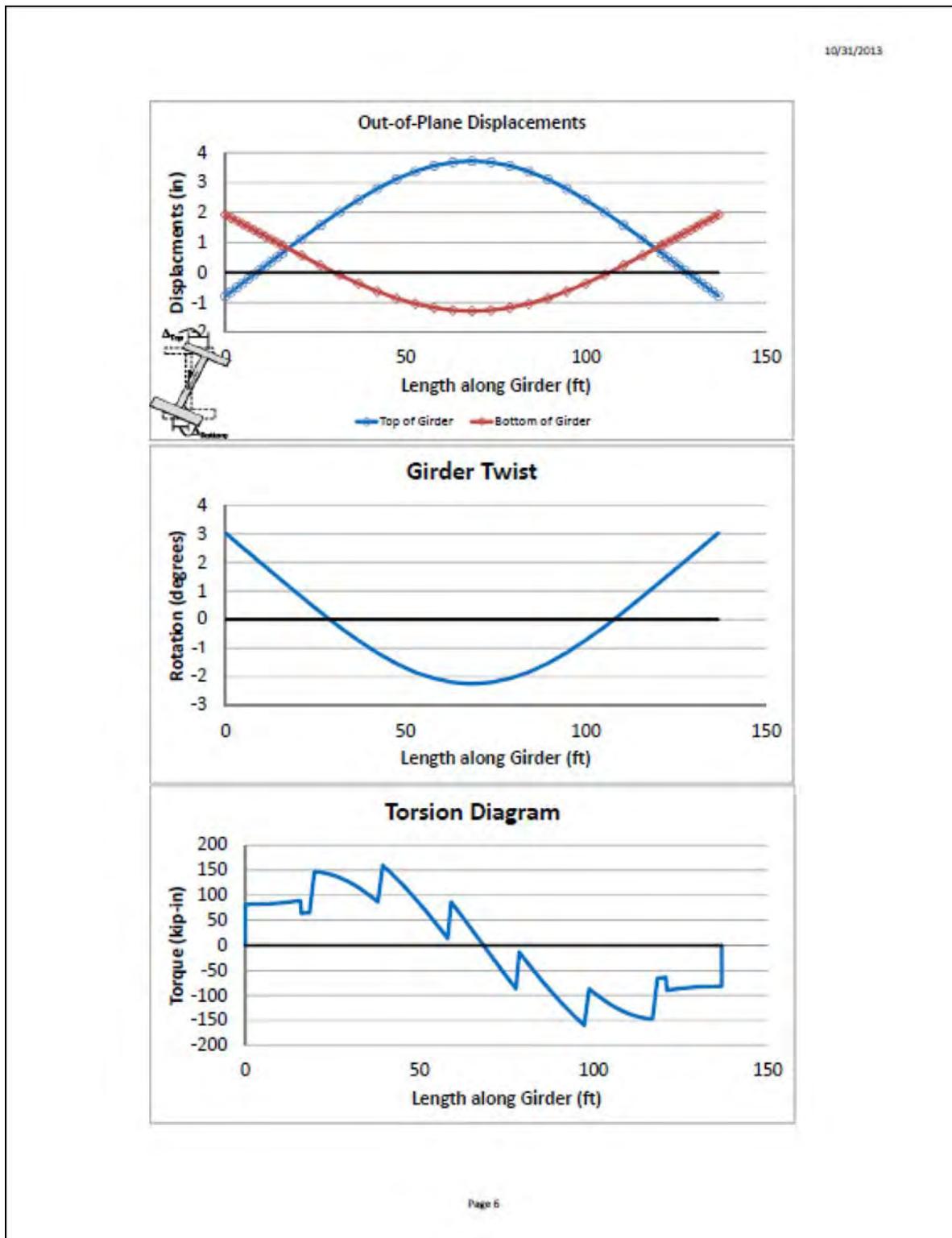


Figure B2-11: Page 6 of UT Lift Output for Girder G4 Field Section 3, with Cross-Frames

Single Girder Pick of G4, Field Section 3 (without inside cross-frames attached)

See the following pages for the UT Lift 1.3 output (Excel spreadsheet). The chosen lift point is 28'-4 1/2" from the end of the girder with a chord length of 80'-0" between lift points. The reaction at each point is approximately 28 kips, which must not exceed the allowable capacity of the lift clamps.

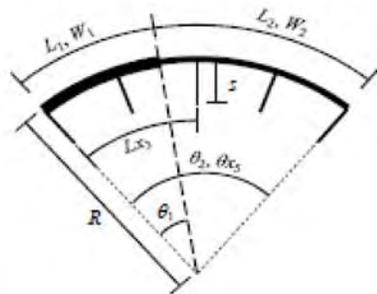
Also note that the calculated moment in the lifted curved girder is less than the buckling resistance (224.27 kip-ft vs. 2370.09 kip-ft).

The maximum twist at the end of the girder $\theta_{total} := 0.274\text{deg}$

Since the twist is much less than 1.5 degrees, this should be adequate for fit-up. Since the cross-frames will not be attached to G4 during its lift, they will have to be lifted with G3 instead. To avoid excessive torsion on the G3 piece, both the inside and outside cross-frames will be attached to that girder prior to lifting.

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Behavior of Curved Girder During Lifting



- L_1 : Length of Section 1
- L_2 : Length of Section 2
- W_1 : Weight per Unit Length of Section 1
- W_2 : Weight per Unit Length of Section 2
- $\theta_0 = 0$
- θ_1 : Internal Angle from the Beginning of the Girder to the End of Section 1
- θ_2 : Internal Angle from the Beginning of the Girder to the End of Section 2
- s : Cross Frame Width
- Lx_3 : Length along the Girder to X-Frame 3
- θx_3 : Internal Angle from the Beginning of the Girder to X-Frame 3
- R : Radius of Curvature of the Girder

Girder Input:

Project : FHWA Appendix B Example Problems
 Girder # : Example 2 - Girder G4 Field Section 3

Clear Input User Input
 Notable Results

Number of Cross Sections:
 NUMSECTIONS = 1
 Radius of Curvature (ft): R
 R = 716.5 ft

Girder Scale Factor: E = 29000 ksi
 S.F._{girder} = 1.10 G = 11154 ksi
 Material Constants: p = 490 lbs/ft³

	Section 1						
TFLW	17						
TFLT	1						

DEPTH	84						
WEBT	0.5625						

BFLW	28						
BFLT	1.5						

	Section 1						
$L_1 =$	136.8						
$W_1 =$	397.7						
$\theta_1 =$	10.94						
$J_1 =$	42.15						
$C_{w1} =$	2589149						

- Length of Cross Sections (ft): L_1
- Weight per Unit Length (lb/ft): W_1
- Internal Angle from Beginning (deg.): θ_1
- Top Flange Width & Thickness (in): TFLW, TFLT
- Web Thickness & Depth (in): WEBT DEPTH
- Bottom Flange Width & Thickness (in): BFLW, BFLT
- Polar Moment of Inertia (in⁴): J_1
- Warping Constant (in⁶): C_{w1}

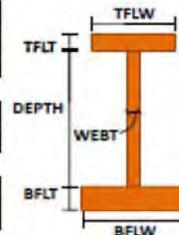


Figure B2-12: Page 1 of UT Lift Output for Girder G4 Field Section 3, without Cross-Frames

10/31/2013

Cross Frame Input:

Number of Cross Frame Locations:	
NUMXFRAMES =	8
Cross Frame Width: s	
$s =$	11.00 ft

Uniform Cross Frame Weights:	Constant X-Frame Weight
Weight =	

Uniformly Spaced Cross Frames:		Constant X-Frame Spacing	All Cross Frames on Inside of Curve, Outside of Curve, or Both		
Spacing =	19.542857 ft		All I	All O	All I/O
Location of the 1 st Cross Frame:					
1st X-Frame Loc. =	0 ft				

	X-Frame 1	X-Frame 2	X-Frame 3	X-Frame 4	X-Frame 5	X-Frame 6	X-Frame 7	X-Frame 8
$Lx_j =$	0	19.54286	39.085714	58.62857	78.17143	97.71429	117.2571	136.8
$Wx_j =$	0	0	0	0	0	0	0	0
I/O	I	I	I	I	I	I	I	I
$\theta x_j =$	0.000	1.563	3.126	4.688	6.251	7.814	9.377	10.939

$Lx_j =$								
$Wx_j =$								
I/O								
$\theta x_j =$								

Cross Frame Location along centerline (CL) of the Girder (ft): Lx_j
 Weight of One Cross Frame (lbs): Wx_j
 Inside of Curve, Outside of Curve or Both (I, O, or I/O): I/O
 Internal Angle from Beginning (deg.): θx_j

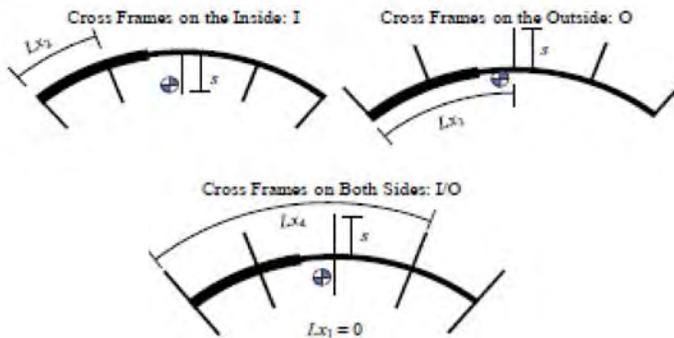
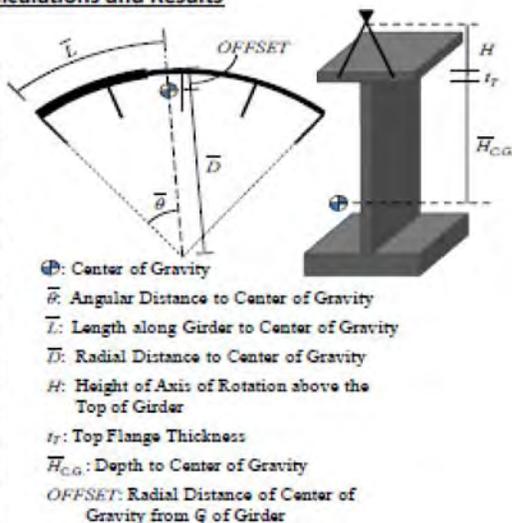


Figure B2-13: Page 2 of UT Lift Output for Girder G4 Field Section 3, without Cross-Frames

10/31/2013

Girder Segment Center of Gravity Calculations and Results

Total Length of Girder	
TOTAL LENGTH =	136.80 ft
Total Girder Weight	
WEIGHT =	54405 lbs
	54.405 kips
Angular Distance to Center of Gravity	
$\bar{\theta}$ =	5.470 degrees
Distance along CL to Center of Gravity	
\bar{L} =	68.400 ft
Radial Distance to Center of Gravity	
\bar{D} =	715.412 ft
Offset of Center of Gravity from the CL	
OFFSET =	1.088 ft
Depth to Center of Gravity	
\bar{H}_{CG} =	52.099 in



Zero Twist/Equal Force Lift Locations of a Nonprismatic Curved Girder

Angular Distance from Lift Pts. to Center of Gravity	
θ' =	3.158 degrees
Distance along CL to Lift Pt. 1	
L_{LH1} =	28.913 ft
Distance along CL to Lift Pt. 2	
L_{LH2} =	107.887 ft
Spreader Bar Needed	
ΔL =	78.934 ft
Lift Load (kips)	
LIFT_LOAD =	27.20 kips

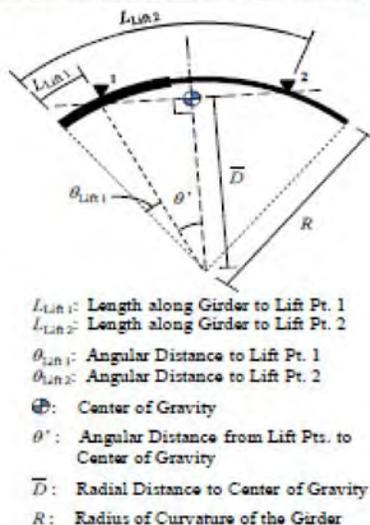
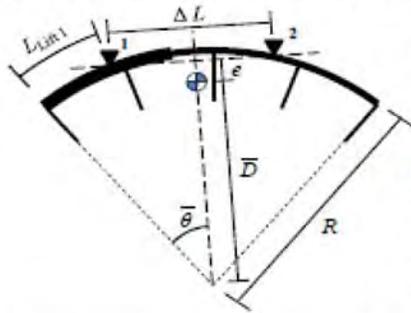


Figure B2-14: Page 3 of UT Lift Output for Girder G4 Field Section 3, without Cross-Frames

10/31/2013

Predicted Rigid Body Twist and Lift Reactions of a Nonprismatic Curved Girder



- L_{Lift1} : Length along Girder to Lift Pt. 1
- ΔL : Chord Length to Lift Pt. 2 (Spreader Bar)
- e : Eccentricity Between the Line of Support and the Center of Mass
- \oplus : Center of Gravity
- $\bar{\theta}$: Angular Distance to Center of Gravity
- \bar{D} : Radial Distance to Center of Gravity
- R : Radius of Curvature of the Girder

Distance along CL to Lift Pt. 1	$L_{Lift1} = 28.375$ ft
---------------------------------	-------------------------

Chord Length Between Lift Pts.	$\Delta L = 80.000$ ft
--------------------------------	------------------------

Axis of Rotation above Top of Girder	$H = 30.000$ in
--------------------------------------	-----------------

Eccentricity	$e = -0.030$ ft -0.355 in
--------------	--------------------------------

Twist	$\theta_{Rigid} = -0.004$ rad -0.245 degrees
-------	---

Reactions	Lift Point 1 = 27.20 kips Lift Point 2 = 27.21 kips
-----------	--

Calculate Twist & Stress

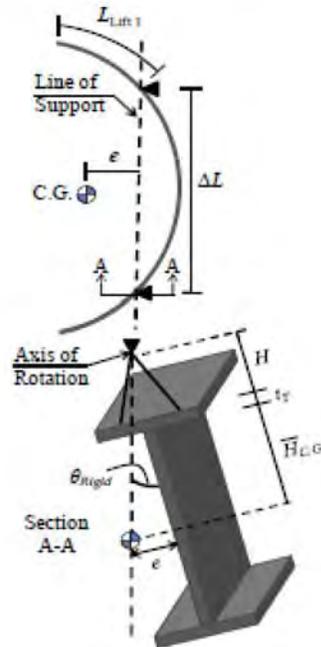


Figure B2-15: Page 4 of UT Lift Output for Girder G4 Field Section 3, without Cross-Frames

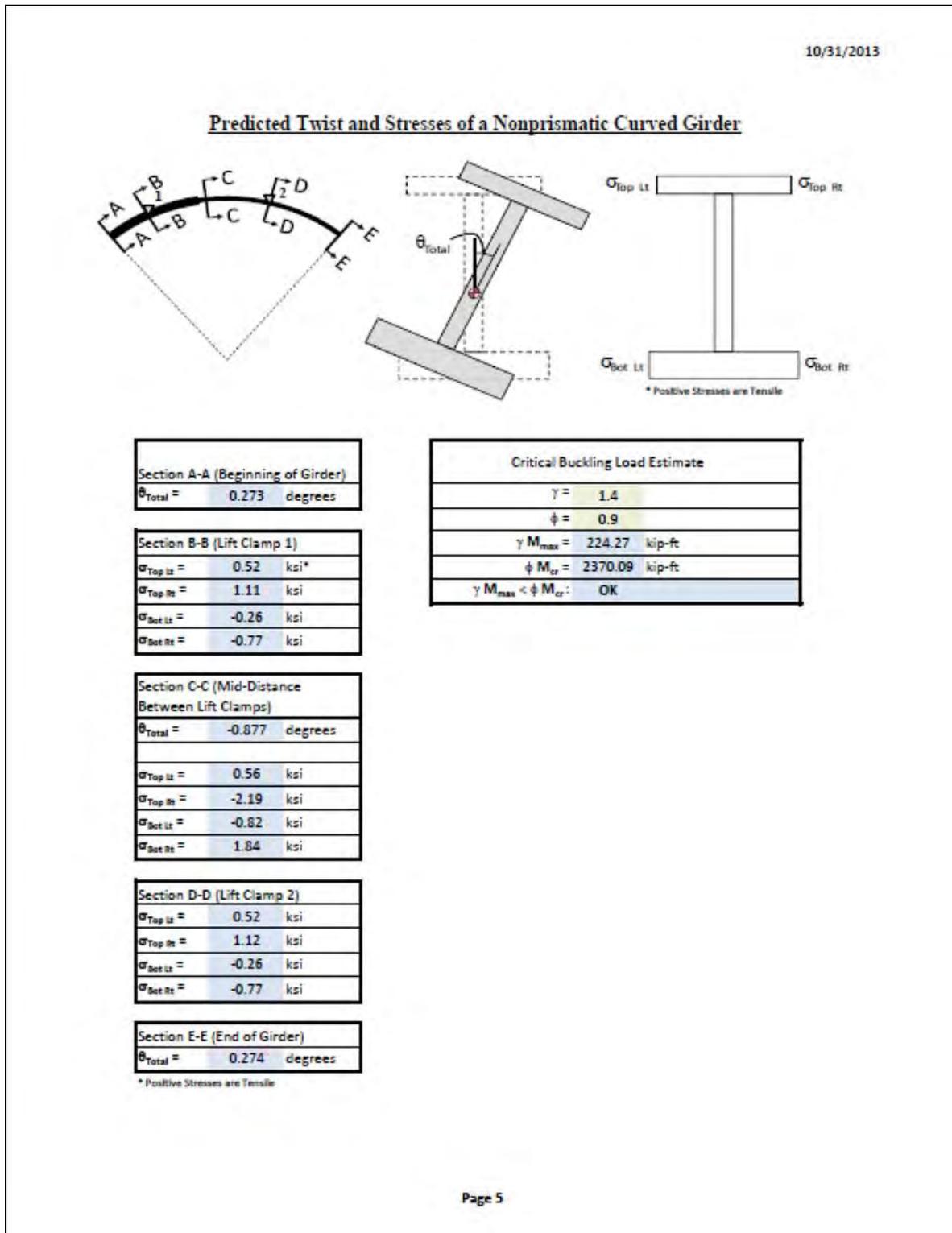


Figure B2-16: Page 5 of UT Lift Output for Girder G4 Field Section 3, without Cross-Frames

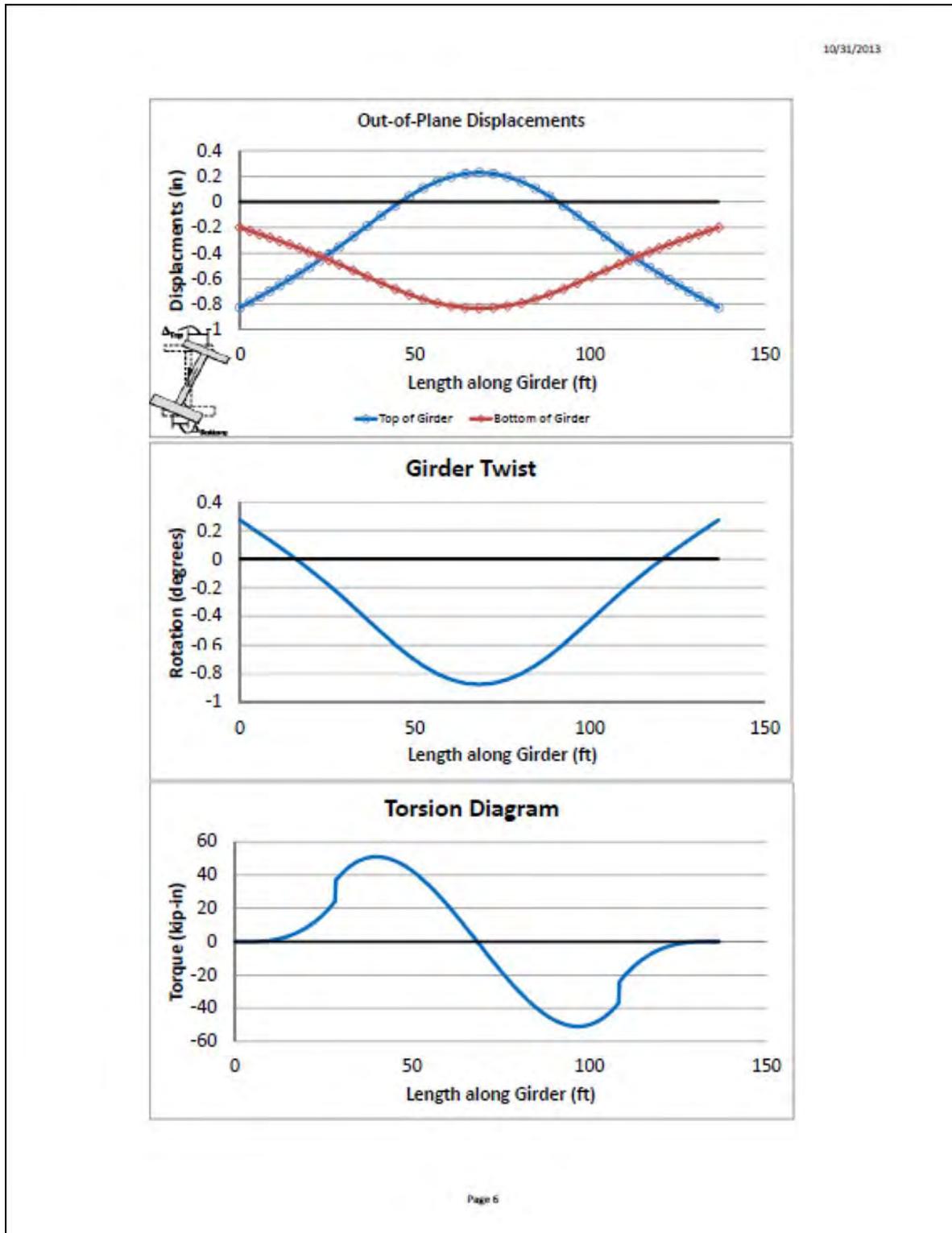


Figure B2-17: Page 6 of UT Lift Output for Girder G4 Field Section 3, without Cross-Frames

Single Girder Pick of G3, Field Section 3 (with both inside and outside cross-frames attached)

See the following pages for the UT Lift 1.3 output (Excel spreadsheet). The chosen lift point is 27'-4" from the end of the girder with a chord length of 80'-0" between lift points. The reaction at each point is approximately 31 kips, which must not exceed the allowable capacity of the lift clamps.

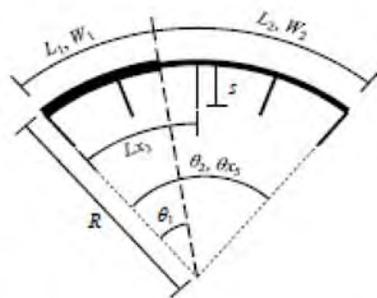
Also note that the calculated moment in the lifted curved girder is less than the buckling resistance (283.54 kip-ft vs. 894.84 kip-ft).

The maximum twist at the end of the girder $\theta_{\text{total}} := 1.454 \text{ deg}$

Since the twist is less than 1.5 degrees, this should be adequate for fit-up.

10/31/2013

Behavior of Curved Girder During Lifting



- L_1 : Length of Section 1
- L_2 : Length of Section 2
- W_1 : Weight per Unit Length of Section 1
- W_2 : Weight per Unit Length of Section 2
- $\theta_0 = 0$
- θ_1 : Internal Angle from the Beginning of the Girder to the End of Section 1
- θ_2 : Internal Angle from the Beginning of the Girder to the End of Section 2
- s : Cross Frame Width
- Lx_3 : Length along the Girder to X-Frame 3
- θx_3 : Internal Angle from the Beginning of the Girder to X-Frame 3
- R : Radius of Curvature of the Girder

Girder Input:

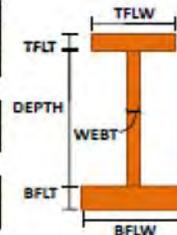
Project : FHWA Appendix B Example Problems
 Girder # : Example 2 - Girder G3 Field Section 3

Clear Input User Input
 Notable Results

Number of Cross Sections:	
NUMSECTIONS =	1
Radius of Curvature (ft): R	
R =	705.5 ft

Material Constants:	
Girder Scale Factor:	E = 29000 ksi
S.F.girder =	G = 11154 ksi
	$\rho = 490 \text{ lbs/ft}^3$

Section 1							
TFLW	15						
TFLT	1						
DEPTH	84						
WEBT	0.5625						
BFLW	20						
BFLT	1						



Section 1							
$L_1 =$	134.7						
$W_1 =$	307.9						
$\theta_1 =$	10.94						
$J_1 =$	16.65						
$C_{wt} =$	1429121						

- Length of Cross Sections (ft): L_1
- Weight per Unit Length (lb/ft): W_1
- Internal Angle from Beginning (deg.): θ_1
- Top Flange Width & Thickness (in): TFLW, TFLT
- Web Thickness & Depth (in): WEBT DEPTH
- Bottom Flange Width & Thickness (in): BFLW, BFLT
- Polar Moment of Inertia (in⁴): J_1
- Warping Constant (in⁶): C_{wt}

Figure B2-18: Page 1 of UT Lift Output for Girder G3 Field Section 3

10/31/2013

Cross Frame Input:

Number of Cross Frame Locations:	
NUMXFRAMES =	8
Cross Frame Width: s	
$s =$	11.00 ft

Uniform Cross Frame Weights:	Constant X-Frame Weight
Weight =	1236.776 lbs

Uniformly Spaced Cross Frames:		Constant X-Frame Spacing	All Cross Frames on Inside of Curve, Outside of Curve, or Both		
Spacing =	19.242857 ft		All I	All O	All I/O
Location of the 1 st Cross Frame:					
1st X-Frame Loc. =	0 ft				

	X-Frame 1	X-Frame 2	X-Frame 3	X-Frame 4	X-Frame 5	X-Frame 6	X-Frame 7	X-Frame 8
$Lx_j =$	0	19.24286	38.485714	57.72857	76.97143	96.21429	115.4571	134.7
$Wx_j =$	1236.776	1236.776	1236.7759	1236.776	1236.776	1236.776	1236.776	1236.776
I/O	I/O	I/O	I/O	I/O	I/O	I/O	I/O	I/O
$\theta x_j =$	0.000	1.563	3.126	4.688	6.251	7.814	9.377	10.939

$Lx_j =$								
$Wx_j =$								
I/O								
$\theta x_j =$								

Cross Frame Location along centerline (CL) of the Girder (ft): Lx_j
 Weight of One Cross Frame (lbs): Wx_j
 Inside of Curve, Outside of Curve or Both (I, O, or I/O): I/O
 Internal Angle from Beginning (deg.): θx_j

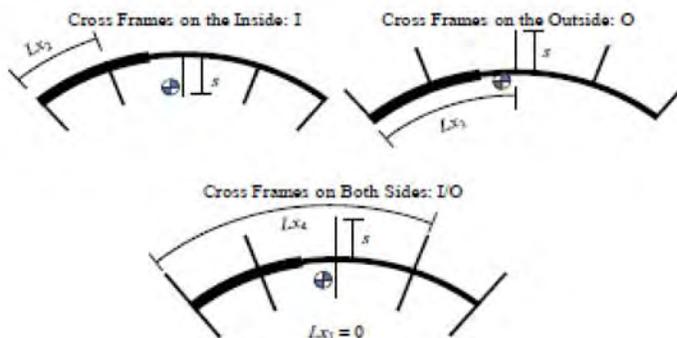
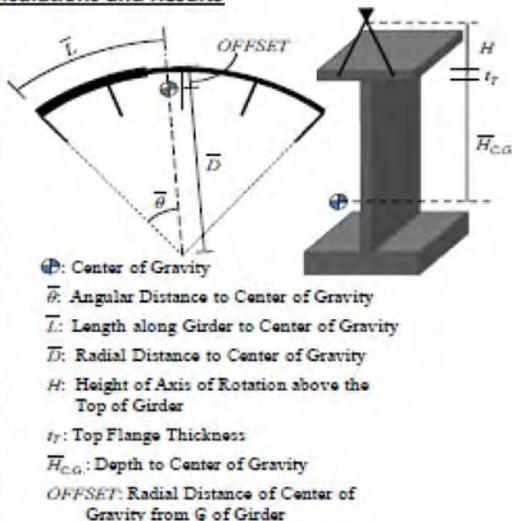


Figure B2-19: Page 2 of UT Lift Output for Girder G3 Field Section 3

10/31/2013

Girder Segment Center of Gravity Calculations and Results

Total Length of Girder	
TOTAL LENGTH =	134.70 ft
Total Girder Weight	
WEIGHT =	61258 lbs
	61.258 kips
Angular Distance to Center of Gravity	
$\bar{\theta}$ =	5.470 degrees
Distance along CL to Center of Gravity	
\bar{L} =	67.350 ft
Radial Distance to Center of Gravity	
\bar{D} =	704.330 ft
Offset of Center of Gravity from the CL	
OFFSET =	1.170 ft
Depth to Center of Gravity	
\bar{H}_{CG} =	43.749 in



Zero Twist/Equal Force Lift Locations of a Nonprismatic Curved Girder

Angular Distance from Lift Pts. to Center of Gravity	
θ' =	3.300 degrees
Distance along CL to Lift Pt. 1	
$L_{Lift 1}$ =	26.715 ft
Distance along CL to Lift Pt. 2	
$L_{Lift 2}$ =	107.985 ft
Spreader Bar Needed	
ΔL =	81.225 ft
Lift Load (kips)	
LIFT_LOAD =	30.63 kips

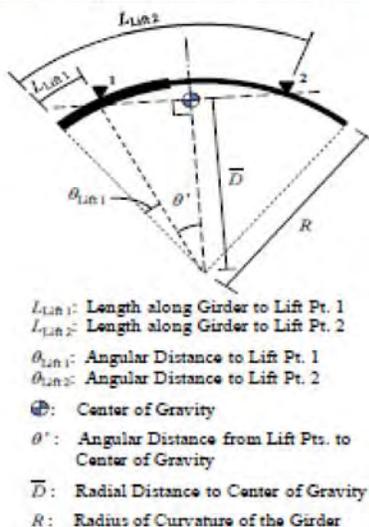
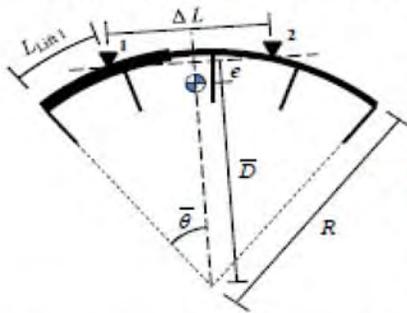


Figure B2-20: Page 3 of UT Lift Output for Girder G3 Field Section 3

10/31/2013

Predicted Rigid Body Twist and Lift Reactions of a Nonprismatic Curved Girder



- L_{lift1} : Length along Girder to Lift Pt. 1
- ΔL : Chord Length to Lift Pt. 2 (Spreader Bar)
- e : Eccentricity Between the Line of Support and the Center of Mass
- \oplus : Center of Gravity
- $\bar{\theta}$: Angular Distance to Center of Gravity
- \bar{D} : Radial Distance to Center of Gravity
- R : Radius of Curvature of the Girder

Distance along CL to Lift Pt. 1
$L_{lift1} = 27.333$ ft

Chord Length Between Lift Pts.
$\Delta L = 80.000$ ft

Axis of Rotation above Top of Girder
$H = 30.000$ in

Eccentricity
$e = 0.035$ ft
0.421 in

Twist
$\theta_{Rigid} = 0.006$ rad
0.322 degrees

Reactions
Lift Point 1 = 30.63 kips
Lift Point 2 = 30.63 kips

Calculate Twist & Stress

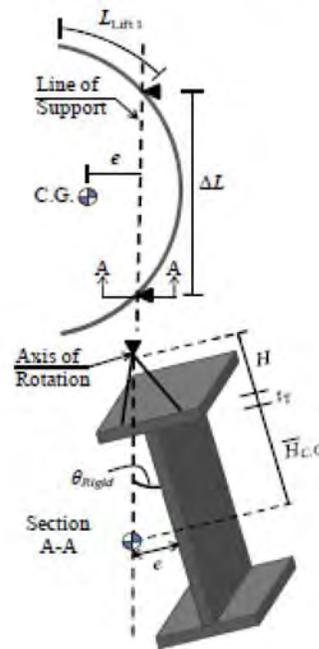
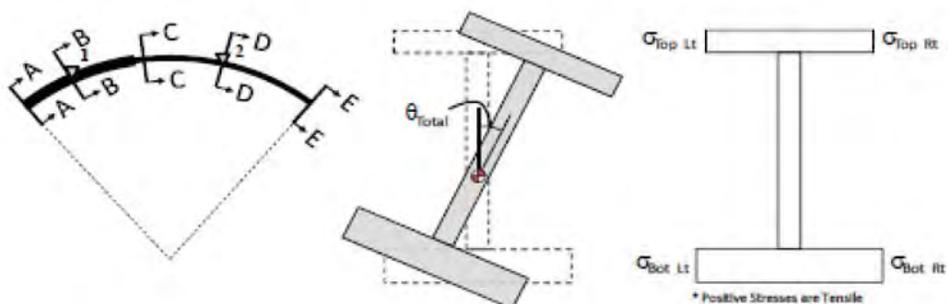


Figure B2-21: Page 4 of UT Lift Output for Girder G3 Field Section 3

10/31/2013

Predicted Twist and Stresses of a Nonprismatic Curved Girder



Section A-A (Beginning of Girder)	
θ_{Total}	= 1.454 degrees

Section B-B (Lift Clamp 1)	
$\sigma_{Top Lt}$	= 1.26 ksi*
$\sigma_{Top Rt}$	= 1.18 ksi
$\sigma_{Bot Lt}$	= -0.91 ksi
$\sigma_{Bot Rt}$	= -1.26 ksi

Section C-C (Mid-Distance Between Lift Clamps)	
θ_{Total}	= -0.908 degrees
$\sigma_{Top Lt}$	= 1.28 ksi
$\sigma_{Top Rt}$	= -3.06 ksi
$\sigma_{Bot Lt}$	= -1.54 ksi
$\sigma_{Bot Rt}$	= 3.12 ksi

Section D-D (Lift Clamp 2)	
$\sigma_{Top Lt}$	= 1.27 ksi
$\sigma_{Top Rt}$	= 1.18 ksi
$\sigma_{Bot Lt}$	= -0.91 ksi
$\sigma_{Bot Rt}$	= -1.26 ksi

Section E-E (End of Girder)	
θ_{Total}	= 1.453 degrees

* Positive Stresses are Tensile

Critical Buckling Load Estimate	
γ	= 1.4
ϕ	= 0.9
γM_{max}	= 283.54 kip-ft
ϕM_{cr}	= 894.84 kip-ft
$\gamma M_{max} < \phi M_{cr}$: OK

Figure B2-22: Page 5 of UT Lift Output for Girder G3 Field Section 3

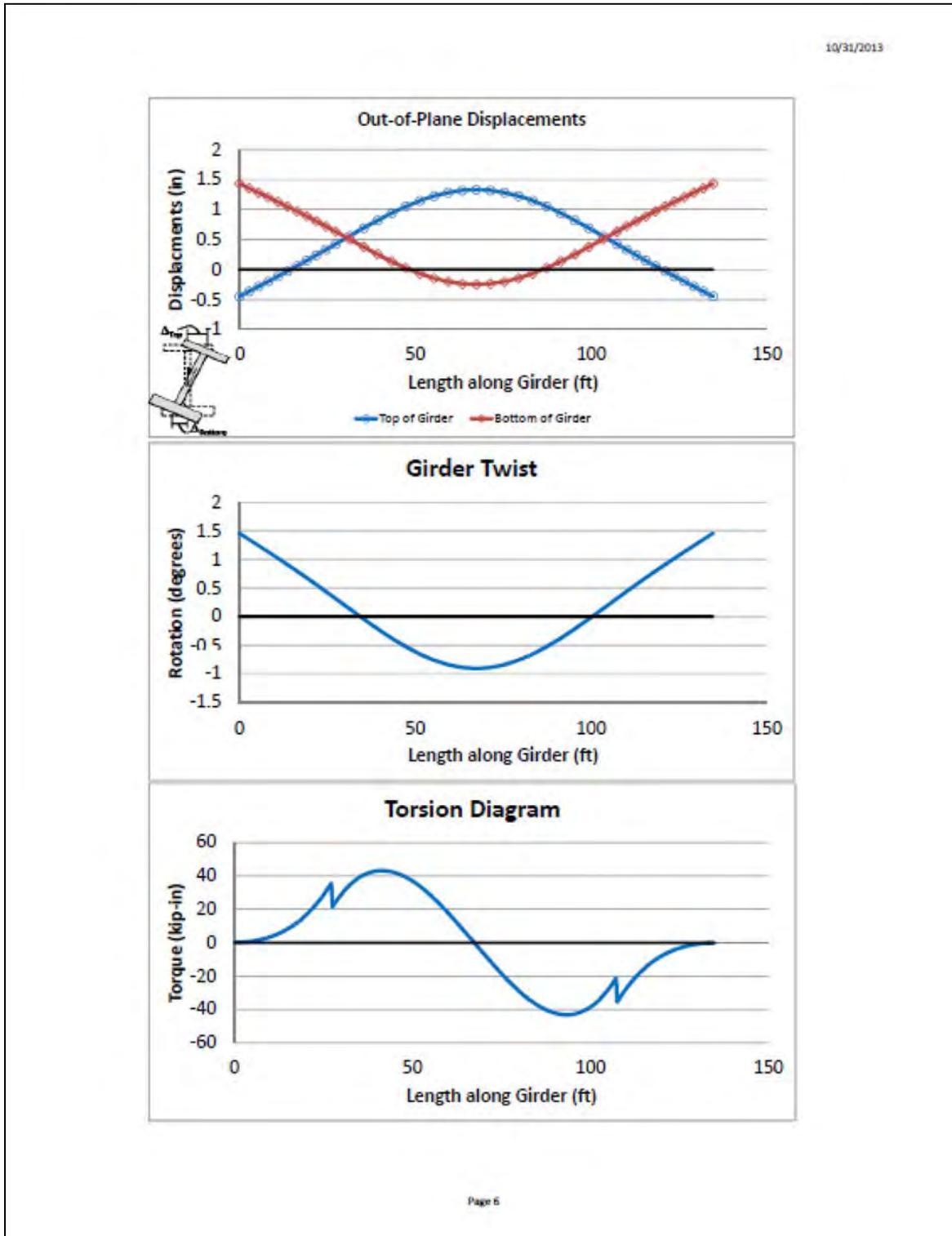


Figure B2-23: Page 6 of UT Lift Output for Girder G3 Field Section 3

Single Girder Pick of G2, Field Section 3 (without either inside or outside cross-frames attached)

See the following pages for the UT Lift 1.3 output (Excel spreadsheet). The chosen lift point is 28'-9 1/2" from the end of the girder with a chord length of 75'-0" between lift points. The reaction at each point is approximately 20 kips, which must not exceed the allowable capacity of the lift clamps.

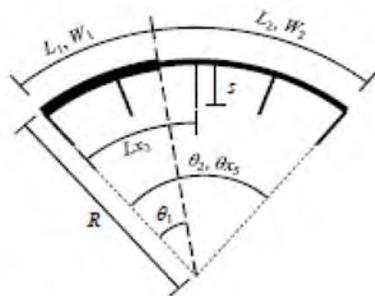
Also note that the calculated moment in the lifted curved girder is less than the buckling resistance (171.81 kip-ft vs. 739.49 kip-ft).

The maximum twist at the end of the girder $\theta_{total} := 1.045\text{deg}$

Since the twist is less than 1.5 degrees, this should be adequate for fit-up.

10/31/2013

Behavior of Curved Girder During Lifting



- L_1 : Length of Section 1
- L_2 : Length of Section 2
- W_1 : Weight per Unit Length of Section 1
- W_2 : Weight per Unit Length of Section 2
- $\theta_0 = 0$
- θ_1 : Internal Angle from the Beginning of the Girder to the End of Section 1
- θ_2 : Internal Angle from the Beginning of the Girder to the End of Section 2
- s : Cross Frame Width
- Lx_3 : Length along the Girder to X-Frame 3
- θx_3 : Internal Angle from the Beginning of the Girder to X-Frame 3
- R : Radius of Curvature of the Girder

Girder Input:

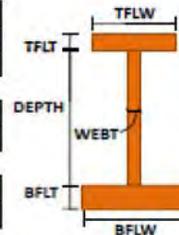
Project : FHWA Appendix B Example Problems
 Girder # : Example 2 - Girder G2 Field Section 3

Clear Input User Input
 Notable Results

Number of Cross Sections:	
NUMSECTIONS =	1
Radius of Curvature (ft): R	
R =	694.5 ft

Material Constants:	
Girder Scale Factor:	E = 29000 ksi
S.F. _{girder} =	G = 11154 ksi
	$\rho = 490 \text{ lbs/ft}^3$

TFLW	Section 1						
TFLT	15						
DEPTH	84						
WEBT	0.5625						
BFLW	17						
BFLT	1						
$L_1 =$	132.6						
$W_1 =$	296.6						
$\theta_1 =$	10.94						
$J_1 =$	15.65						
$C_{wt} =$	1204557						



- Length of Cross Sections (ft): L_1
- Weight per Unit Length (lb/ft): W_1
- Internal Angle from Beginning (deg.): θ_1
- Top Flange Width & Thickness (in): TFLW, TFLT
- Web Thickness & Depth (in): WEBT DEPTH
- Bottom Flange Width & Thickness (in): BFLW, BFLT
- Polar Moment of Inertia (in⁴): J_1
- Warping Constant (in⁶): C_{wt}

Figure B2-24: Page 1 of UT Lift Output for Girder G2 Field Section 3

10/31/2013

Cross Frame Input:

Number of Cross Frame Locations:	
NUMXFRAMES =	8
Cross Frame Width: s	
$s =$	11.00 ft

Uniform Cross Frame Weights:	Constant X-Frame Weight
Weight =	

Uniformly Spaced Cross Frames:		Constant X-Frame Spacing	All Cross Frames on Inside of Curve, Outside of Curve, or Both		
Spacing =	18.942857 ft		All I	All O	All I/O
Location of the 1 st Cross Frame:					
1st X-Frame Loc. =	0 ft				

	X-Frame 1	X-Frame 2	X-Frame 3	X-Frame 4	X-Frame 5	X-Frame 6	X-Frame 7	X-Frame 8
$Lx_j =$	0	18.94286	37.885714	56.82857	75.77143	94.71429	113.6571	132.6
$Wx_j =$	0	0	0	0	0	0	0	0
I/O	I	I	I	I	I	I	I	I
$\theta x_j =$	0.000	1.563	3.126	4.688	6.251	7.814	9.377	10.939

$Lx_j =$								
$Wx_j =$								
I/O								
$\theta x_j =$								

Cross Frame Location along centerline (CL) of the Girder (ft): Lx_j
 Weight of One Cross Frame (lbs): Wx_j
 Inside of Curve, Outside of Curve or Both (I, O, or I/O): I/O
 Internal Angle from Beginning (deg.): θx_j

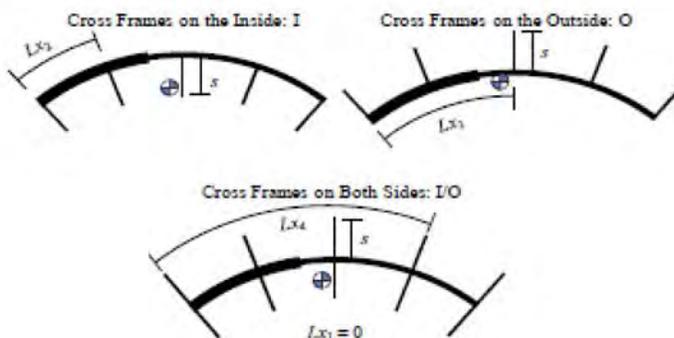
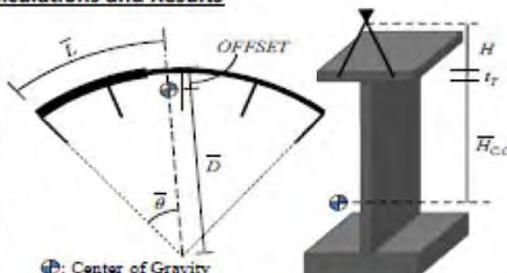


Figure B2-25: Page 3 of UT Lift Output for Girder G2 Field Section 3

10/31/2013

Girder Segment Center of Gravity Calculations and Results

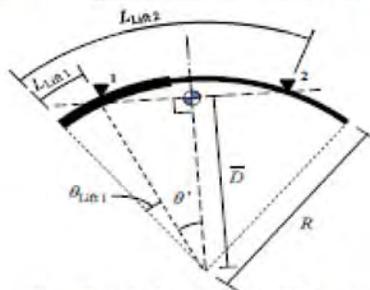
Total Length of Girder	
TOTAL_LENGTH =	132.60 ft
Total Girder Weight	
WEIGHT =	39334 lbs
	39.334 kips
Angular Distance to Center of Gravity	
$\bar{\theta}$ =	5.470 degrees
Distance along CL to Center of Gravity	
\bar{L} =	66.300 ft
Radial Distance to Center of Gravity	
\bar{D} =	693.446 ft
Offset of Center of Gravity from the CL	
OFFSET =	1.054 ft
Depth to Center of Gravity	
\bar{H}_{CG} =	43.073 in



- \oplus : Center of Gravity
- $\bar{\theta}$: Angular Distance to Center of Gravity
- \bar{L} : Length along Girder to Center of Gravity
- \bar{D} : Radial Distance to Center of Gravity
- H: Height of Axis of Rotation above the Top of Girder
- t_f : Top Flange Thickness
- \bar{H}_{CG} : Depth to Center of Gravity
- OFFSET: Radial Distance of Center of Gravity from \bar{C} of Girder

Zero Twist/Equal Force Lift Locations of a Nonprismatic Curved Girder

Angular Distance from Lift Pts. to Center of Gravity	
θ' =	3.158 degrees
Distance along CL to Lift Pt. 1	
L_{L1B1} =	28.026 ft
Distance along CL to Lift Pt. 2	
L_{L1B2} =	104.574 ft
Spreader Bar Needed	
ΔL =	76.510 ft
Lift Load (kips)	
LIFT_LOAD =	19.67 kips

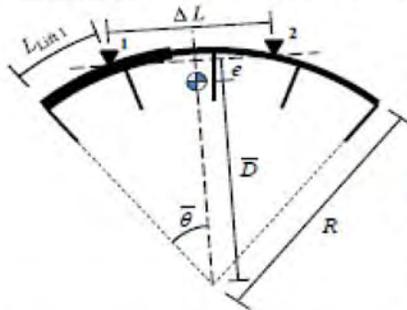


- L_{L1B1} : Length along Girder to Lift Pt. 1
- L_{L1B2} : Length along Girder to Lift Pt. 2
- θ_{L1B1} : Angular Distance to Lift Pt. 1
- θ_{L1B2} : Angular Distance to Lift Pt. 2
- \oplus : Center of Gravity
- θ' : Angular Distance from Lift Pts. to Center of Gravity
- \bar{D} : Radial Distance to Center of Gravity
- R: Radius of Curvature of the Girder

Figure B2-26: Page 3 of UT Lift Output for Girder G2 Field Section 3

10/31/2013

Predicted Rigid Body Twist and Lift Reactions of a Nonprismatic Curved Girder



- $L_{Lift 1}$: Length along Girder to Lift Pt. 1
- ΔL : Chord Length to Lift Pt. 2 (Spreader Bar)
- e : Eccentricity Between the Line of Support and the Center of Mass
- \oplus : Center of Gravity
- $\bar{\theta}$: Angular Distance to Center of Gravity
- \bar{D} : Radial Distance to Center of Gravity
- R : Radius of Curvature of the Girder

Distance along CL to Lift Pt. 1	$L_{Lift 1} = 28.791$ ft
---------------------------------	--------------------------

Chord Length Between Lift Pts.	$\Delta L = 75.000$ ft
--------------------------------	------------------------

Axis of Rotation above Top of Girder	$H = 30.000$ in
--------------------------------------	-----------------

Eccentricity	$e = 0.041$ ft 0.495 in
--------------	----------------------------

Twist	$\theta_{Rigid} = 0.007$ rad 0.383 degrees
-------	---

Reactions	Lift Point 1 = 19.67 kips Lift Point 2 = 19.66 kips
-----------	--

Calculate Twist & Stress

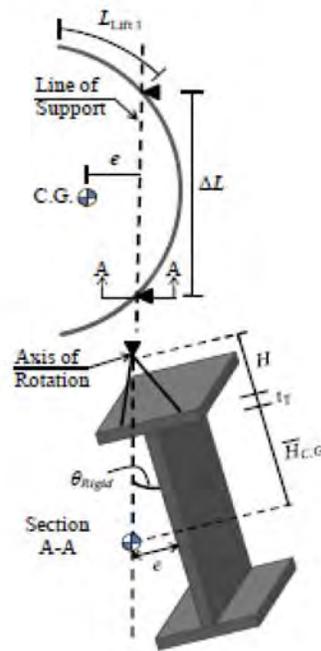


Figure B2-27: Page 4 of UT Lift Output for Girder G2 Field Section 3

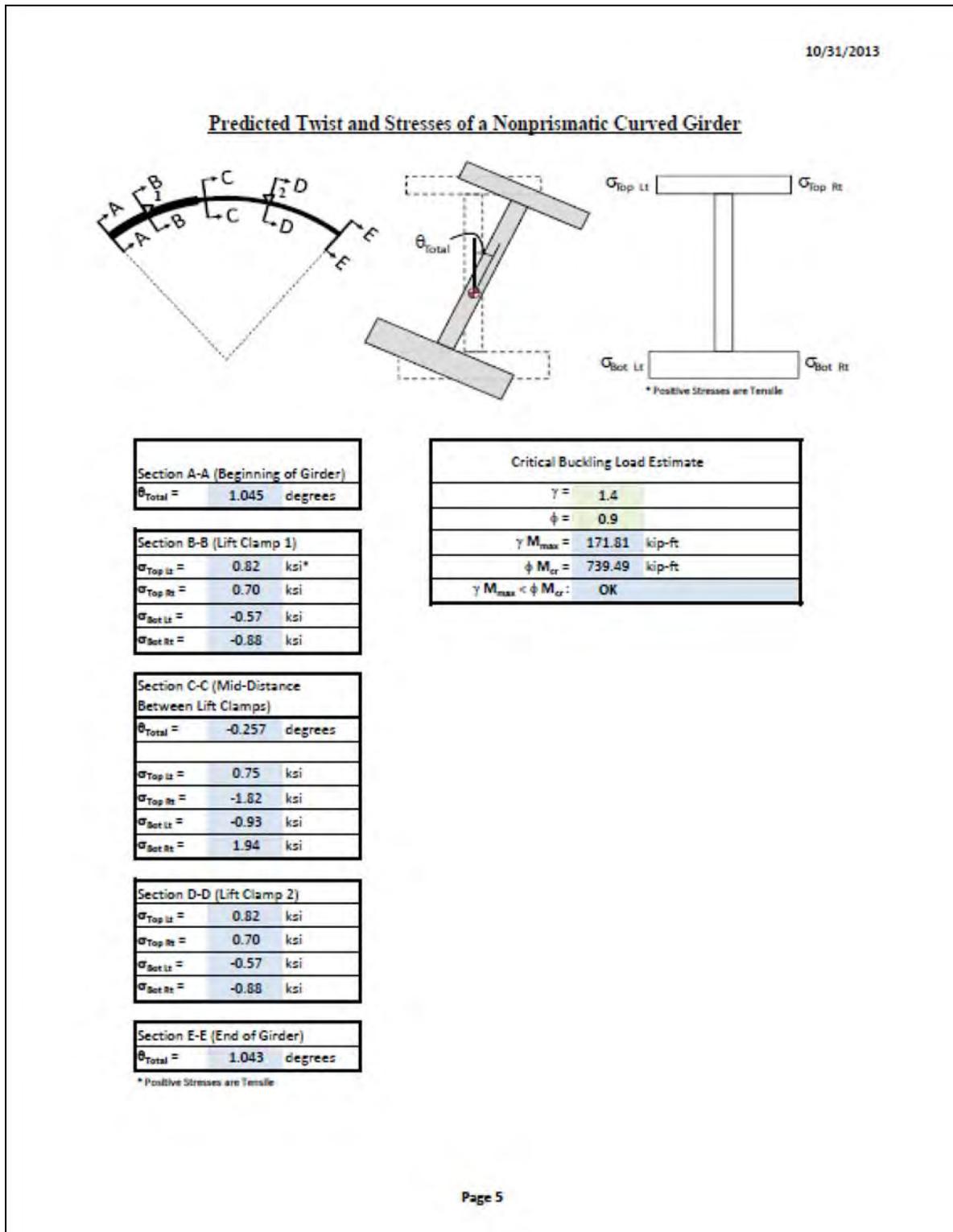


Figure B2-28: Page 5 of UT Lift Output for Girder G2 Field Section 3

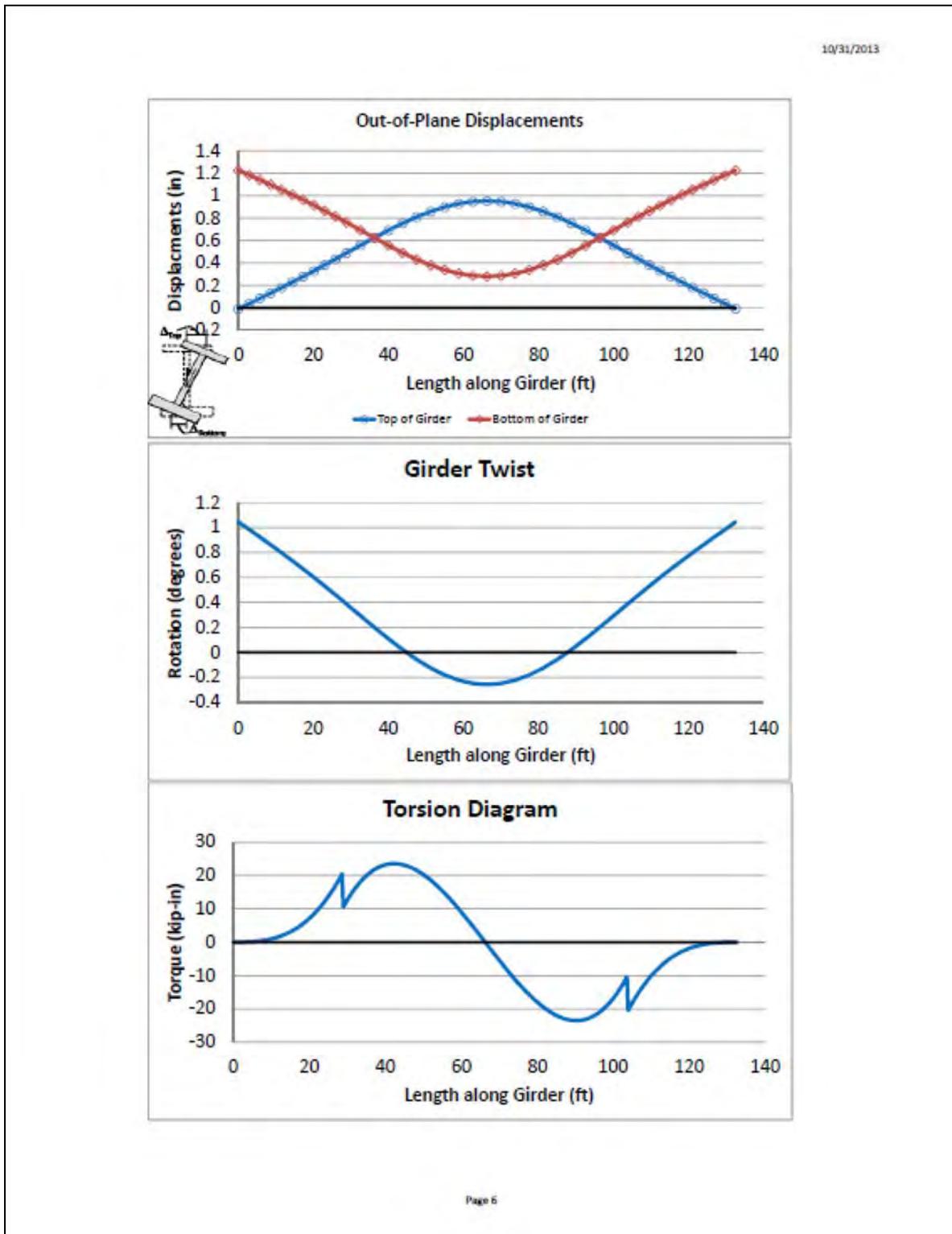


Figure B2-29: Page 6 of UT Lift Output for Girder G2 Field Section 3

Single Girder Pick of G1, Field Section 3 (without outside cross-frames attached)

See the following pages for the UT Lift 1.3 output (Excel spreadsheet). The chosen lift point is 27'-8 3/4" from the end of the girder with a chord length of 75'-0" between lift points. The reaction at each point is approximately 20 kips, which must not exceed the allowable capacity of the lift clamps.

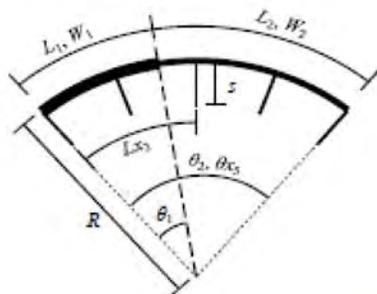
Also note that the calculated moment in the lifted curved girder is less than the buckling resistance (161.68 kip-ft vs. 814.21 kip-ft).

The maximum twist at the end of the girder $\theta_{total} := 0.768$ deg

Since the twist is less than 1.5 degrees, this should be adequate for fit-up. A separate run (not included here) showed that excessive twists would result from a pick of G1 with the outside cross-frames attached. Thus, girder G1 must be picked bare and the cross-frames then installed between G2 and G1 after both girders are in place.

10/31/2013

Behavior of Curved Girder During Lifting



- L_1 : Length of Section 1
- L_2 : Length of Section 2
- W_1 : Weight per Unit Length of Section 1
- W_2 : Weight per Unit Length of Section 2
- $\theta_0 = 0$
- θ_1 : Internal Angle from the Beginning of the Girder to the End of Section 1
- θ_2 : Internal Angle from the Beginning of the Girder to the End of Section 2
- s : Cross Frame Width
- L_{x3} : Length along the Girder to X-Frame 3
- θ_{x3} : Internal Angle from the Beginning of the Girder to X-Frame 3
- R : Radius of Curvature of the Girder

Girder Input:

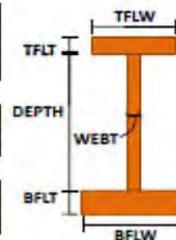
Project : FHWA Appendix B Example Problems
 Girder # : Example 2 - Girder G1 Field Section 3

Clear Input User Input
 Notable Results

Number of Cross Sections:	
NUMSECTIONS =	1
Radius of Curvature (ft): R	
R =	683.5 ft

Material Constants:	
Girder Scale Factor:	E = 29000 ksi
S.F. _{girder} =	G = 11154 ksi
	$\rho = 490 \text{ lbs/ft}^3$

Section 1							
TFLW	15						
TFLT	1						
DEPTH		84					
WEBT	0.5625						
BFLW		18					
BFLT	1						
Section 1							
$L_1 =$	130.5						
$W_1 =$	300.4						
$\theta_1 =$	10.94						
$J_1 =$	15.98						
$C_{wt} =$	1287152						



- Length of Cross Sections (ft): L_1
- Weight per Unit Length (lb/ft): W_1
- Internal Angle from Beginning (deg.): θ_1
- Top Flange Width & Thickness (in): TFLW, TFLT
- Web Thickness & Depth (in): WEBT DEPTH
- Bottom Flange Width & Thickness (in): BFLW, BFLT
- Polar Moment of Inertia (in^4): J_1
- Warping Constant (in^6): C_{wt}

Figure B2-30: Page 1 of UT Lift Output for Girder G1 Field Section 3

10/31/2013

Cross Frame Input:

Number of Cross Frame Locations:	
NUMXFRAMES =	8
Cross Frame Width: s	
$s =$	11.00 ft

Uniform Cross Frame Weights:	Constant X-Frame Weight
Weight =	

Uniformly Spaced Cross Frames:		Constant X-Frame Spacing	All Cross Frames on Inside of Curve, Outside of Curve, or Both		
Spacing =	18.642857 ft		All I	All O	All I/O
Location of the 1 st Cross Frame:					
1st X-Frame Loc. =	0 ft				

	X-Frame 1	X-Frame 2	X-Frame 3	X-Frame 4	X-Frame 5	X-Frame 6	X-Frame 7	X-Frame 8
$Lx_j =$	0	18.64286	37.285714	55.92857	74.57143	93.21429	111.8571	130.5
$Wx_j =$	0	0	0	0	0	0	0	0
I/O	0	0	0	0	0	0	0	0
$\theta_{x_j} =$	0.000	1.563	3.126	4.688	6.251	7.814	9.377	10.939

$Lx_j =$								
$Wx_j =$								
I/O								
$\theta_{x_j} =$								

Cross Frame Location along centerline (CL) of the Girder (ft): Lx_j
 Weight of One Cross Frame (lbs): Wx_j
 Inside of Curve, Outside of Curve or Both (I, O, or I/O): I/O
 Internal Angle from Beginning (deg.): θ_{x_j}

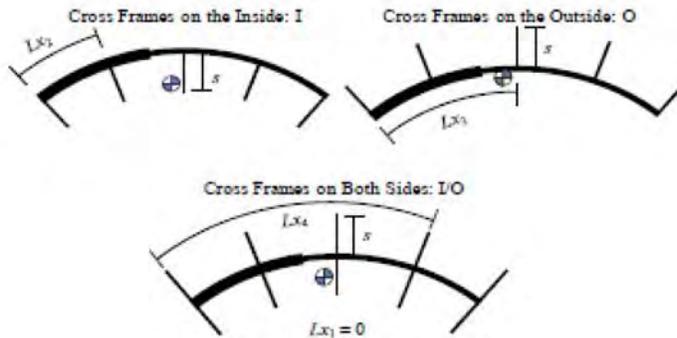


Figure B2-31: Page 2 of UT Lift Output for Girder G1 Field Section 3

10/31/2013

Girder Segment Center of Gravity Calculations and Results

Total Length of Girder	
TOTAL_LENGTH =	130.50 ft
Total Girder Weight	
WEIGHT =	39200 lbs
	39,200 kips

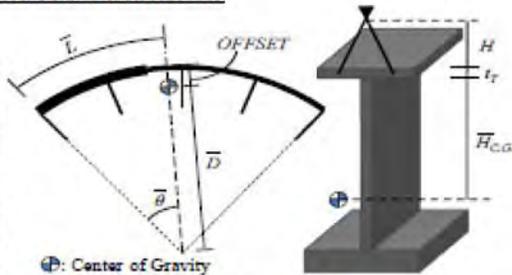
Angular Distance to Center of Gravity	
$\bar{\theta}$ =	5.470 degrees

Distance along CL to Center of Gravity	
\bar{L} =	65.250 ft

Radial Distance to Center of Gravity	
\bar{D} =	682.462 ft

Offset of Center of Gravity from the CL	
OFFSET =	1.038 ft

Depth to Center of Gravity	
\bar{H}_{CG} =	43.589 in



- \oplus : Center of Gravity
- $\bar{\theta}$: Angular Distance to Center of Gravity
- \bar{L} : Length along Girder to Center of Gravity
- \bar{D} : Radial Distance to Center of Gravity
- H : Height of Axis of Rotation above the Top of Girder
- t_f : Top Flange Thickness
- \bar{H}_{CG} : Depth to Center of Gravity
- OFFSET: Radial Distance of Center of Gravity from \bar{Q} of Girder

Zero Twist/Equal Force Lift Locations of a Nonprismatic Curved Girder

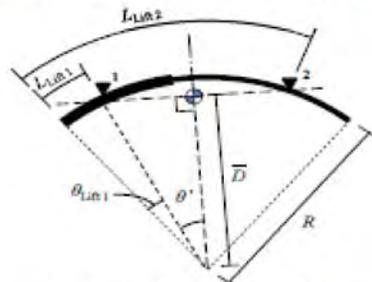
Angular Distance from Lift Pts. to Center of Gravity	
θ' =	3.158 degrees

Distance along CL to Lift Pt. 1	
$L_{Lift 1}$ =	27.582 ft

Distance along CL to Lift Pt. 2	
$L_{Lift 2}$ =	102.918 ft

Spreader Bar Needed	
ΔL =	75.298 ft

Lift Load (kips)	
LIFT_LOAD =	19.60 kips

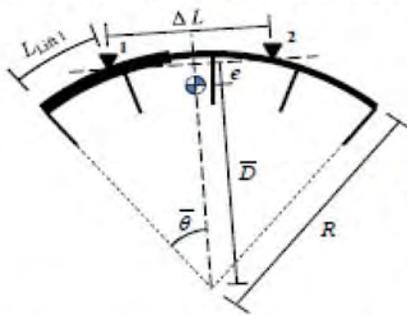


- $L_{Lift 1}$: Length along Girder to Lift Pt. 1
- $L_{Lift 2}$: Length along Girder to Lift Pt. 2
- $\theta_{Lift 1}$: Angular Distance to Lift Pt. 1
- $\theta_{Lift 2}$: Angular Distance to Lift Pt. 2
- \oplus : Center of Gravity
- θ' : Angular Distance from Lift Pts. to Center of Gravity
- \bar{D} : Radial Distance to Center of Gravity
- R : Radius of Curvature of the Girder

Figure B2-32: Page 3 of UT Lift Output for Girder G1 Field Section 3

10/31/2013

Predicted Rigid Body Twist and Lift Reactions of a Nonprismatic Curved Girder



- $L_{Lift 1}$: Length along Girder to Lift Pt. 1
- ΔL : Chord Length to Lift Pt. 2 (Spreader Bar)
- e : Eccentricity Between the Line of Support and the Center of Mass
- \oplus : Center of Gravity
- $\bar{\theta}$: Angular Distance to Center of Gravity
- \bar{D} : Radial Distance to Center of Gravity
- R : Radius of Curvature of the Girder

Distance along CL to Lift Pt. 1
$L_{Lift 1} = 27.729$ ft

Chord Length Between Lift Pts.
$\Delta L = 75.000$ ft

Axis of Rotation above Top of Girder
$H = 30.000$ in

Eccentricity
$e = 0.008$ ft
0.099 in

Twist
$\theta_{Rigid} = 0.001$ rad
0.076 degrees

Reactions
Lift Point 1 = 19.60 kips
Lift Point 2 = 19.60 kips

Calculate Twist & Stress

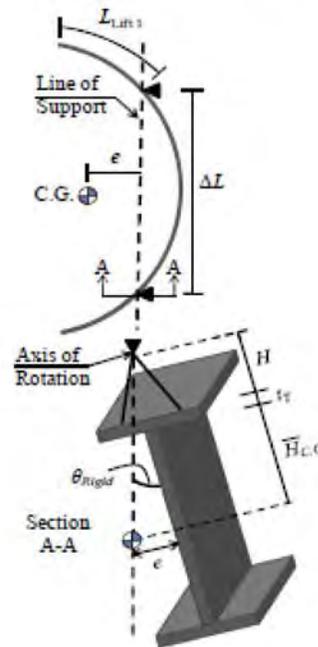
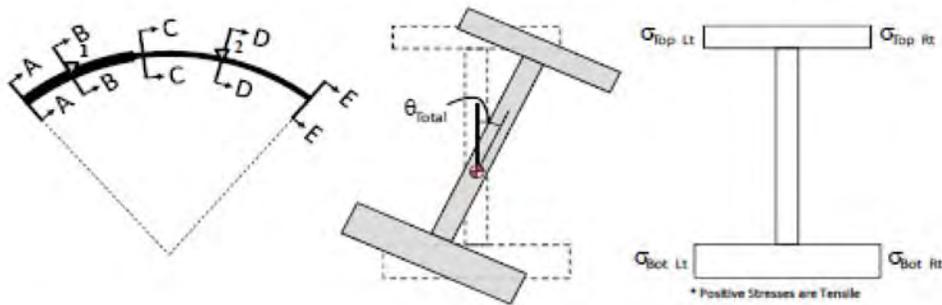


Figure B2-33: Page 4 of UT Lift Output for Girder G1 Field Section 3

10/31/2013

Predicted Twist and Stresses of a Nonprismatic Curved Girder



Section A-A (Beginning of Girder)	
θ_{Total} =	0.767 degrees

Section B-B (Lift Clamp 1)	
$\sigma_{Top Lt}$ =	0.60 ksi*
$\sigma_{Top Rt}$ =	0.81 ksi
$\sigma_{Bot Lt}$ =	-0.52 ksi
$\sigma_{Bot Rt}$ =	-0.79 ksi

Section C-C (Mid-Distance Between Lift Clamps)	
θ_{Total} =	-0.639 degrees
$\sigma_{Top Lt}$ =	0.90 ksi
$\sigma_{Top Rt}$ =	-2.08 ksi
$\sigma_{Bot Lt}$ =	-0.97 ksi
$\sigma_{Bot Rt}$ =	2.07 ksi

Section D-D (Lift Clamp 2)	
$\sigma_{Top Lt}$ =	0.61 ksi
$\sigma_{Top Rt}$ =	0.81 ksi
$\sigma_{Bot Lt}$ =	-0.52 ksi
$\sigma_{Bot Rt}$ =	-0.80 ksi

Section E-E (End of Girder)	
θ_{Total} =	0.768 degrees

* Positive Stresses are Tensile

Critical Buckling Load Estimate	
γ =	1.4
ϕ =	0.9
γM_{max} =	161.68 kip-ft
ϕM_{cr} =	814.21 kip-ft
$\gamma M_{max} < \phi M_{cr}$:	OK

Figure B2-34: Page 5 of UT Lift Output for Girder G1 Field Section 3

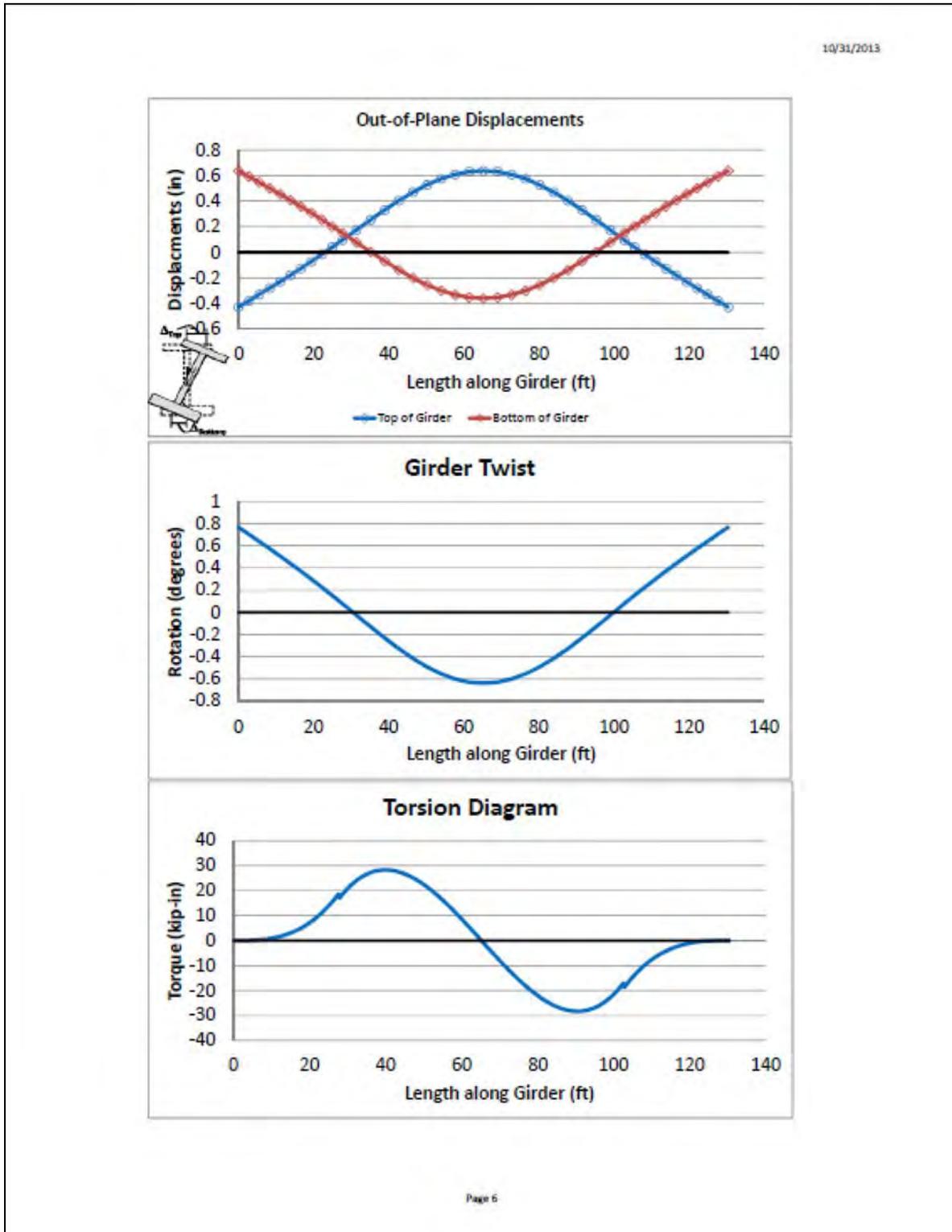


Figure B2-35: Page 6 of UT Lift Output for Girder G1 Field Section 3

Erection of G3 and G4 in Span 3 (with all cross-frames and no shoring)

The entire structure was analyzed for the erection sequence step by step in UT Bridge. Several critical erection steps were selected to show as part of this example. The structure was erected from Abutment 2 to Abutment 1. The first critical case analyzed consists of the first two girders of Span 3 erected with all the cross-frames installed between the girders. See Figures B2-37 and B2-38 for output of this case.

The minimum vertical positive reaction at the interior girder at the Abutment is 11.2 kips. Therefore, the two girder system would not require temporary shoring or tie-downs at the bearings due to uplift concerns. The lateral displacement of the two girder system is 27 inches which is equal to $L/70$ for the 160 feet long span. This does not meet the deflection criteria of $L/150$ presented in Section D-7. In addition, the Eigenvalue for the two girder system is 0.5, which is not adequate and indicates the girder system would buckle spanning from the abutment to the pier with no additional supports.

Note that this analysis and subsequent erection sequence analyses in this example problem use the Strength III: 1.25 DC + 1.0 CW load combination, so the presented Eigenvalues, deflections, and span ratios are actually factored (conservative). If more exact results are desired, the designer could rerun the analyses with service loads to get the true service level deflections, as well as the unfactored Eigenvalues.

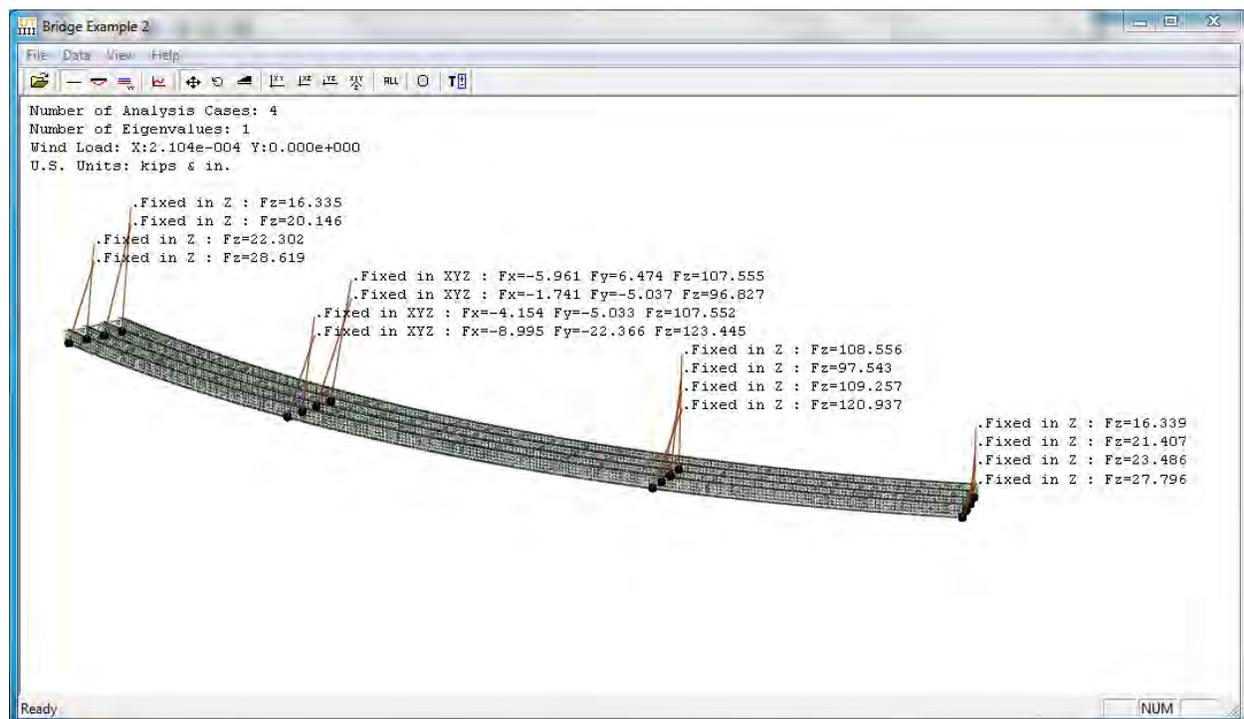


Figure B2-36: Analysis of Fully Erected Bridge

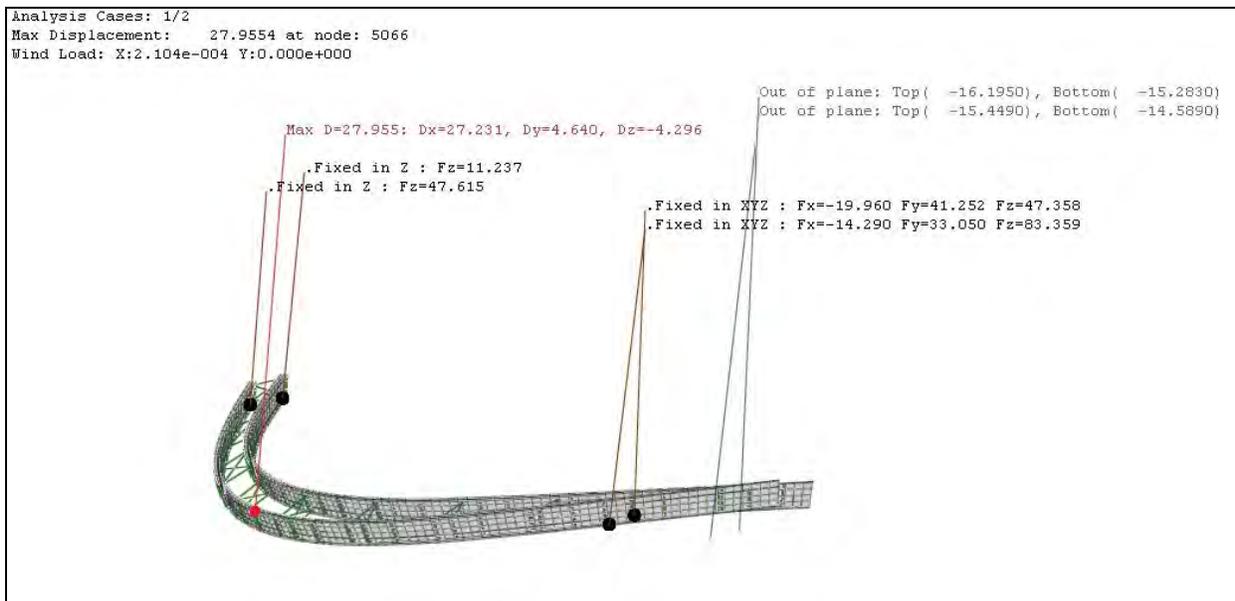


Figure B2-37: Girder Deflection During Two Girder Erection

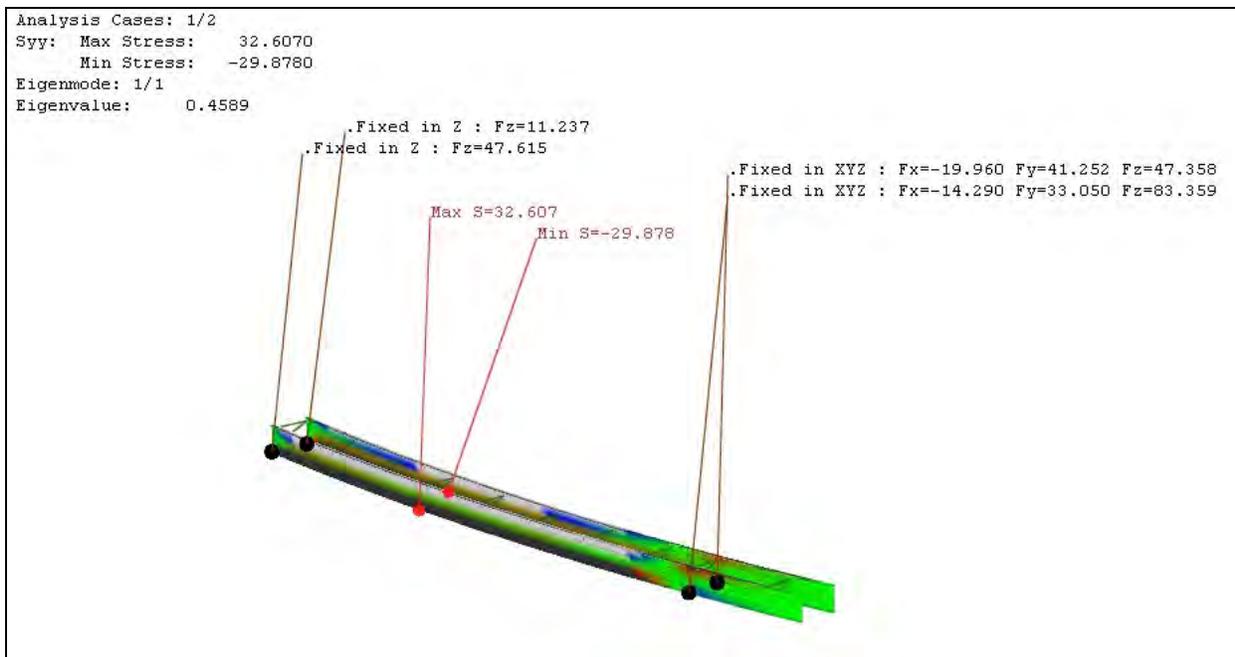


Figure B2-38: Girder Stresses and Eigenvalue Analysis During Two Girder Erection

Erection of G3 and G4 in Span 3 (with all cross-frames and shoring)

Based on the previous analysis, temporary shoring or a hold crane is required to support the two girder system during erection of Span 3. Temporary shoring supports were added to the UT Bridge analysis model at the midspan of Span 3. See Figures B2-39 and B2-40 for output of the two girder system with shoring.

The maximum lateral deflection of a girder in the system is 2.9 inches, which equates to $L/660$. The Eigenvalue has increased to 12.1 indicating an adequate factor of safety against buckling. The maximum girders stresses are at 11.1 ksi. These stresses were found to be adequate per AASHTO constructability criteria but for brevity the calculations for girder resistance are only included in the deck pour analysis located at the end of this example.

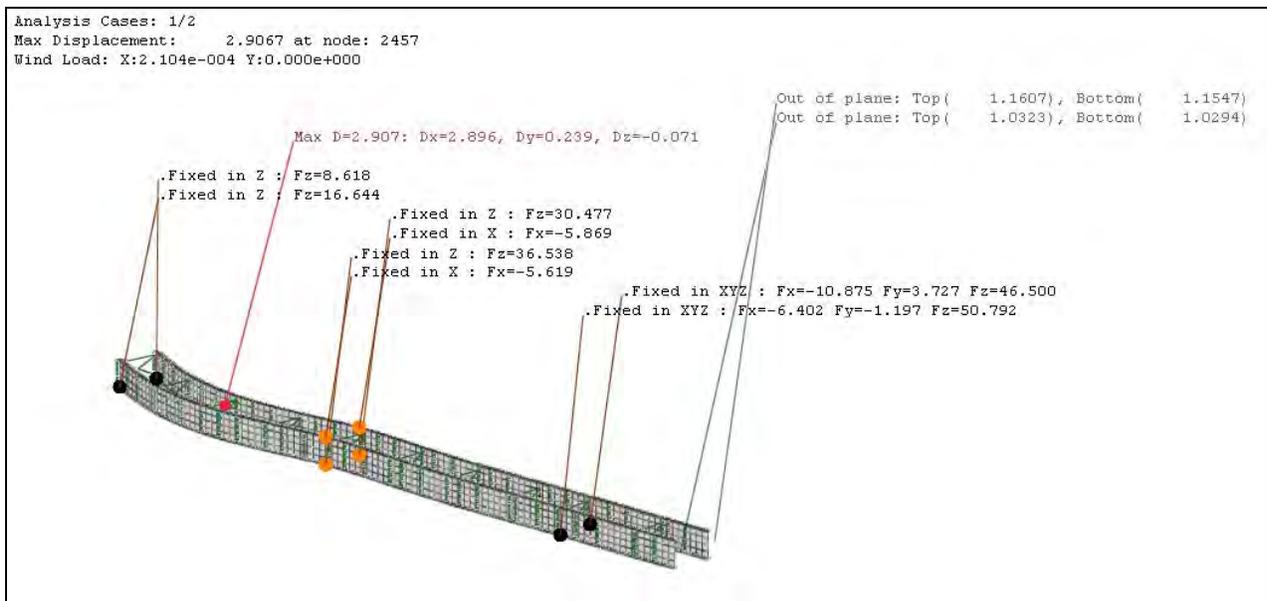


Figure B2-39: Girder Deflection During Two Girder Erection with Shoring

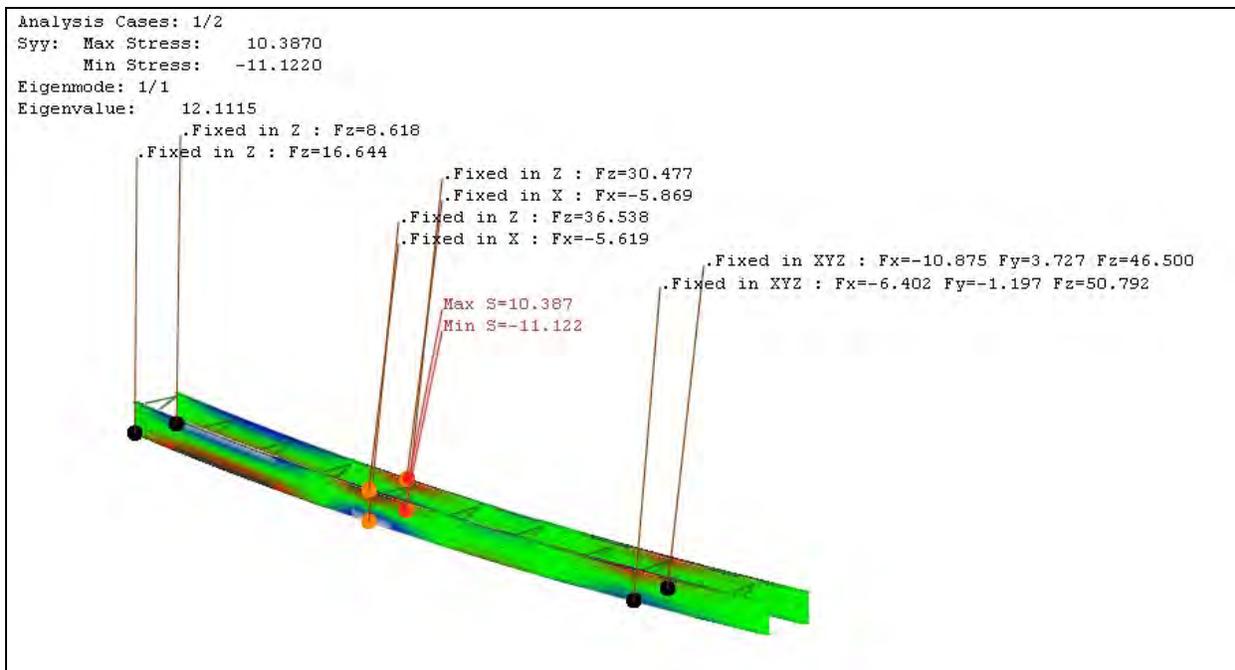


Figure B2-40: Girder Stresses and Eigenvalue Analysis, Two Girders with Shoring

Erection of G3 and G4 in Span 3 (with half of the cross-frames and shoring)

Based on the previous analysis, the two girder system is adequate with temporary shoring at midspan. The contractor would like to erect the girders with only half of the cross-frames installed to speed up total erection time. The remaining cross-frames would then be installed once the bridge is fully erected prior to the deck pour. The two girder system was analyzed with every other cross-frame removed in UT Bridge as shown in Figure B2-41. The Eigenvalue of 0.29 indicates the girder would buckle due to the increased unbraced length from removing cross-frames. Therefore, the contractor will erect the bridge with all cross-frames installed.

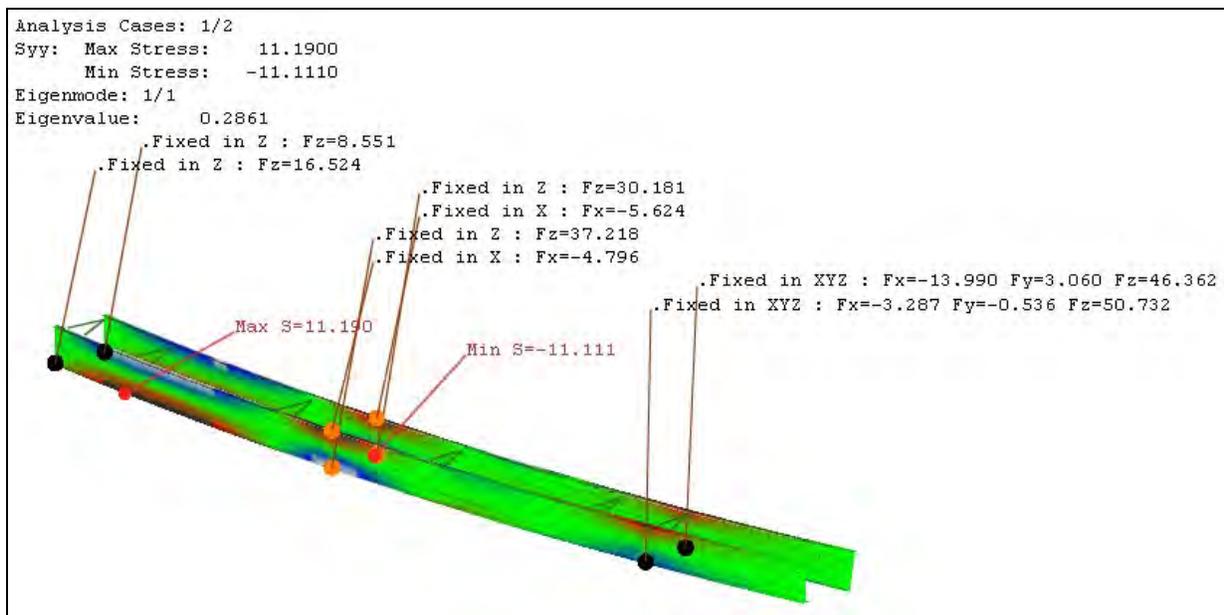


Figure B2-41: Girder Stresses and Eigenvalue Analysis, Two Girders with Shoring, Half of the Cross-Frames

Erection of G1 through G4 in Span 3 (with all cross-frames and no shoring)

The contractor would prefer to remove the shoring in Span 3 once all four girders in Span 1 are erected. Figures B2-42 and B2-43 examine the fully erected Span 3 with no shoring. The Eigenvalue of 5.9 represents an adequate factor of safety against buckling of the system. However, the maximum lateral displacement of the system is 11.8 inches which equates to $L/160$. While this is within criteria of $L/150$ presented in Section D-7, the contractor has been advised and agreed to keep the Span 3 midspan shoring in place until the erection of Span 2 is complete. The lateral deflection of approximately 12 inches is significant and may cause fit-up issues with the Span 2 erection.

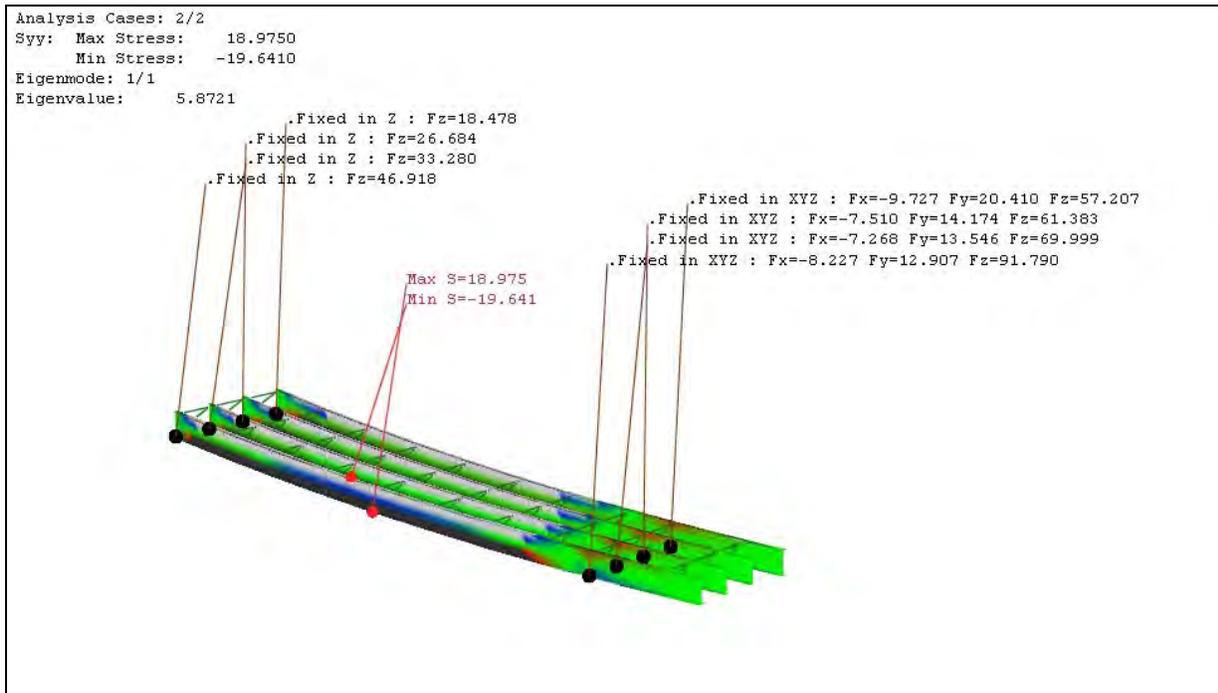


Figure B2-42: Girder Stresses and Eigenvalue Analysis, Four Girders with no Shoring

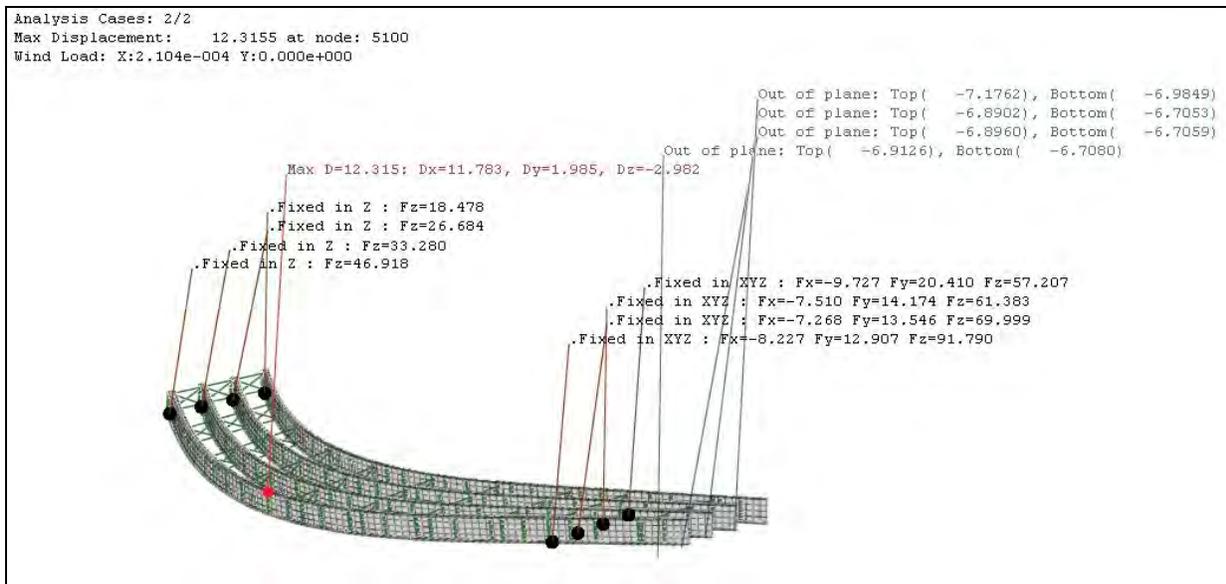


Figure B2-43: Girder Deflection During Four Girder Erection with no Shoring

Erection of G1 through G4 in Span 3 (with all cross-frames and shoring)

The lateral deflection for the four girder system is reduced to 1.1 inches with the midspan shoring in place as shown in Figure B2-44. This is adequate and will allow for proper fit-up during the Span 2 erection.

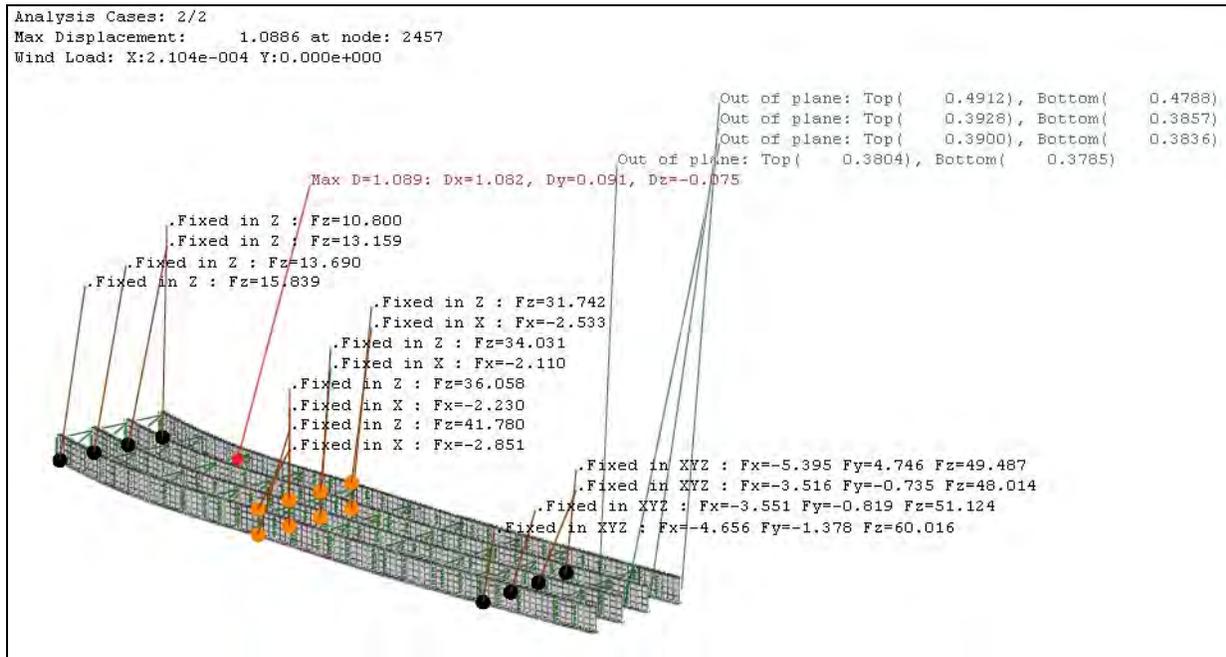


Figure B2-44: Girder Deflection During Four Girder Erection with Shoring

Erection of G1 and G2 in Span 1 (with all cross-frames and no shoring)

The completed structure in Span 3 and 2 provides additional stability for the erection of Girders 1 and 2 in Span 1. The Girders in Span 1 are adequate to span from pier to abutment without additional shoring. The worst case girder deflection at this stage occurs in Span 3 with a lateral deflection of 7.0 inches. This is equivalent to $L/275$ and is adequate based on Section D-7. The Eigenvalue of 3.7 for this stage provides adequate safety against buckling.

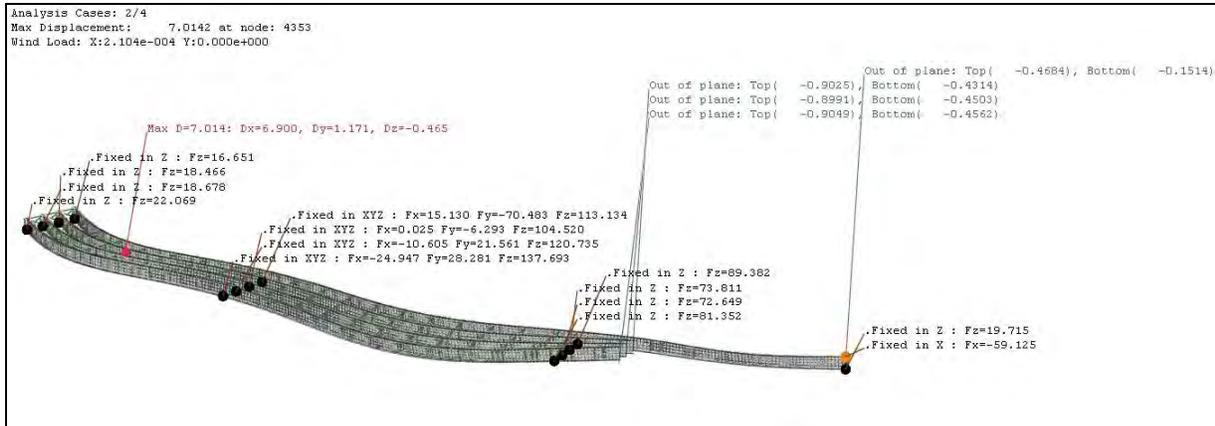


Figure B2-45: Girder Deflection During Span 1 Erection

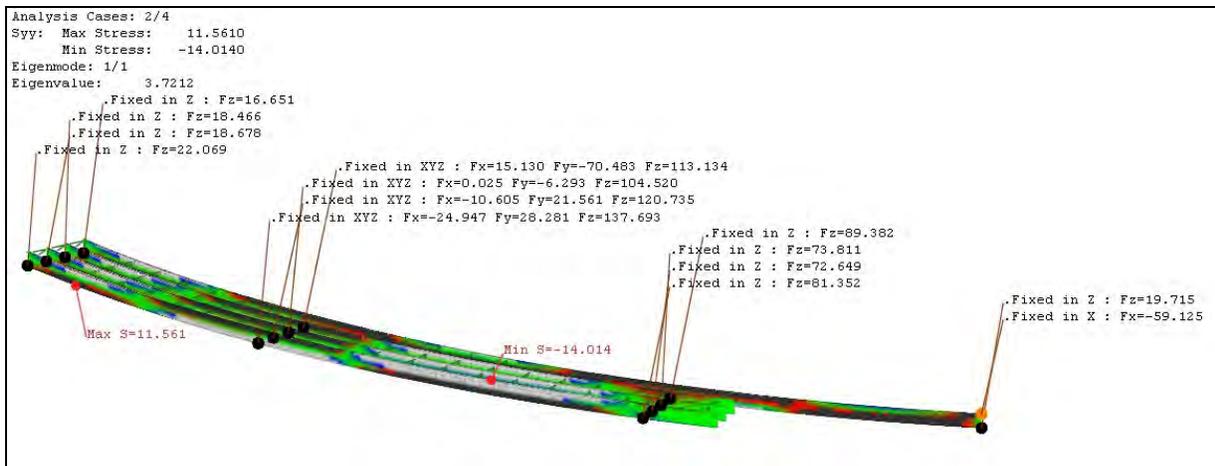


Figure B2-46: Girder Stresses and Eigenvalue Analysis During Span 1 Erection

Adequacy of the Bearings During Erection

Depending on the bearing type and possible blocking setup, it may be important to check the bearings for rotations and reactions during stages of the bridge construction. The reactions and rotations can be output from UT Bridge at each bearing during the various stages of erection and deck pour. For this example, the bearing loads and rotations will be examined for one of the critical stages. Abutment 2 bearings experience greater than normal service level rotations and less than normal service level dead load reactions during the erection of Span 1. This situation of high rotation and low vertical reaction could compromise some types of bearings. The vertical reactions (16.7 kips for Girder 1 in this example) and rotations at the bearing (0.08 degrees for Girder 1 in this example) should be checked against the limits of the bearings. Often times in curved bridges, bearings are specially detailed by a fabricator for the specific loading conditions of the bridge and the bearing fabricator should be contacted to determine the adequacy of the bearings during the construction stages.

Resistance of Girder G4, Field Section 1 for First Cast During Concrete Deck Pour

Analysis and Methodology:

The entire deck pour is modeled in UT Bridge via four analysis cases. The first cast, which occurs in Span 3, corresponds to Analysis Case 1. Refer to Figure B2-47 for the reactions from this analysis.

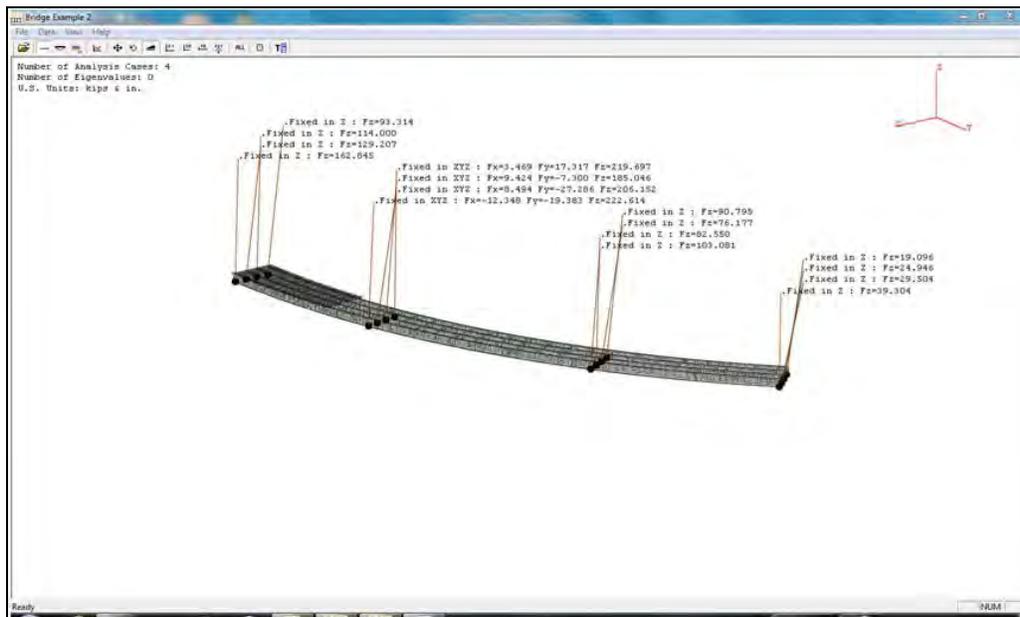


Figure B2-47: Reactions During Case 1 (1st Deck Pour)

Refer to Figure B2-48 for the normal stresses in the girders from this analysis.

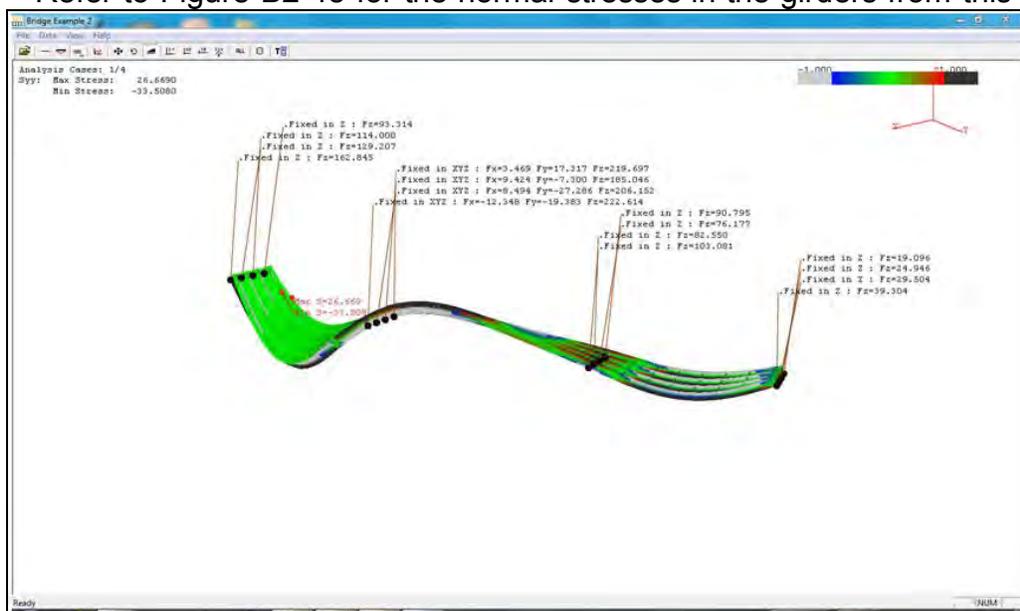


Figure B2-48: Normal Stresses During Case 1 (1st Deck Pour)

Note that in the UT Bridge model, the span numbering is reversed relative to the framing plan of Figure B2-2. Since the bridge is symmetric, this makes no difference to the analysis.

Resistance of Girder G4, Field Section 1 for First Cast During Concrete Deck Pour, continued

Because this is a continuous bridge, the maximum positive moment in the end span should occur at approximately 0.4L into that span, where L is the length of the end span. Refer to Exhibits B2-49, B2-50, B2-51, and B2-52, which offer visual proof of this by plotting normal stresses in the four girders (one exhibit per each flange tip) throughout the length of the bridge.

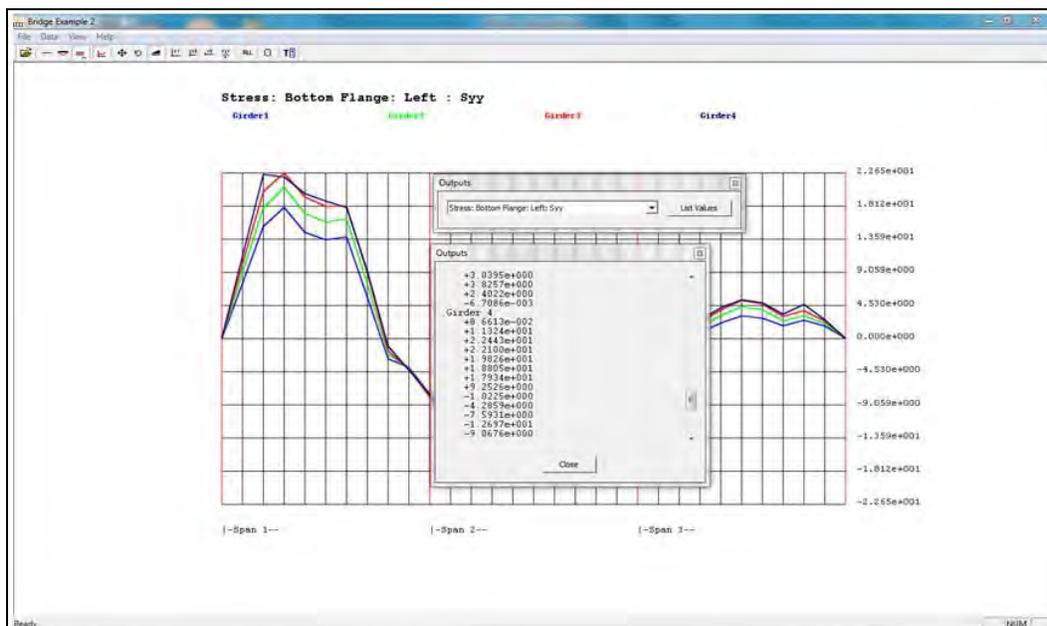


Figure B2-49: Girder G4 Normal Stresses at Left Tip of Bottom Flange for Analysis Case 1 (1st Deck Pour)



Figure B2-50: Girder G4 Normal Stresses at Right Tip of Bottom Flange for Analysis Case 1 (1st Deck Pour)

Resistance of Girder G4, Field Section 1 for First Cast During Concrete Deck Pour, continued

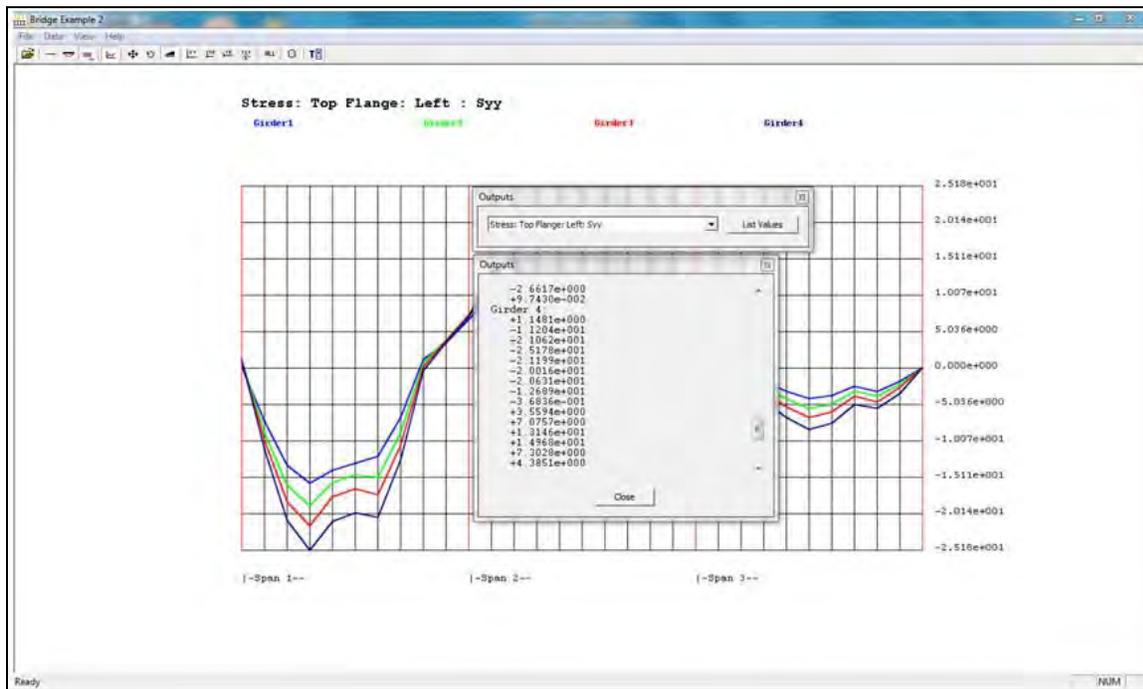


Figure B2-51: Girder G4 Normal Stresses at Left Tip of Top Flange for Analysis Case 1 (1st Deck Pour)

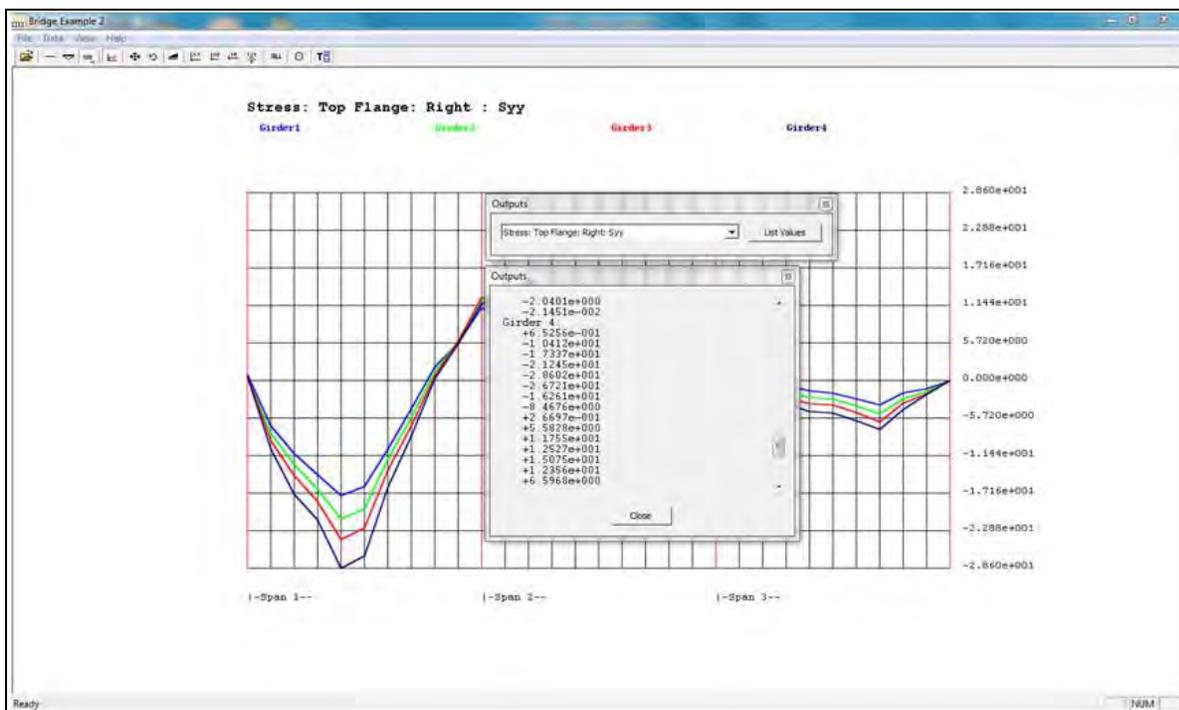


Figure B2-52: Girder G4 Normal Stresses at Right Tip of Top Flange for Analysis Case 1 (1st Deck Pour)

Resistance of Girder G4, Field Section 1 for First Cast During Concrete Deck Pour, continued

The AASHTO equations require the stresses in the flanges to be broken up into components (due only to lateral bending, as well as without lateral bending), but most analysis programs do not provide output in this fashion. Therefore, it is necessary to either make some assumptions or otherwise manipulate the data. One approach would be to look at the normal stresses at the location where the maximum positive moment in Span 1 occurs (approximately at $0.4L$) and average the sum of the flange tip stresses (from Figures B2-49 through B2-52 above) to compute the bending stress component. This approach is adopted in this example below. The warping stresses due to lateral bending could be computed by taking the average of the difference of the flange tip stresses; refer to Figure B2-53 for a diagram illustrating the decomposition of stresses into normal and warping.

Another approach for the warping stresses, adopted in this example below, is to use approximations to account for lateral bending due to the curvature and the deck overhang brackets.

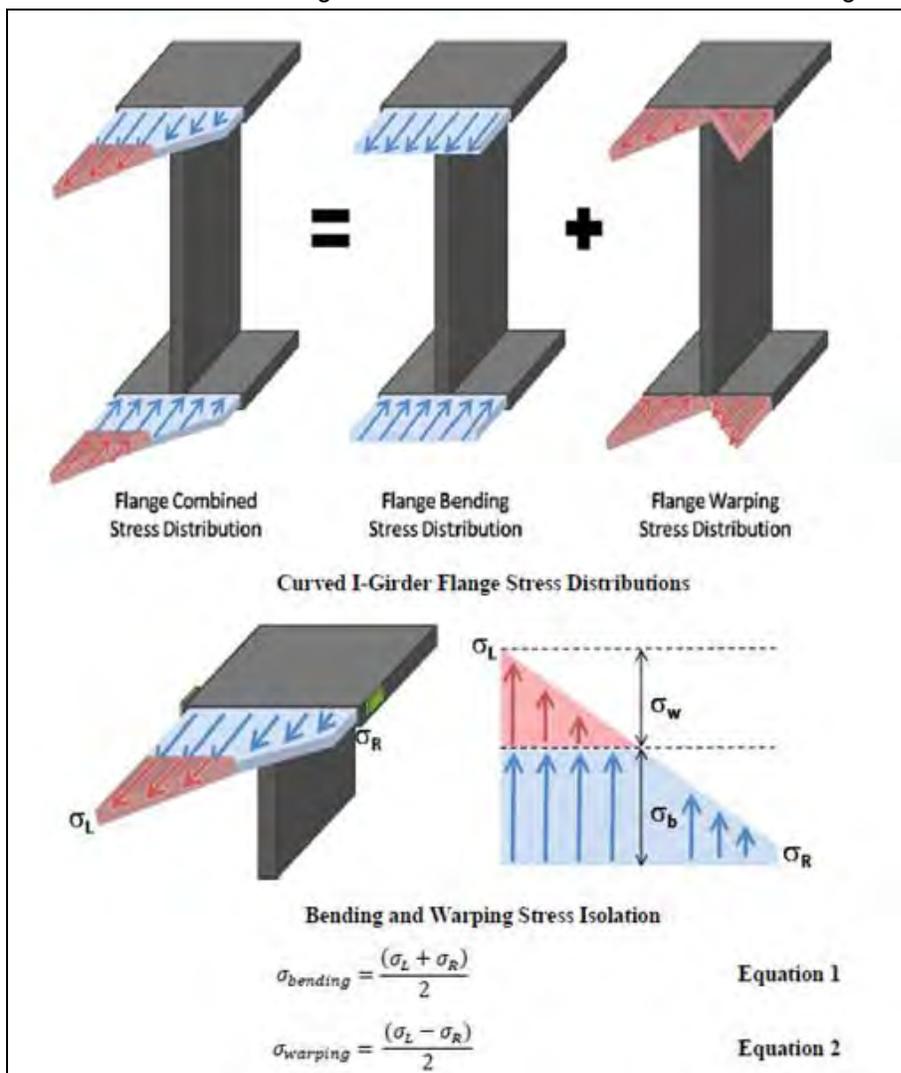


Figure B2-53: Normal and Warping Stresses in Curved I-Girders

**Resistance of Girder G4, Field Section 1 for First Cast During Concrete Deck Pour, continued
Section Properties:**

$$E_s := 29000 \text{ ksi} \quad F_y := 50 \text{ ksi}$$

Flanges:

$$t_{tf} := 1 \text{ in} \quad b_{bf} := 20 \text{ in} \quad t_{bf} := 1.5 \text{ in} \quad b_{bf} := 21 \text{ in}$$

Web:

Overall Depth:

$$t_w := \frac{9}{16} \text{ in} \quad D := 84 \text{ in} \quad d := D + t_{tf} + t_{bf} \quad d = 86.5 \text{ in}$$

Calculated Properties:

$$A_{bf} := t_{bf} \cdot b_{bf} = 31.5 \text{ in}^2 \quad y_{bf} := 0.5 \cdot t_{bf} = 0.75 \text{ in}$$

$$A_w := t_w \cdot D = 47.25 \text{ in}^2 \quad y_w := t_{bf} + 0.5 \cdot D = 43.5 \text{ in}$$

$$A_{tf} := t_{tf} \cdot b_{tf} = 20 \text{ in}^2 \quad y_{tf} := t_{bf} + D + 0.5 \cdot t_{tf} = 86 \text{ in}$$

$$A_g := A_{bf} + A_w + A_{tf} = 98.75 \text{ in}^2$$

Area of girder

$$Y_b := \frac{A_{bf} \cdot y_{bf} + A_w \cdot y_w + A_{tf} \cdot y_{tf}}{A_{bf} + A_w + A_{tf}} = 38.47 \text{ in}$$

Centroid measured
from bottom

$$Y_t := d - Y_b = 48.03 \text{ in}$$

Centroid measured
from top

$$I_x := \frac{b_{bf} \cdot t_{bf}^3}{12} + \frac{t_w \cdot D^3}{12} + \frac{b_{tf} \cdot t_{tf}^3}{12} + A_{bf} \cdot (Y_b - y_{bf})^2 + A_w \cdot (Y_b - y_w)^2 + A_{tf} \cdot (Y_b - y_{tf})^2 = 118986.2 \text{ in}^4$$

Major axis moment
of inertia

$$S_{bx} := \frac{I_x}{Y_b} = 3092.9 \text{ in}^3$$

Major axis elastic section
modulus to bottom

$$S_{tx} := \frac{I_x}{Y_t} = 2477.4 \text{ in}^3$$

Major axis elastic section
modulus to top

$$I_y := \frac{t_{bf} \cdot b_{bf}^3}{12} + \frac{D \cdot t_w^3}{12} + \frac{t_{tf} \cdot b_{tf}^3}{12} = 1825.5 \text{ in}^4$$

Major axis moment
of inertia

$$S_y := \frac{I_y}{0.5 \cdot \max(b_{bf}, b_{tf})} = 173.9 \text{ in}^3$$

Minimum minor axis
elastic
section modulus

Resistance of Girder G4, Field Section 1 for First Cast During Concrete Deck Pour, continued
Applied Factored Forces: Note: Major-axis moment is based on average flange tip stresses at $0.4L$

$$\text{Major-axis Bending: } M_{ux} := \max \left[\frac{(21.20 \text{ ksi} + 28.60 \text{ ksi})}{2} \cdot S_{tx}, \frac{(19.83 \text{ ksi} + 22.60 \text{ ksi})}{2} \cdot S_{bx} \right] = 5468 \text{ kip ft}$$

$$\text{Unbraced Length: } L_b := \frac{123 \text{ ft}}{6} = 20.5 \text{ ft with cross-frame spacing approximately 20 ft}$$

Lateral Flange Bending: Note: Lat. Bending could also be based on average flange tip stress difference

$$\text{Radius of curvature } R := 700 \text{ ft} + \frac{33 \text{ ft}}{2} = 716.5 \text{ ft at Girder G4} \quad \text{Constant } N := 10$$

$$\text{Moment in each flange due to lateral bending from curvature } M_{lat} := \frac{M_{ux} \cdot L_b^2}{N \cdot R \cdot D} \quad (\text{Eq. C4.6.1.2.4b-1})$$

$$M_{lat} = 45.8 \text{ kip ft} \quad \text{Note that this value is already factored since the major-axis moment was factored}$$

Minor-axis Bending: $M_{uy} := 0 \text{ kip ft}$ Assume wind load on girders in negligible during deck pour

Overhang Bracket Forces at Fascia Girder:

Assume bracket loads are applied uniformly along girder although actual bracket spacing = 3 ft

$$\text{overhang} := \frac{40.5 \text{ ft} - 33 \text{ ft}}{2} \quad \text{overhang} = 3.75 \text{ ft} \quad \text{at fascia}$$

Assume 10" thick deck at fascia overhang; $\frac{1}{2}$ of weight goes to bracket and $\frac{1}{2}$ directly to girder

$$w_{fascia} := 0.5 \cdot \text{overhang} \cdot 10 \text{ in} \cdot 150 \text{ pcf} \quad w_{fascia} = 234.4 \text{ plf} \quad \text{Concrete weight to bracket}$$

Assume 200 plf for screed rail at fascia overhang

$$w_{rail} := 200 \text{ plf} \quad \text{Rail weight to bracket}$$

Assume 10 psf for forms and another 5 psf for bracket components and miscellaneous at fascia overhang; $\frac{1}{2}$ of weight goes to bracket and $\frac{1}{2}$ directly to girder

$$w_{forms} := 0.5 \cdot \text{overhang} \cdot (10 \text{ psf} + 5 \text{ psf}) \quad w_{forms} = 28.1 \cdot \text{plf} \quad \text{Form weight to bracket}$$

$$w_{bracket} := w_{fascia} + w_{rail} + w_{forms} \quad w_{bracket} = 462.5 \text{ plf} \quad \text{Total uniform load to bracket}$$

$$\alpha := \text{atan} \left(\frac{\text{overhang}}{D} \right) = 28.2 \text{ deg} \quad \text{Angle of bracket relative to fascia girder web}$$

$$F_L := w_{bracket} \tan(\alpha) = 247.8 \text{ plf} \quad \text{Lateral force on ea. flange from bracket}$$

$$\text{Lateral Moment in Each Flange from Overhang Bracket at Fascia Girder: } M_L := \frac{F_L \cdot L_b^2}{12} \quad (\text{Eq. C6.10.3.4-2})$$

Factored Lat. Moment in Ea. Flange from Overhang Bracket at Fascia Girder: $M_{Lu} := 1.50 M_L = 13 \text{ kip ft}$
 (conservatively assume highest load factor corresponding to all construction dead load)

Resistance of Girder G4, Field Section 1 for First Cast During Concrete Deck Pour, continued**Flexural Resistance:**

Top flange is in compression:	$b_{fc} := b_{tf}$	$t_{fc} := t_{tf}$
Bottom flange is in tension:	$b_{ft} := b_{bf}$	$t_{ft} := t_{bf}$

Applied Stresses:

Top flange is in compression:	$S_{yc} := \frac{t_{fc} \cdot b_{fc}^2}{6}$	$S_{yc} = 66.7 \text{ in}^3$
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Bottom flange is in tension:	$S_{yt} := \frac{t_{ft} \cdot b_{ft}^2}{6}$	$S_{yt} = 110.2 \text{ in}^3$
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Stress in compression flange without consideration of lateral bending:	$f_{buc} := \frac{M_{ux}}{S_{bx}}$	$f_{buc} = 26.5 \text{ ksi}$
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First-order stress due to lateral bending in compression flange:	$f_{L1c} := \frac{0.5M_{uy} + M_{lat} + M_{Lu}}{S_{yc}}$	$f_{L1c} = 10.6 \text{ ksi}$
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Stress in tension flange without consideration of lateral bending:	$f_{but} := \frac{M_{ux}}{S_{bx}}$	$f_{but} = 21.2 \text{ ksi}$
--	------------------------------------	------------------------------

First-order stress due to lateral bending in tension flange:	$f_{Lt} := \frac{0.5M_{uy} + M_{lat} + M_{Lu}}{S_{yt}}$	$f_{Lt} = 6.4 \text{ ksi}$
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Flange Strength Reduction Factors:

Hybrid Factor (AASHTO 6.10.1.10.1)

Since the flexural member is a homogenous built-up section, the hybrid factor shall be taken as unity

$$R_h := 1$$

Web Load Shedding Factor (AASHTO 6.10.1.10.2)

$$\text{Depth of web in compression } D_c := Y_t - t_{tf}$$

$\lambda_{rw} := \left(5.7 \cdot \sqrt{\frac{E_s}{F_y}} \right)$	$\lambda_{rw} = 137.3$	$\frac{2 \cdot D_c}{t_w} = 167.2$
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$a_{wc} := \frac{2D_c \cdot t_w}{b_{fc} \cdot t_{fc}}$		
$R_b := \begin{cases} 1.0 & \text{if } \frac{2 \cdot D_c}{t_w} \leq \lambda_{rw} \\ 1.0 - \left(\frac{a_{wc}}{1200 + 300 \cdot a_{wc}} \right) \cdot \left(\frac{2 \cdot D_c}{t_w} - \lambda_{rw} \right) & \text{otherwise} \end{cases}$		$R_b = 0.96$

Web is slender, so Eq. 6.10.3.2.1-3 (web bend-buckling) must be checked per AASHTO 6.10.3.2.1. For constructability checks, per AASHTO 6.10.3.2.1, use $R_b = 1$

Resistance of Girder G4, Field Section 1 for First Cast During Concrete Deck Pour, continued

Local Buckling Resistance (AASHTO 6.10.8.2.2):

Slenderness ratio of the compression flange

$$\lambda_f := \frac{b_{fc}}{2 \cdot t_{fc}} \quad \lambda_f = 10 \quad (\text{Eq. 6.10.8.2.2-3})$$

$$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E_s}{F_y}} \quad \lambda_{pf} = 9.2 \quad (\text{Eq. 6.10.8.2.2-4})$$

$$\lambda_{rf} := 0.56 \cdot \sqrt{\frac{E_s}{0.7F_y}} \quad \lambda_{rf} = 16.1 \quad (\text{Eq. 6.10.8.2.2-5})$$

$$F_{nc1} := \begin{cases} (R_b \cdot R_h \cdot F_y) & \text{if } \lambda_f \leq \lambda_{pf} \\ \left[1 - \left(1 - \frac{0.7F_y}{R_h \cdot F_y} \right) \cdot \frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right] \cdot R_b \cdot R_h \cdot F_y & \text{otherwise} \end{cases} \quad (\text{Eq. 6.10.8.2.2-1})$$

$$F_{nc1} = 48.2 \text{ ksi}$$

$$(\text{Eq. 6.10.8.2.2-2})$$

Lateral Torsional Buckling Resistance (AASHTO 6.10.8.2.3):

Unbraced length $L_b = 20.5$ ft with cross-frame spacing of approximately 20 ft

Effective Radius
of Gyration

$$r_t := \frac{b_{fc}}{\sqrt{12 \cdot \left(1 + \frac{1}{3} \cdot \frac{D_c \cdot t_w}{b_{fc} \cdot t_{fc}} \right)}} \quad r_t = 4.8 \text{ in.} \quad (\text{Eq. 6.10.8.2.3-9})$$

Limiting Unbraced Length Calculations:

$$L_p := r_t \cdot \sqrt{\frac{E_s}{F_y}} \quad L_p = 9.7 \text{ ft.} \quad (\text{Eq. 6.10.8.2.3-4})$$

$$L_r := \pi r_t \cdot \sqrt{\frac{E_s}{0.7F_y}} \quad L_r = 36.2 \text{ ft.} \quad (\text{Eq. 6.10.8.2.3-5})$$

Moment gradient modifier

$$C_b := 1 \text{ since } \frac{f_{mid}}{f_2} > 1$$

Resistance of Girder G4, Field Section 1 for First Cast During Concrete Deck Pour, continued

Lateral Torsional Buckling Resistance

$$F_{nc2} := \begin{cases} (R_b \cdot R_h \cdot F_y) & \text{if } L_b \leq L_p \\ \left[C_b \cdot \left[1 - \left(1 - \frac{0.7F_y}{R_h \cdot F_y} \right) \cdot \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \cdot R_b \cdot R_h \cdot F_y \right] & \text{if } L_p < L_b \leq L_r \\ \frac{C_b \cdot R_b \cdot \pi^2 \cdot E_s}{\left(\frac{L_b}{r_t} \right)^2} & \text{otherwise} \end{cases} \quad (\text{Eq. 6.10.8.2.3-1})$$

$$F_{nc2} = 43.9 \text{ ksi}$$

(Eq. 6.10.8.2.3-2)

(Eq. 6.10.8.2.3-3)

Controlling Nominal Flexural Resistance:

$$F_{nc} := \min(F_{nc1}, F_{nc2}) \quad F_{nc} = 43.9 \text{ ksi}$$

$$\text{Resistance factor} \quad \Phi_f := 1.0 \quad (\text{AASHTO 6.5.4.2})$$

$$\Phi_f \cdot F_{nc} = 43.9 \text{ ksi}$$

$$M_{rx} := \Phi_f \cdot F_{nc} \cdot S_{tx} \quad M_{rx} = 9059.3 \text{ kip ft}$$

$$\frac{M_{ux}}{M_{rx}} = 0.6 \quad \text{Adequate resistance for lateral-torsional buckling}$$

Determine Stress due to Lateral Bending:

$$\text{First-order lateral bending stress (from previous):} \quad f_{L1c} = 10.6 \text{ ksi}$$

Limiting unbraced length for first-order lateral bending stress

$$1.2 \cdot L_p \cdot \sqrt{\frac{C_b \cdot R_b}{\frac{f_{buc}}{F_y}}} = 15.9 \text{ ft} \quad (\text{Eq. 6.10.1.6-2})$$

Lateral bending stress:

elastic lateral torsional buckling stress

$$F_{cr} := \frac{C_b \cdot R_b \cdot \pi^2 \cdot E_s}{\left(\frac{L_b}{r_t}\right)^2}$$

$$F_{cr} = 109.4 \text{ ksi}$$

(Eq. 6.10.8.2.3-8)

Resistance of Girder G4, Field Section 1 for First Cast During Concrete Deck Pour, continued

Check if the first-order stress needs to be amplified: lateral Approximated second-order bending stress:

$$f_{Lc} := \begin{cases} f_{L1c} & \text{if } L_b \leq 1.2 \cdot L_p \cdot \sqrt{\frac{C_b \cdot R_b}{\frac{f_{buc}}{F_y}}} \\ \left(\frac{0.85}{1 - \frac{f_{buc}}{F_{cr}}} \right) \cdot f_{L1c} & \text{otherwise} \end{cases}$$

$$f_{Lc} = 11.9 \text{ ksi}$$

(Eq. 6.10.1.6-4)

Lateral bending check:

$$\text{Lateral_Bending_Resistance} := \begin{cases} \text{“Lateral Bending Requirements Satisfied”} & \text{if } f_{Lc} \leq 0.6F_y \\ \text{“Girder is NOT Adequate in Lateral Flexure”} & \text{otherwise} \end{cases}$$

$$\text{Lateral_Bending_Resistance} = \text{“Lateral Bending Requirements Satisfied”}$$

Overall Flexural Resistance Check:

The following must be satisfied:

$$f_{buc} + \frac{1}{3} \cdot f_{Lc} \leq \phi_f \cdot F_{nc} \quad (\text{Eq. 6.10.3.2.1-2})$$

$$f_{buc} + \frac{1}{3} \cdot f_{Lc} = 30.4 \text{ ksi} \quad \phi_f \cdot F_{nc} = 43.9 \text{ ksi}$$

$$\text{Resistance_Check} := \begin{cases} \text{“Girder Meets AASHTO Flexural Requirements”} & \text{if } f_{buc} + \frac{1}{3} \cdot f_{Lc} \leq \phi_f \cdot F_{nc} \\ \text{“Girder is NOT Adequate in Lateral Flexure”} & \text{otherwise} \end{cases}$$

$$\text{Resistance_Check} = \text{“Girder Meets AASHTO Flexural Requirements”}$$

Resistance of Girder G4, Field Section 1 for First Cast During Concrete Deck Pour, continued

Constructability Check:

The following must be satisfied:

The following must be satisfied:

$$\begin{aligned}
 f_{\text{buc}} + f_{\text{Lc}} &\leq \Phi_f \cdot R_h \cdot F_y && \text{(Eq. 6.10.3.2.1-1)} \\
 f_{\text{buc}} + f_{\text{Lc}} &= 38.4 \text{ ksi} < && \Phi_f \cdot R_h \cdot F_y = 50 \text{ ksi} \\
 &\text{Adequate resistance in compression flange}
 \end{aligned}$$

The following must be satisfied:

$$\begin{aligned}
 f_{\text{but}} + f_{\text{Lt}} &\leq \Phi_f \cdot R_h \cdot F_y && \text{(Eq. 6.10.3.2.2-1)} \\
 f_{\text{but}} + f_{\text{Lt}} &= 27.6 \text{ ksi} < && \Phi_f \cdot R_h \cdot F_y = 50 \text{ ksi} \\
 &\text{Adequate resistance in tension flange}
 \end{aligned}$$

Web Bend-Buckling Resistance without Longitudinal Stiffeners (AASHTO 6.10.1.9.1):

$$k := \frac{9}{\left(\frac{D_c}{D}\right)^2} = 28.71 \quad \text{(Eq. 6.10.1.9.1-2)}$$

$$F_{\text{crw}} := \frac{0.9 \cdot E_s \cdot k}{\left(\frac{D}{t_w}\right)^2} = 33.6 \text{ ksi} \quad \text{(Eq. 6.10.1.9.1-1)}$$

$$F_{\text{crw}} := \min\left(F_{\text{crw}}, R_h \cdot F_y, \frac{F_y}{0.7}\right) = 33.6 \text{ ksi}$$

Constructability Check:

$$F_{\text{buc}} \leq \Phi_f \cdot F_{\text{crw}} \quad \text{(Eq. 6.10.3.2.1-3)}$$

$$F_{\text{buc}} = 26.5 \text{ ksi} < \Phi_f \cdot F_{\text{crw}} = 33.6 \text{ ksi}$$

Adequate resistance in web

Adequacy of the Cross-Frame Bracing Member for Deck Pour Loading

The cross-frames provide stability of the girder flanges during erection and placement of the deck. Deck pour loads often control the cross-frame design. For this example a cross-frame member will be examined for maximum loading during the deck pour. L8x6x9/16 Single Angle Cross-frame Member.

Axial Tension Resistance of the Brace L8x6x9/16 (AASHTO 6.8.2):

$P_{ut} := 50.5$ kip Maximum tension force in any brace member from UT Bridge analysis model.

Material / Section Properties (Taken from AISC manual) $F_y := 36$ ksi $F_u := 58$ ksi $A_g := 7.56$ in²

Note that A36 is still the preferred material specification for angles. Per AISC, availability of angles in other grades should be confirmed prior to their specification.

From Figure B2-1 it appears that the brace is welded rather than bolted to the conn. plate, so $A_n := A_g$

For shear lag reduction factor, lacking weld details, assume $U := 0.60$ (0.5 is worst case per 6.8.2.2)

Resistance factors $\Phi_y := 0.95$ $\Phi_u := 0.80$ (AASHTO 6.5.4.2)

Tension resistance for yielding $P_{ry} := \Phi_y \cdot F_y \cdot A_g = 258.6$ kip (Eq. 6.8.2.1-1)

Tension resistance for fracture $P_{ru} := \Phi_u \cdot F_u \cdot A_n \cdot U = 210.5$ kip (Eq. 6.8.2.1-2)

Axial Resistance Check $P_{ut} 50.5$ kip $< P_{rt} := \min(P_{ry}, P_{ru}) = 210.5$ kip Tension resistance is adequate

Axial Compression Resistance of the Brace L8x6x9/16 (AASHTO 6.9.4):

$P_{uc} := 18.2$ kip Maximum compression force in any brace member from UT Bridge analysis model.

Material / Section Properties (Taken from AISC manual)

$E_s := 29000$ ksi $F_y := 36$ ksi $A_g := 7.56$ in²

Check Slenderness of the Member (Sect. 6.9.4.2)

Following requirement needs to be satisfied for the element to qualify as nonslender:

$$\frac{b}{t} \leq k \cdot \sqrt{\frac{E}{F_y}} \quad (\text{Eq. 6.9.4.2.1-1}) \quad k := 0.45 \quad \text{from Table 6.9.4.2.1-1}$$

angle leg:

$$b := 8 \text{ in}$$

$$t := 0.8625 \text{ in}$$

$$\frac{b}{t} = 14.2$$

$$k \cdot \sqrt{\frac{E_s}{F_y}} = 12.8$$

leg check:

Find Q if element is slender:

$$Q_s := \begin{cases} 1.34 - 0.76 \left(\frac{b}{t} \right) \cdot \sqrt{\frac{F_y}{E_s}} & \text{if } \frac{b}{t} \leq 0.91 \cdot \sqrt{\frac{E_s}{F_y}} \\ 0.53 \cdot E_s & \text{otherwise} \\ F_y \cdot \left(\frac{b}{t} \right)^2 & \end{cases} \quad (\text{Eq. 6.9.4.2.2-5})$$

(Eq. 6.9.4.2.2-6)

Adequacy of the Cross-Frame Bracing Member for Deck Pour Loading, continued

$$Q_s := \begin{cases} 1 & \\ 1.0 & \text{if } \frac{b}{t} \leq 0.45 \cdot \sqrt{\frac{E_s}{F_y}} \\ Q_s & \text{otherwise} \end{cases}$$

$$Q = 1$$

Determine Effective Slenderness Ratio $(KL/r)_{\text{eff}} = \lambda_{\text{eff}}$ (Sect. 6.9.4.4)

Recall $S := 11 \text{ ft}$ $L := \max\left(S, \frac{\sqrt{S^2 + D^2}}{2}\right)$ S governs the brace length, so $L = 132 \text{ in}$

$$r_x := 2.55 \text{ in so } \frac{L}{r_x} = 51.8$$

$$\lambda_{\text{eff}} := \begin{cases} \left(72 + 0.75 \frac{L}{r_x}\right) & \text{if } \frac{L}{r_x} \leq 80 \\ \left(32 + 1.25 \frac{L}{r_x}\right) & \text{otherwise} \end{cases}$$

$$\lambda_{\text{eff}} = 110.8$$

(Eq. 6.9.4.4-1)

(Eq. 6.9.4.4-2)

Limiting KL/r for secondary compression members $\lambda_{\text{limit}} := 140$ (Sect. 6.9.3)

Maximum actual slenderness corresponds to minor principal axis buckling $r_z := 1.30 \text{ in}$ $K := 1$
 $\frac{K \cdot L}{r_z} = 101.5 < \lambda_{\text{limit}} = 140$ Therefore, actual maximum slenderness ratio is adequate

Flexural Buckling Resistance

$$P_e := \frac{\pi^2 \cdot E_s}{(\lambda_{\text{eff}})^2} \cdot A_g \quad P_e = 176.2 \text{ kip} \quad (\text{Eq. 6.9.4.1.2-1})$$

Since the various conditions for single-angle members are satisfied as enumerated in AASHTO LRFD Sect. 6.9.4.4, the effective slenderness ratio can be calculated per that section; therefore, only flexural buckling resistance will be used to determine nominal compressive resistance of the brace. The effect of the eccentricities can be neglected when evaluated in this manner. Equivalent Nominal Yield Resistance

$$P_o := Q \cdot F_y \cdot A_g \quad P_o = 261 \text{ kip} \quad (\text{Sect. 6.9.4.1.1})$$

$$\frac{P_e}{P_o} = 0.7$$

Nominal Compressive Resistance

$$P_n := \begin{cases} \left[0.658^{\left(\frac{P_o}{P_e}\right)} \cdot P_o\right] & \text{if } \frac{P_e}{P_o} \geq 0.44 \\ (0.877 P_e) & \text{otherwise} \end{cases} \quad (\text{Eq. 6.9.4.1.1-1})$$

$$P_n = 140.4 \text{ kip}$$

(Eq. 6.9.4.1.1-2)

Adequacy of the Cross-Frame Bracing Member for Deck Pour Loading, continuedResistance factor $\Phi_c := 0.9$ (AASHTO 6.5.4.2)

Factored Axial Resistance

$$P_{rc} := \Phi_c \cdot P_n \quad P_{rc} = 126.4 \text{ kip} \quad (\text{Eq. 6.9.2.1-1})$$

Axial Resistance Check

$$P_{uc} \ 18.2 \text{ kip} < P_{rc} = 126.4 \text{ kip} \quad \text{Compression resistance is adequate}$$

Verify Bracing Strength to Provide Girder StabilityUnbraced length $L_b := 20 \text{ ft} = 240 \text{ in}$ with cross-frame spacing of 20 ftSpan length $L := 160 \text{ ft} = 1920 \text{ in}$ Maximum moment with span $M_f := 5468 \text{ kip ft} = 65616 \text{ kip in}$ Height of cross-frame $h_b := D = 84 \text{ in}$ Number of braces in span, excluding supports $n := 7$ Modulus of elasticity $E_s = 29000 \text{ ksi}$ Moment modification factor $C_b := 1$ Distance between flange centroids $h_o := d - \frac{t_{ff}}{2} - \frac{t_{bf}}{2} = 85.2 \text{ in}$ Recall girder cross-sectional properties $Y_b = 38.47 \text{ in}$ $Y_t = 48.03 \text{ in}$ $t_{ff} = 1 \text{ in}$ $t_{bf} = 1.5 \text{ in}$

Calculate effective minor axis moment of inertia

$$t := Y_b = 38.47 \text{ in} \quad c := Y_t = 48.03 \text{ in} \quad I_{yc} := \frac{t_{ff} \cdot b_{ff}^3}{12} = 666.7 \text{ in}^4 \quad I_{yt} := \frac{t_{bf} \cdot b_{bf}^3}{12} = 1157.6 \text{ in}^4$$

$$I_{eff} := I_{yc} + \left(\frac{t}{c}\right) \cdot I_{yt} = 1593.9 \text{ in}^4 \quad (\text{Manual Eq. D-5.2j})$$

$$\text{Recall} \quad I_y = 1825.5 \text{ in}^4 \quad \text{Therefore} \quad \frac{I_{eff}}{I_y} = 0.873$$

$$\text{Required strength} \quad F_{br} := \frac{0.005 L_b \cdot L \cdot M_f^2}{h_b \cdot n \cdot E_s \cdot I_{eff} C_b^2 \cdot h_o} = 4.28 \text{ kip} \quad (\text{Manual Eq. D-2.1a})$$

Length of diagonal member

$$L_c := \sqrt{(D^2 + S^2)} = 156.46 \text{ in}$$

Required strength of compression brace $F_{brc} := F_{br} = 4.28 \text{ kip}$ (Manual Fig. 5-7, Tension System)Required strength of tension brace $F_{brt} := \frac{2F_{br} \cdot L_c}{S} = 10.15 \text{ kip}$ (Manual Fig 5-7, Tension System)Available strength of tension brace to resist stability force $P_{rt} - P_{ut} = 159.97 > F_{brt} = 10.15 \text{ kip}$

Available strength of compression brace to resist stability force

$$P_{rc} - P_{uc} = 108.16 \text{ kip} > F_{brc} = 4.28 \text{ kip}$$

Therefore, bracing is adequate for strength

Adequacy of the Cross-Frame Bracing Member for Deck Pour Loading, continued**Check Required Stiffness of Bracing System $\beta_{T,reqd}$** Resistance factor $\Phi_{br} := 0.75$ Required stiffness $\beta_{T,reqd} := \frac{2.4 \cdot L \cdot M_f^2}{\Phi_{br} \cdot n \cdot E_s \cdot I_{eff} \cdot C_b^2} = 81754.2 \text{ kip in}$ (Manual Eq. D5.2b)**Calculate Attached Brace Stiffness β_b** Area of diagonal member $A_c := A_g = 7.56 \text{ in}^2$ Area of horizontal member $A_h := A_g = 7.56 \text{ in}^2$

Attached brace stiffness (Manual Fig. 5-7, Eq. for Tension System)

$$\beta_b := \frac{E_s \cdot S^2 \cdot h_b^2}{\left(\frac{2 \cdot L_c^3}{A_c}\right) + \frac{S^3}{A_h}} = 2706164.4 \text{ kip in}$$

Calculate Web Distortional Stiffness β_{sec} Recall web thickness $t_w = 0.5625 \text{ in}$ Intermediate stiffener plate thickness $t_s := 0.625 \text{ in}$ Intermediate stiffener plate width $b_s := 6.75 \text{ in}$ Web distortional stiffness $\beta_{sec} := 3.3 \cdot \frac{E_s}{h_o} \left[\frac{(1.5 \cdot h_o) \cdot t_w^3}{12} + \frac{t_s \cdot b_s^3}{12} \right] = 20110.6 \text{ kip in}$ (Manual Eq. D-5.2h)However, for a full depth cross-frame, $\beta_{sec} := 999999999999 \text{ kip} \cdot \text{in}$ (infinity)**Calculate In-Plane Girder System Stiffness β_g** Number of girders $n_g := 4$ Recall major axis moment of inertia $I_x = 118986.2 \text{ in}^4$ Girder system stiffness $\beta_g := \frac{24 \cdot (n_g - 1)^2}{n_g} \frac{S^2 \cdot E_s \cdot I_x}{L^3} = 458704 \text{ kip in}$ (Manual Eq. D-5.2i)**Calculate Total Provided System Stiffness β_T and Compare with Required System Stiffness $\beta_{T,reqd}$** Total provided system stiffness $\beta_T := \frac{1}{\left(\frac{1}{\beta_b} + \frac{1}{\beta_{sec}} + \frac{1}{\beta_g}\right)} = 392221.1 \text{ kip} \cdot \text{in}$ (Rearranged Manual Eq. D-5.2g) $\beta_T = 392221.1 \text{ kip in} > \beta_{T,reqd} = 81754.2 \text{ kip in}$ Therefore, stiffness of bracing system is adequate

Adequacy of Splice During Erection with 50% Bolt Installed

It is standard industry practice for the steel erectors to install 50% of the bolts in splice connections while erection progresses to ease fit-up complications. Once the steel is fully erected, the splice connections are completed prior to the deck pour. This example will examine one stage of the erection sequence for forces on a splice with 50% of the bolts installed. The bolts will be analyzed as bearing bolts since the bolts have not yet be torqued. Figures B2-54 and B2-55 show the details of the top flange and web splice bolt configurations. The bottom flange and web will not be examined in this example for brevity, but will be solved similarly as shown for the top flange.

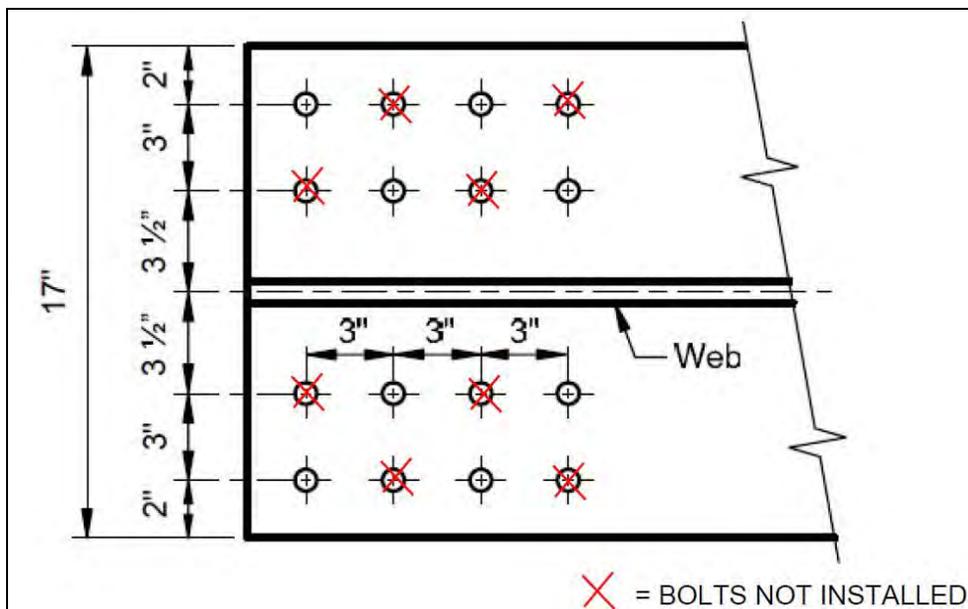


Figure B2-54: Top Flange Splice Bolt Configuration

Adequacy of Splice During Erection with 50% Bolt Installed, continued

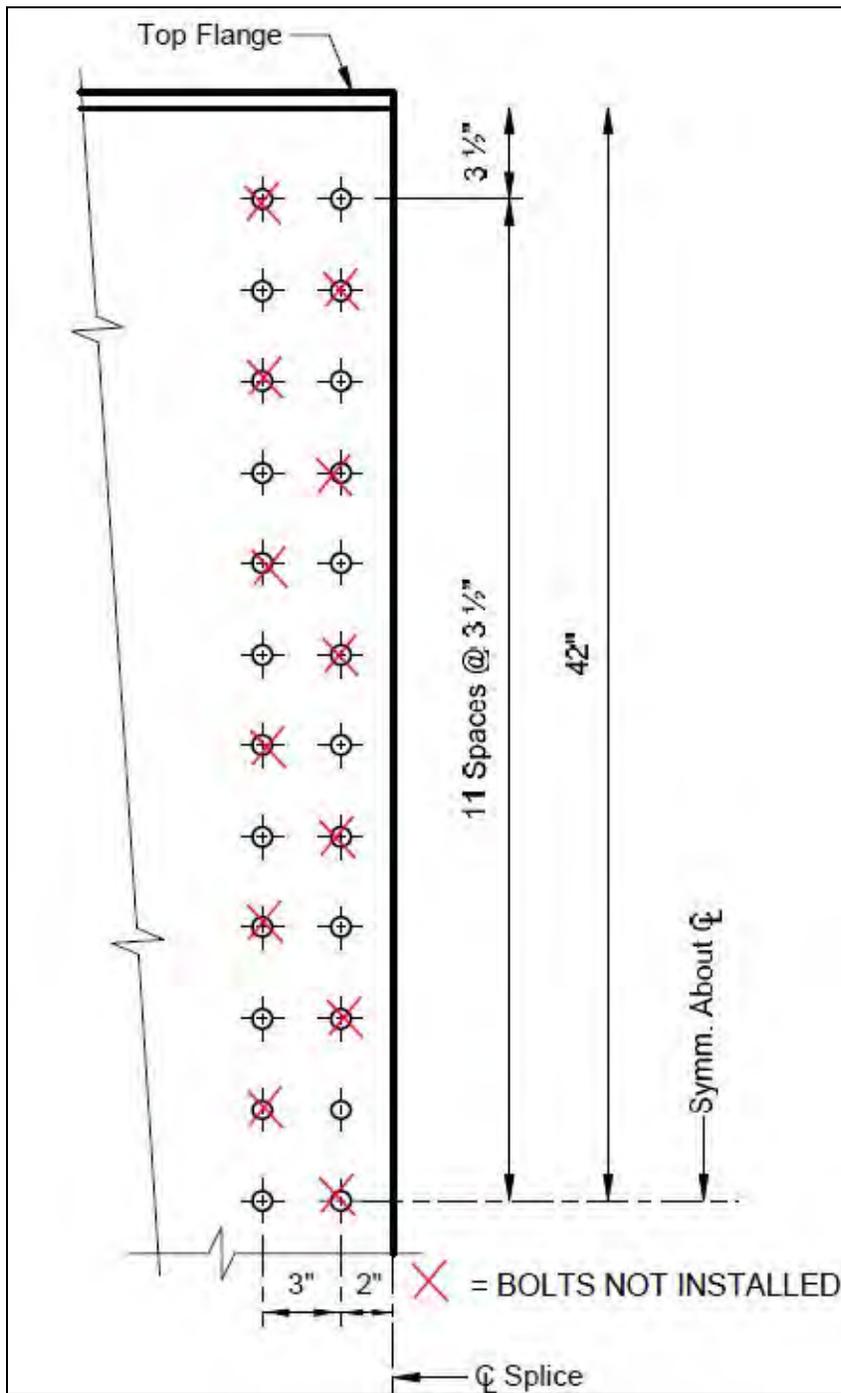


Figure B2-55: Web Splice Bolt Configuration

Adequacy of Splice During Erection with 50% Bolt Installed continued**Splice Forces**

Since UT Bridge does not give moments directly as output, the design forces must be back-calculated using the flange tip stresses. For this example, the splices in Span 2 for Girder G4 are checked (with the steel fully erected but prior to the deck pour).

$$x := \frac{(210 \text{ ft} - 133.65 \text{ ft})}{2} = 38.175 \text{ ft} \quad \text{Approximate Location of Splice in Span 2}$$

$$\text{Thus} \quad \frac{x}{210 \text{ ft}} = 0.18 \quad \text{of span length } L$$

$$b_f := 17 \text{ in Top flange width and } t_f := 1 \text{ in} \quad \text{Top flange thickness for G4 Field Section 3}$$

$$S_f := t_f \frac{b_f^2}{6} = 48.2 \text{ in}^3 \quad \text{Lateral Bending Section Modulus at Top Flange}$$

$$A_f := t_f \cdot b_f = 17 \text{ in}^2 \quad \text{Cross-sectional Area of Top Flange}$$

$$\sigma_{L1} := 6.55 \text{ ksi} \quad \text{from UT Bridge, Left Tip Stress at Top Flange at } 0.2L \text{ into Span 2}$$

$$\sigma_{R1} := -2.66 \text{ ksi} \quad \text{from UT Bridge, Right Tip Stress at Top Flange at } 0.2L \text{ into Span 2}$$

$$\sigma_{L2} := 2.51 \text{ ksi} \quad \text{from UT Bridge, Left Tip Stress at Top Flange at } 0.8L \text{ into Span 2}$$

$$\sigma_{R2} := 1.25 \text{ ksi} \quad \text{from UT Bridge, Right Tip Stress at Top Flange at } 0.8L \text{ into Span 2}$$

$$\sigma_{\text{diff}} := \frac{\max(\sigma_{L1} - \sigma_{R1}, \sigma_{L2} - \sigma_{R2})}{2} = 4.6 \text{ ksi} \quad \text{Lateral Bending (Warping) Stress in Top Flange}$$

$$\sigma_{\text{sum}} := \frac{\max(\sigma_{L1} + \sigma_{R1}, \sigma_{L2} + \sigma_{R2})}{2} = 1.9 \text{ ksi} \quad \text{Average Vertical Bending Stress in Top Flange}$$

$$M_{fl} := S_f \cdot \sigma_{\text{diff}} = 18.5 \text{ kip ft} \quad \text{Factored Lateral Flange Moment at Splice}$$

$$f_{\text{top}} := A_f \cdot \sigma_{\text{sum}} = 33.1 \text{ kip} \quad \text{Factored Flange Force from Strong Axis Moment at Splice}$$

Bolt Shear Resistance

$$7/8" \text{ dia. A325 Bolts} \quad A_b := 0.60 \text{ in}^2 \quad F_{ub} := 120 \text{ ksi} \quad N_s := 2 \text{ (Double Shear)}$$

$$N_{fb} := 8 \quad \text{No. Flange Bolts} \quad N_{wb} := 12 \quad \text{No. Web Bolts}$$

$$R_n := 0.48 \cdot A_b \cdot F_{ub} \cdot N_s \quad R_n = 69.1 \text{ kip} \quad (\text{Eq. 6.13.2.7-1})$$

$$\text{Resistance factor} \quad \Phi := 0.8 \text{ (6.5.4.2)}$$

$$R_r := \Phi \cdot R_n \quad R_r = 55.3 \text{ kip}$$

$$I_p := A_b \cdot [2 \cdot 2 \cdot (3.5^2 + 6.5^2) + 2 \cdot 2 \cdot (1.5^2 + 4.5^2)] \cdot \text{in}^2 = 184.8 \text{ in}^4$$

Adequacy of Splice During Erection with 50% Bolt Installed, continued

Longitudinal Force in Bolt from Vertical Bending Stress

$$F_{\text{longvert}} := \frac{f_{\text{top}}}{N_{\text{fb}}} = 4.1 \text{ kip}$$

Longitudinal Force in Critical Bolt from Flange Lateral Bending Stress

$$F_{\text{longlat}} := \frac{M_{\text{fl}} \cdot 6.5 \text{ in}}{\frac{I_{\text{p}}}{A_{\text{b}}}} = 4.7 \text{ kip}$$

Total Longitudinal Force in Critical Bolt

$$F_{\text{longtotal}} := F_{\text{longvert}} + F_{\text{longlat}} = 8.8 \text{ kip}$$

Transverse Force in Critical Bolt from Flange Lateral Bending Stress

$$F_{\text{trans}} := \frac{M_{\text{fl}} \cdot 4.5 \text{ in}}{\frac{I_{\text{p}}}{A_{\text{b}}}} = 3.2 \text{ kip}$$

Total Force in Critical Bolt

$$R_{\text{u}} := \sqrt{F_{\text{longtotal}}^2 + F_{\text{trans}}^2} = 9.4 \text{ kip}$$

$R_{\text{r}} = 55.3 \text{ kip} > R_{\text{u}} = 9.4 \text{ kip}$ Therefore, Flange Bolts are Adequate with 50% Installed.

Erection of G3 and G4 in Span 3 (with all cross-frames and holding crane supporting top flange)

Recall that temporary shoring or a hold crane is required to support the two girder system during erection of Span 3. Temporary shoring supports were added to the UT Bridge analysis model at the midspan of Span 3. These are now revised to hold crane supports at the girder top flange, as the contractor is interested in utilizing this option in lieu of shoring towers. See Figures B2-56 and B2-57 for output of the two girder system with a hold crane at the top flange.

The maximum lateral deflection of a girder in the system is 0.01 inches, which equates to $L/192000$. The Eigenvalue has increased to 36.1 indicating an adequate factor of safety against buckling. The maximum girders stresses are at 20.2 ksi. These stresses were found to be adequate per AASHTO constructability criteria but for brevity the calculations for girder resistance are only included in the deck pour analysis.

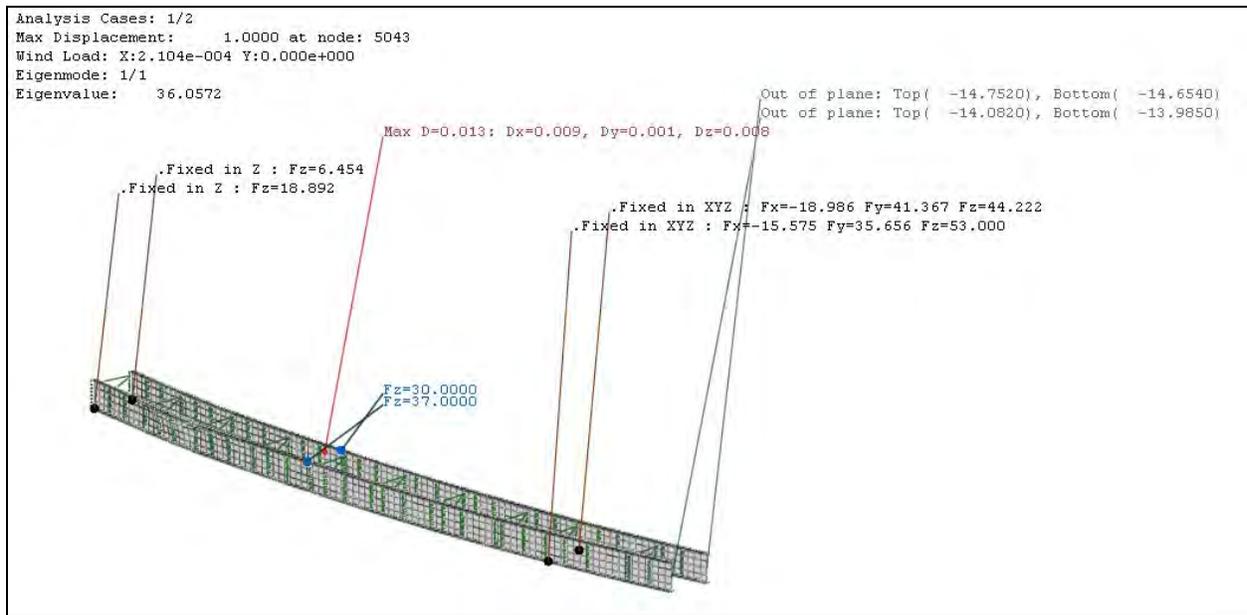


Figure B2-56: Girder Deflection During Two Girder Erection with Holding Crane at Top Flange

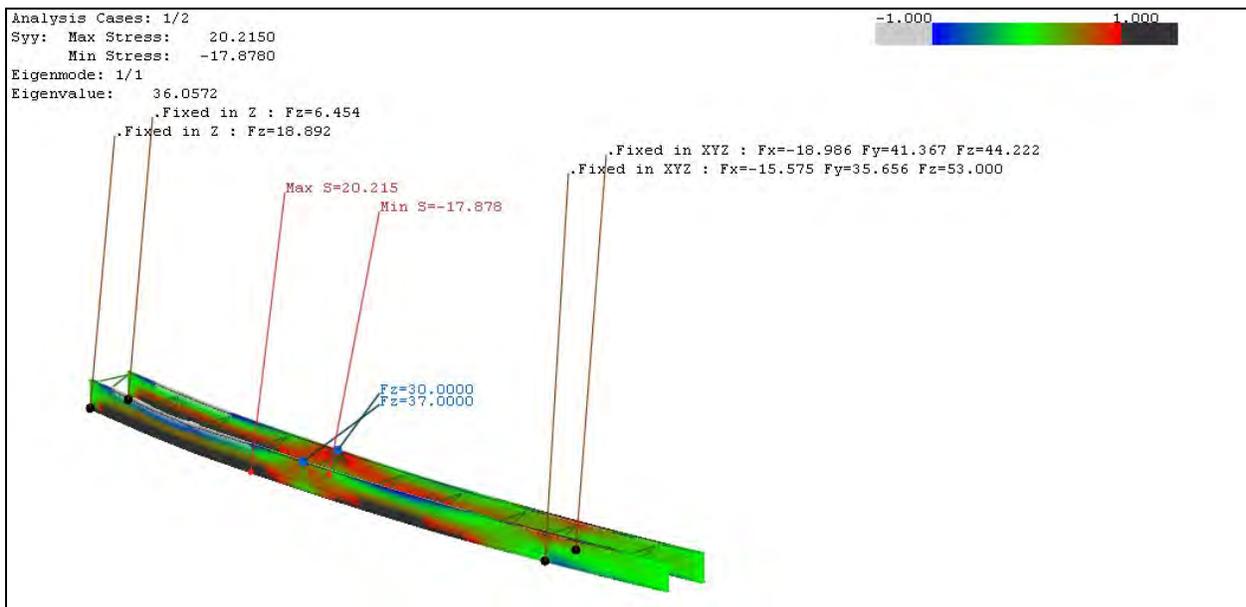


Figure B2-56: Girder Stresses and Eigenvalue Analysis, Two Girders with Holding Crane at Top Flange

Erection of G3 and G4 in Span 3 (with all cross-frames and holding crane supporting bottom flange)

Recall that temporary shoring or a hold crane is required to support the two girder system during erection of Span 3. Temporary shoring supports were added to the UT Bridge analysis model at the midspan of Span 3. These are now revised to hold crane supports at the girder bottom flange, as the contractor is interested in utilizing this option in lieu of shoring towers. See Figures B2-58 and B2-59 for output of the two girder system with a hold crane at the bottom flange.

The maximum lateral deflection of a girder in the system is 0.04 inches, which equates to $L/48000$. The Eigenvalue has decreased to 8.4 indicating the factor of safety against buckling is still adequate. The maximum girders stresses are at 20.2 ksi. These stresses were found to be adequate per AASHTO constructability criteria but for brevity the calculations for girder resistance are only included in the deck pour analysis.

Although the behavior of the two-girder system is still acceptable with the holding crane supporting the bottom flange, it is obvious that the top flange scenario is preferable (improved Eigenvalue and deflection).

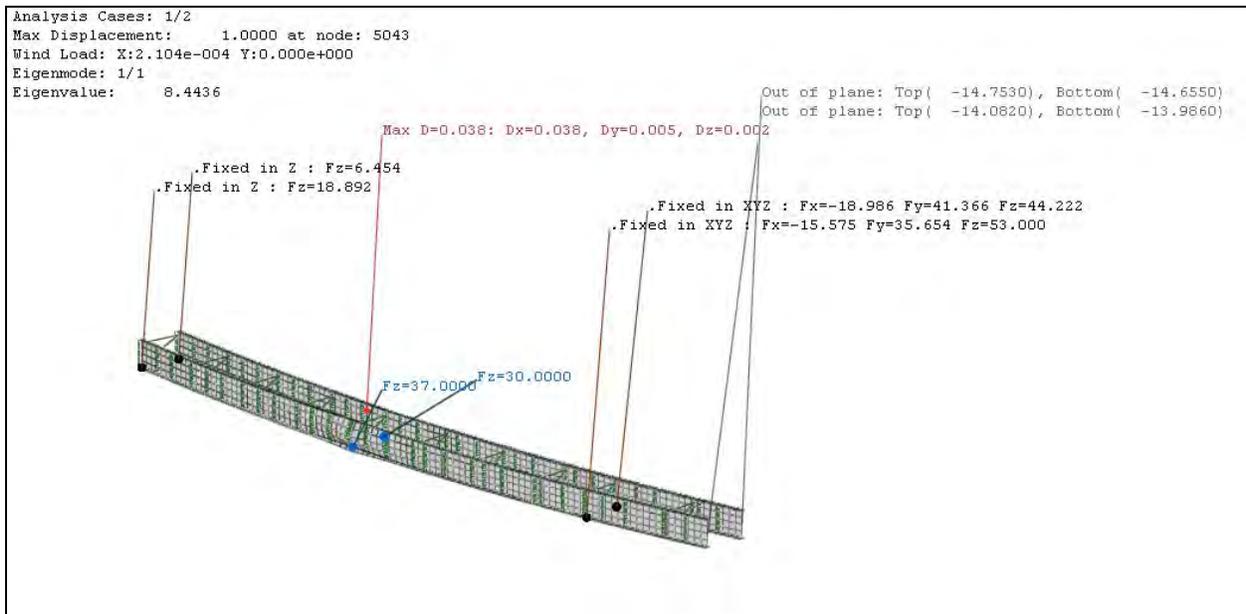


Figure B2-57: Girder Deflection During Two Girder Erection with Holding Crane at Bottom Flange

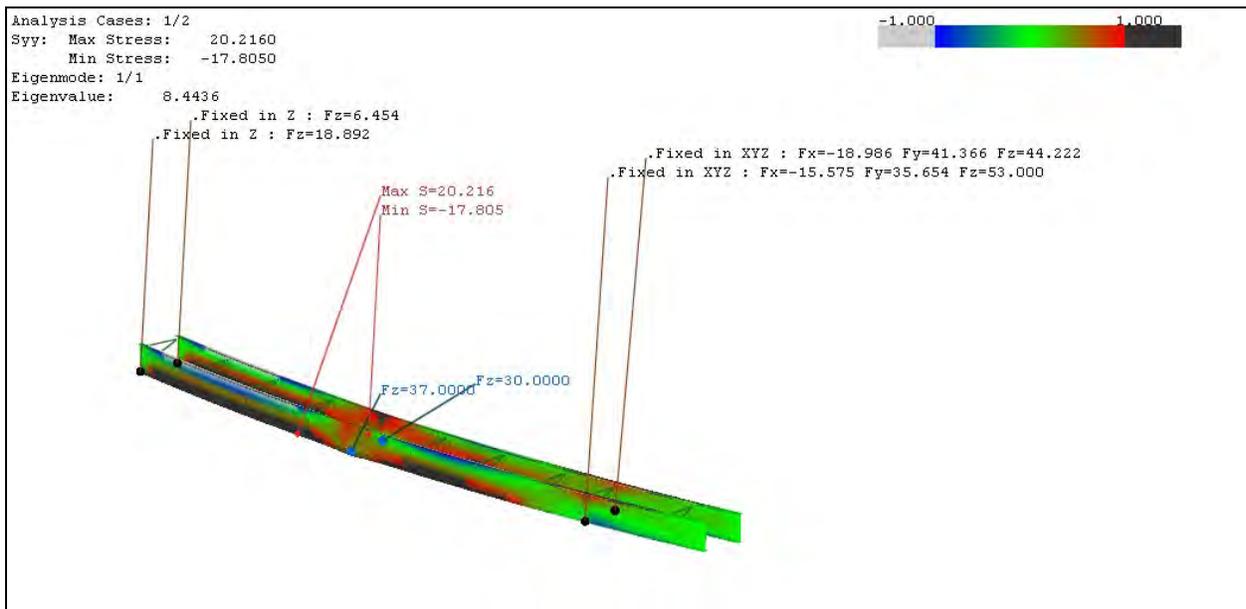
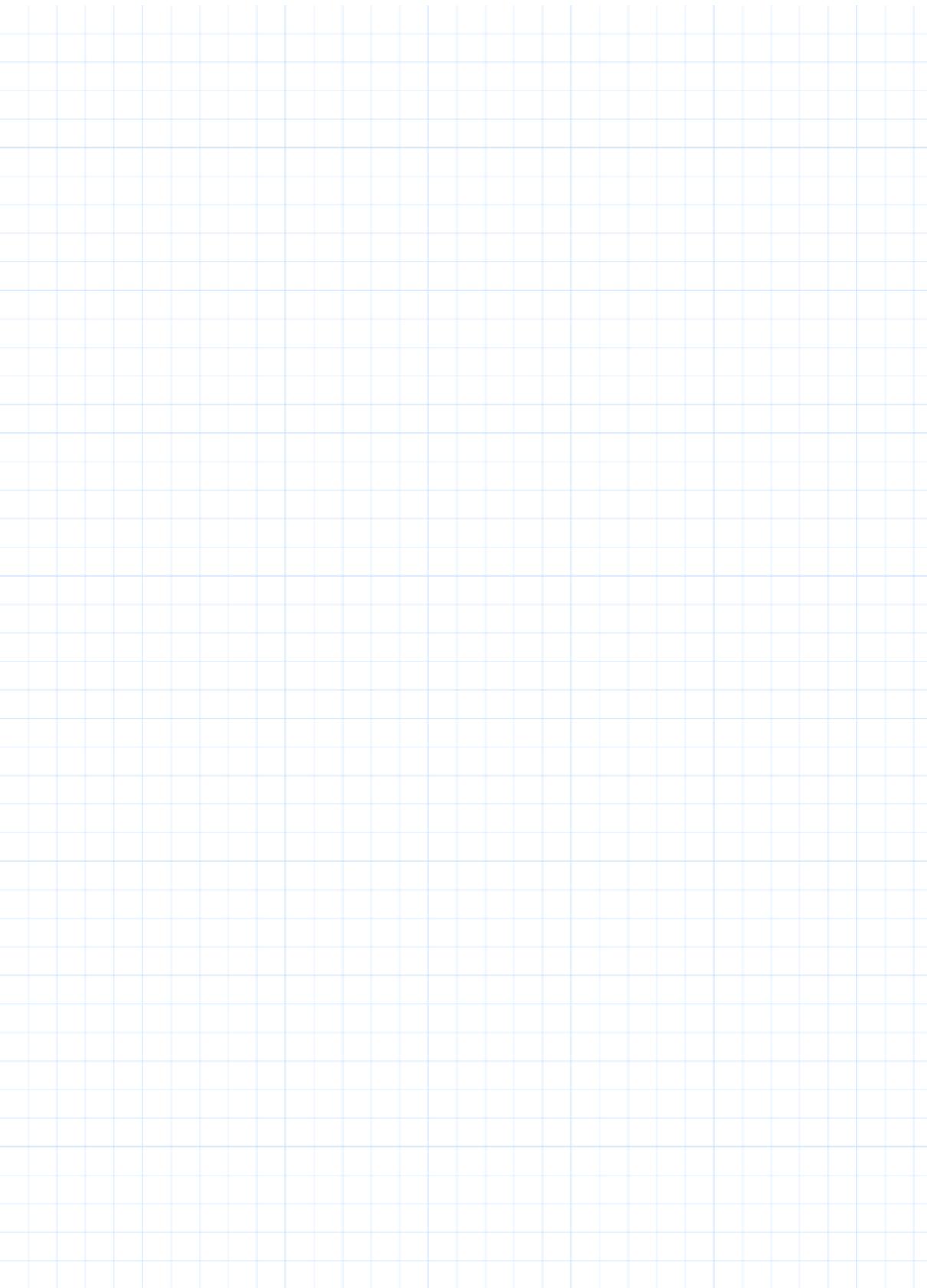


Figure B2-58: Girder Stresses and Eigenvalue Analysis, Two Girders with Holding Crane at Bottom Flange

Reader Notes



Reader
Notes



Example Problem #3: Concrete Spliced Girder

The following table of contents illustrates the general categories into which this example problem is subdivided, and the relevant page number at which the start of each category, and its specific component analysis or calculation tasks, may be found.

General Category / Description	Specific Analysis / Calculation Task	Appendix Page
Table of Contents	N/A	B.131
Bridge Data	N/A	B.132
Structural Analysis	Girder Stability During Lifting	B.132
Figure	Section Properties of FL 78" Bulb Tee w/ Variable Web	B.132
Figure	Example 3 Plan	B.133
Figure	Example 3 Elevation	B.134
Figure	Example 3 Sections for Constant Depth Girders	B.135
Figure	Example 3 Sections at Critical Locations	B.136
Figure	Example 3 Stages of Construction	B.137
Material Data	Concrete Properties	B.138
Drop-In Segment Girder	Check Drop-In Segment Girder for Lifting Stability	B.138
Pier Segment Girder	Check Pier Segment Girder for Lifting Stability	B.140
End Segment Girder	Wind Load	B.142
End Segment Girder	Check Girder End Segment for Beam Rollover	B.143
Strongback System	Design Hanger Beams Between Drop-In & Pier Segment	B.145
Strongback System	Design Hanger Bars Between Drop-In & Pier Segment	B.146
Strongback System	Design Strongback Between Drop-In & Pier Segment	B.148
Figure	Typical Strongback System	B.149
Figure	Minor Axis Moment of Inertia Calculation for BT-78	B.150
Figure	Minor Axis Moment of Inertia Calculation for BT-90	B.152
Figure	Minor Axis Moment of Inertia Calculation for BT-180	B.154
Figure	Prestress Layout for End Girder Segments	B.156
Figure	Prestress Layout for Girder Segments at Splices	B.156
Figure	Prestress Layout for Pier Girder Segments	B.157
Figure	Prestress Layout for Drop-In Girder Segments	B.158

Bridge Data:

Continuous 3 span concrete spliced girder bridge, 5 girders total. Out-to-out deck width is 47 ft. Girder spacing is 9.5 ft for a total of 38 ft between fascia girders. Structural deck slab thickness is 8.5 in. The spans are symmetric, with Spans 1 and 3 at 210 ft long and Span 2 at 280 ft long. Location is northern Florida, using PPC beams based on a 78" Florida Modified Bulb Tee with variable web depth.

Cast-in-place splices are 2 ft long at each end of the pier segments. The end span girder segments are 146 ft long, 78" bulb tees; the 90" bulb tee drop-in segment at Span 2 is 152 ft long. The 124 ft long girder segments at the piers are haunched, varying from a 78" bulb tee shape in the end spans to a depth of 180" over the piers to a 90" bulb tee shape in Span 2.

Girder Stability During Lifting:

The process of lifting the girders into place requires certain considerations. While being lifted the sweep of the girder causes the center of gravity to have some eccentricity. This causes a small amount of rotation that results in some of the self-weight being applied about the weak axis of the girder. The girders will be checked for rollover stability based on Chapter 7. The pier segment girder and drop-in segment girder will be checked.

The general plan and elevation, cross-sections, and construction stages are presented in the following figures.

The table immediately below presents cross-sectional properties for various PPC bulb tee girder shapes. The computation of the minor axis moments of inertia, not included in the table, may be found at the end of this example problem.

Section	Depth (in.)	Area (in ²)	Moment of Inertia (in ⁴)	Y _b (in.)	Y _t (in.)	S _b (in ³)	S _t (in ³)
BT-78	78	1,261	1,014,899	40.21	37.79	25,237	26,859
BT-84	84	1,315	1,223,135	43.19	40.81	28,317	29,974
BT-90	90	1,369	1,455,041	46.18	43.82	31,511	33,201
BT-96	96	1,423	1,711,587	49.16	46.84	34,818	36,540
BT-102	102	1,477	1,993,747	52.14	49.86	38,237	39,989
BT-108	108	1,531	2,302,492	55.13	52.87	41,767	43,548
BT-114	114	1,585	2,638,794	58.11	55.89	45,407	47,217
BT-120	120	1,639	3,003,626	61.10	58.90	49,159	50,996
BT-126	126	1,693	3,397,960	64.09	61.91	53,020	54,884
BT-132	132	1,747	3,822,766	67.08	64.92	56,990	58,882
BT-138	138	1,801	4,279,019	70.07	67.93	61,071	62,989
BT-144	144	1,855	4,767,689	73.06	70.94	65,260	67,204
BT-150	150	1,909	5,289,749	76.05	73.95	69,559	71,529
BT-156	156	1,963	5,846,170	79.04	76.96	73,966	75,962
BT-162	162	2,017	6,437,925	82.03	79.97	78,483	80,504
BT-168	168	2,071	7,065,986	85.02	82.98	83,108	85,155
BT-174	174	2,125	7,731,324	88.01	85.99	87,842	89,914
BT-180	180	2,179	8,434,913	91.01	88.99	92,684	94,782

Table B3-11 Section Properties of Florida 78" Bulb Tee with Variable Web Depth

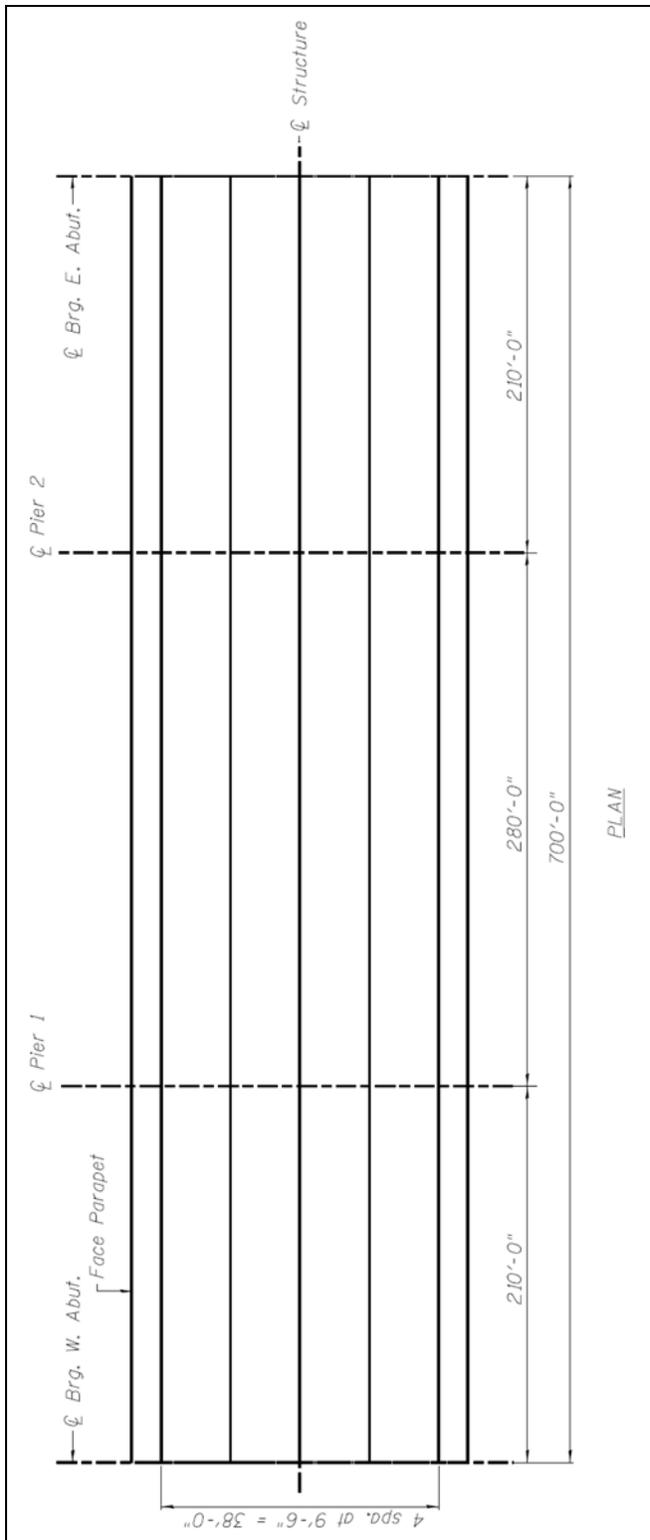


Figure B3-1: Example 3 Plan

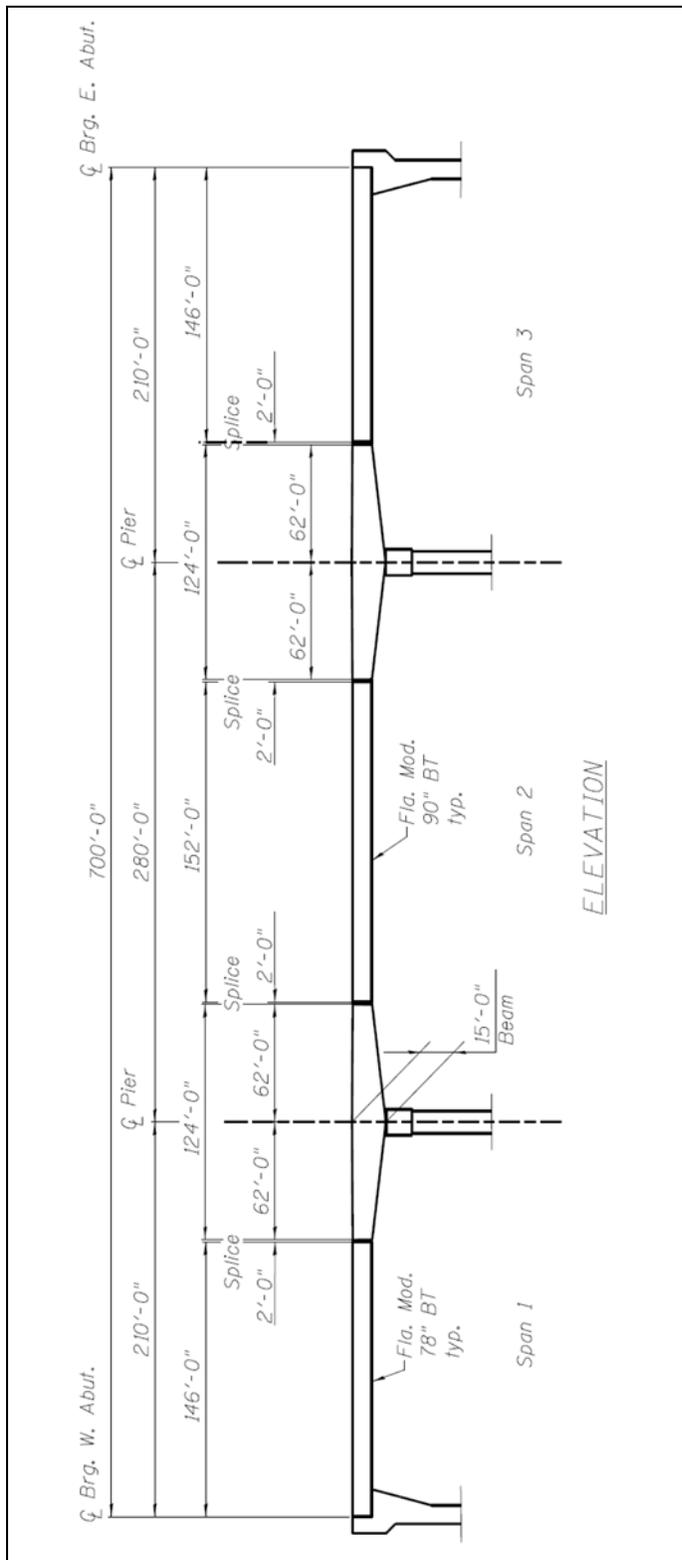


Figure B3-2: Example 3 Elevation

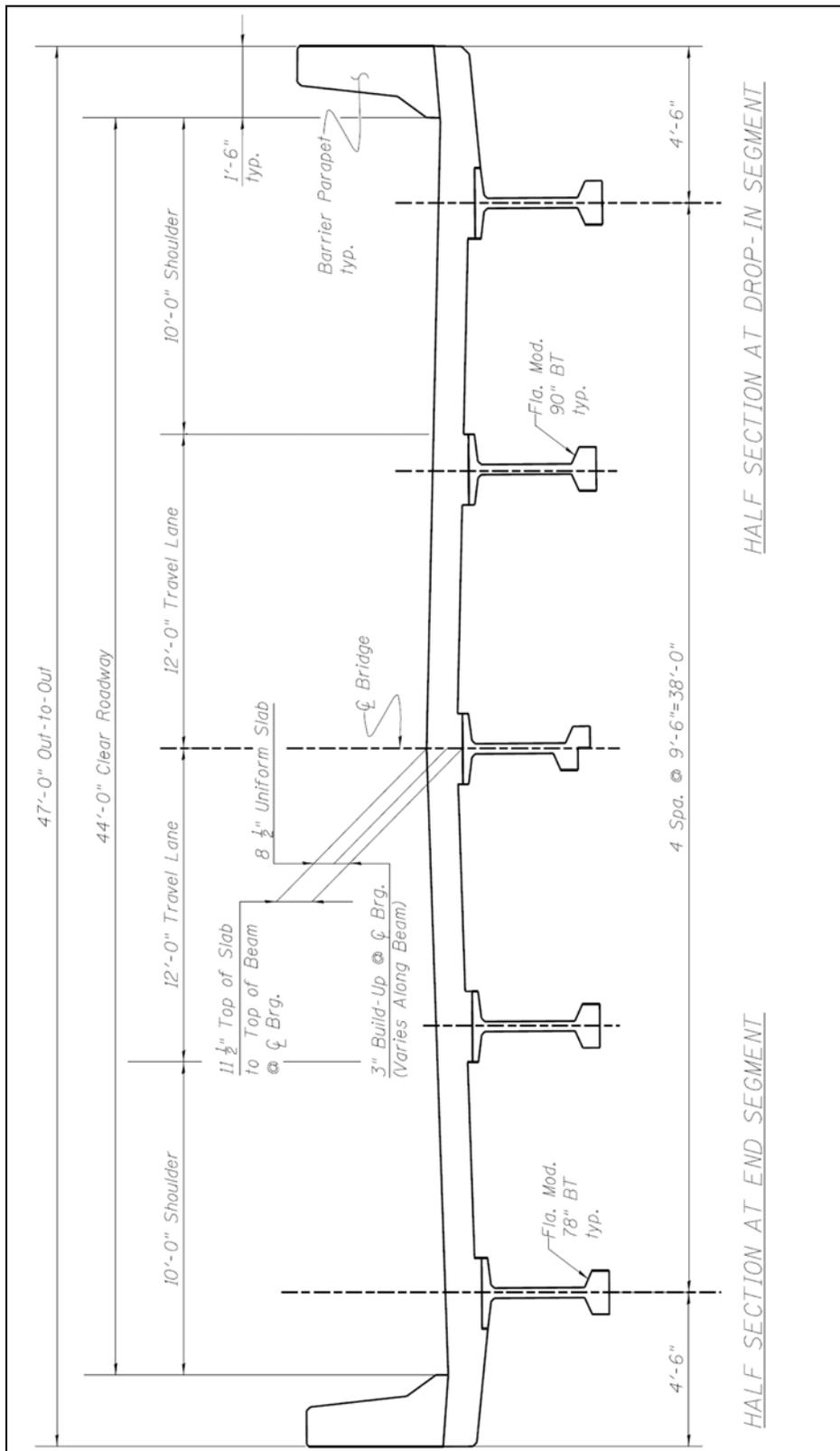


Figure B3-3: Example 3 Sections for Constant Depth Girders

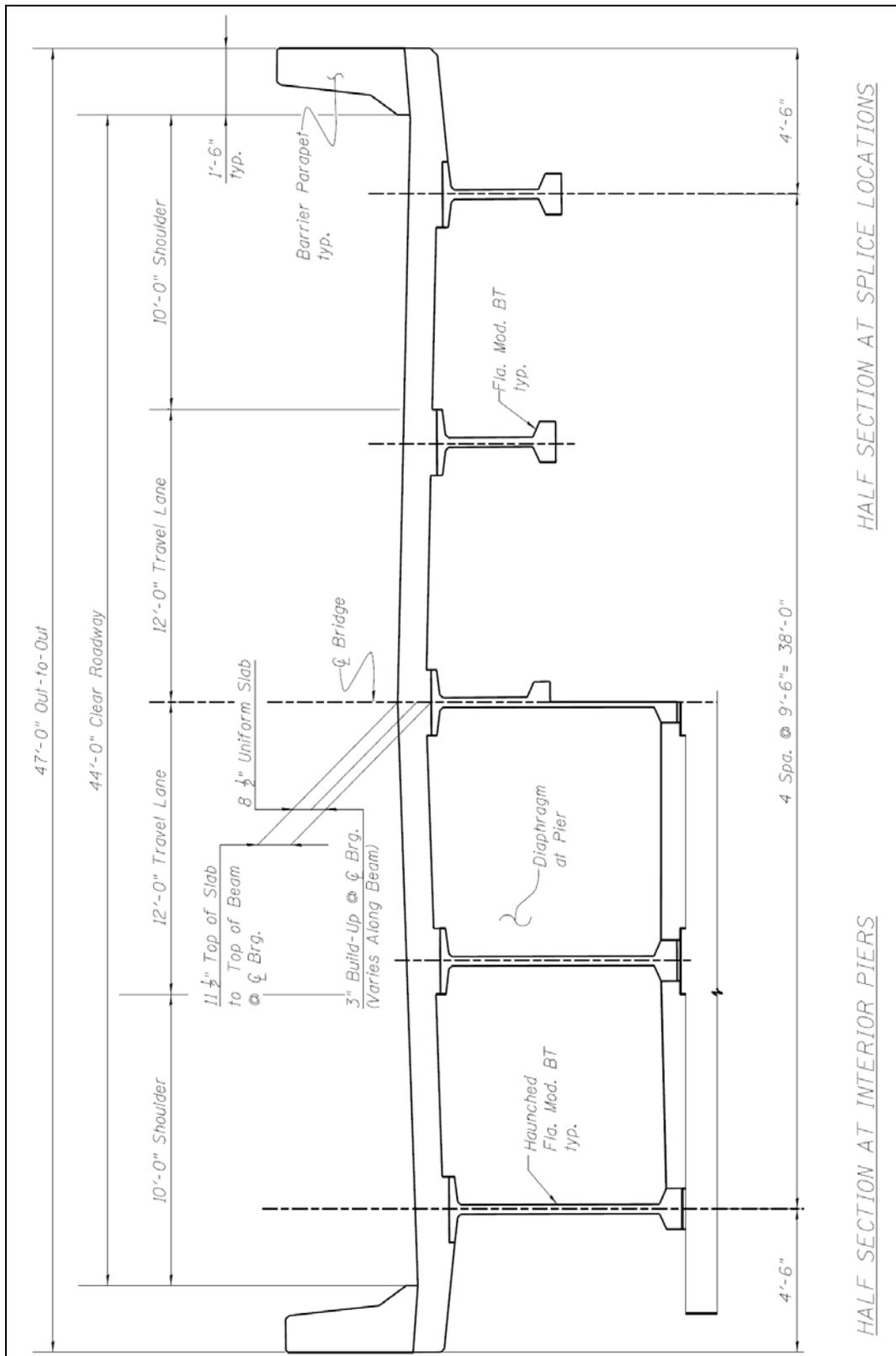


Figure B3-4: Example 3 Sections at Critical Locations

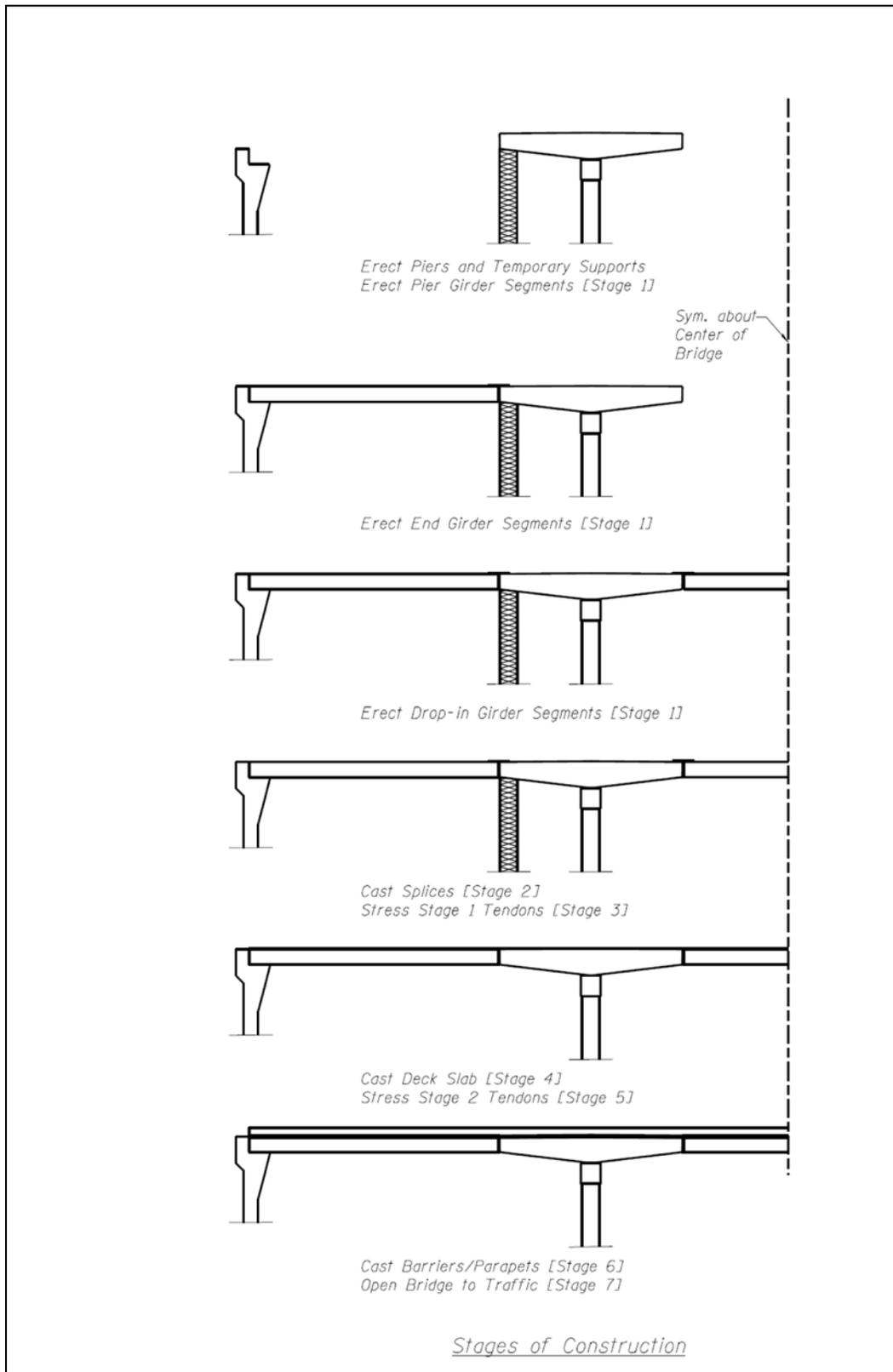


Figure B3-5: Example 3 Stages of Construction

Concrete Properties:

Strength of concrete (assume 28-day strength has already been reached)

$$f_c := 8500 \text{ psi}$$

Modulus of rupture

$$f_r := 0.24 \text{ ksi} \cdot \sqrt{\frac{f_c}{1000 \text{ psi}}} = 0.7 \text{ ksi} \quad (\text{AASHTO 5.4.2.6})$$

Concrete modulus of elasticity

$$E_c := 1820 \text{ ksi} \cdot \sqrt{\frac{f_c}{1000 \text{ psi}}} = 5306 \cdot \text{psi} \quad (\text{AASHTO 5.4.2.4-1})$$

Check Drop-In Segment Girder for Lifting Stability

Length between lifting points

$$L_{di} := 142 \text{ ft}$$

Girder properties (uses BT-90)

Gross area of concrete	$A_{g_di} := 1369 \text{ in}^2$
Girder length	$L_{g_di} := 152 \text{ ft}$
Minor axis moment of inertia	$I_{y_di} := 115887 \text{ in}^4$
Distance from centroid to bottom fiber	$y_{b_di} := 46.18 \text{ in}$
Distance from centroid to top fiber	$y_{t_di} := 43.82 \text{ in}$
Major axis moment of inertia	$I_{x_di} := 1455041 \text{ in}^4$
Width of top flange	$b_{t_di} := 62 \text{ in}$
Height of roll axis above centroid*	$y_{r_di} := y_{t_di} = 43.82 \text{ in}$

*Note: This should be modified to include camber, but ignored here for simplicity (< 1% difference)

Initial lateral eccentricity (based on 1/4 inch plus half of PCI sweep tolerance of 1/8" per 10 ft)

$$e_{i_di} := 0.25 \text{ in} + \left(\frac{1}{8} \text{ in}\right) \cdot \left(\frac{L_{g_di}}{10 \text{ ft}}\right) \left[\left(\frac{L_{di}}{L_{g_di}}\right)^2 - \frac{1}{3}\right] = 1.27 \text{ in}$$

Cantilever overhang during lift

$$a_{di} := \frac{(L_{g_di} - L_{di})}{2} = 5 \text{ ft}$$

Weight of beam per unit length

$$w_{di} := A_{g_di} \cdot 150 \text{ pcf} = 1.426 \frac{\text{kip}}{\text{ft}}$$

Lateral deflection of center of gravity of girder with full dead weight applied laterally

$$z_{o_di} := w_{di} \cdot \left(\frac{0.1 \cdot L_{di}^5 - a_{di}^2 \cdot L_{di}^3 + 3 \cdot a_{di}^4 \cdot L_{di} + 1.2 \cdot a_{di}^5}{12 \cdot E_c \cdot I_{y_di} \cdot L_{g_di}}\right) = 12.53 \text{ in}$$

Assuming straight strands, $x_{di} := 0.5 \cdot L_{g_di} - a_{di} = 71 \text{ ft}$ measured from lift point to midspan

Check Drop-In Segment Girder for Lifting Stability, continued:

Gravity moment at midspan

$$M_{g_di} := \frac{(w_{di} \cdot x_{di})}{2 \cdot L_{di}} \cdot (L_{di}^2 - a_{di}^2 - x_{di} \cdot L_{di}) - \frac{w_{di} \cdot a_{di}^2 \cdot (L_{di} - x_{di})}{2 \cdot L_{di}} = 3576.51 \text{ kip ft}$$

Prestress $P_{di} := 1300 \text{ kip}$ (approximate)Eccentricity of strands $e_{di} := y_{b_di} - 3 \text{ in} - 2 \text{ in} = 41.18 \text{ in}$ (below centroid)

Compressive stress in top flange from gravity load and prestress

$$f_{t_di} := \frac{M_{g_di} \cdot y_{t_di}}{I_{x_di}} + \frac{P_{di}}{A_{g_di}} - \frac{P_{di} \cdot e_{di} \cdot y_{t_di}}{I_{x_di}} = 629.89 \text{ psi}$$

Lateral moment capacity

$$M_{lat_di} := \frac{(f_r + f_{t_di}) I_{y_di}}{\left(\frac{b_{t_di}}{2}\right)} = 414.2 \text{ kip ft}$$

Initial roll angle

$$\Theta_{i_di} := \frac{e_{i_di}}{y_{r_di}} = 0.029 \text{ rad}$$

Assuming a small angle, $\Theta = \sin \Theta = \tan \Theta$

Equilibrium roll angle

$$\Theta_{di} := \Theta_{i_di} \cdot \left[\frac{1}{1 - \left(\frac{z_{o_di}}{y_{r_di}}\right)} \right] = 0.041 \text{ rad}$$

Maximum roll angle for cracking

$$\Theta_{max_di} := \frac{M_{lat_di}}{M_{g_di}} = 0.116 \text{ rad}$$

Maximum roll angle for failure

$$\Theta_{max,p_di} := \sqrt{\frac{e_{i_di}}{2.5 \cdot z_{o_di}}} = 0.202 \text{ rad}$$

Lateral deflection of center of gravity of girder including rotation effects

$$z_{o,p_di} := z_{o_di} \cdot (1 + 2.5 \cdot \Theta_{max,p_di}) = 18.85 \text{ in}$$

Factor of safety for cracking

$$FS_{c_di} := \frac{1}{\frac{z_{o_di}}{y_{r_di}} + \frac{\Theta_{i_di}}{\Theta_{max_di}}} = 1.86 \quad \text{(Manual Eq. D-4.3b)}$$

Factor of safety for failure

$$FS_{f_di} := \frac{y_{r_di} \cdot \Theta_{max,p_di}}{z_{o,p_di} \cdot \Theta_{max,p_di} + e_{i_di}} = 1.74 \quad \text{(Manual Eq. D-4.3a)}$$

Since factor of safety for cracking is above the recommended 1.0 minimum, girder is adequate.
 Since factor of safety for failure is above the recommended 1.5 minimum, girder is adequate.

Check Pier Segment Girder for Lifting Stability:

Length between lifting points $L_{ps} := 114$ ft

Girder properties (use weighted average of BT-78 and BT-180 and BT-90 to represent haunched section)

Girder length $L_{g_ps} := 124$ ft

Minor axis moment of inertia

$$I_{y_ps} := \frac{(115158 \text{ in}^4 + 2 \cdot 136679 \text{ in}^4 + 115887 \text{ in}^4)}{4} = 126100.75 \text{ in}^4$$

Distance from centroid to bottom fiber $y_{b_ps} := \frac{(40.21 \text{ in} + 2 \cdot 91.01 \text{ in} + 46.18 \text{ in})}{4} = 67.1 \text{ in}$

Distance from centroid to top fiber $y_{t_ps} := \frac{(37.79 \text{ in} + 2 \cdot 88.99 \text{ in} + 43.82 \text{ in})}{4} = 64.9 \text{ in}$

Major axis moment of inertia

$$I_{x_ps} := \frac{(1014889 \text{ in}^4 + 2 \cdot 8434913 \text{ in}^4 + 1455041 \text{ in}^4)}{4} = 4834939 \text{ in}^4$$

Width of bottom flange $b_{b_ps} := 30$ in

Initial lateral eccentricity (based on 1/4 inch plus half of PCI sweep tolerance of 1/8" per 10 ft)

$$e_{i_ps} := 0.25 \text{ in} + \left(\frac{1}{8} \text{ in}\right) \cdot \left(\frac{L_{g_ps}}{10 \text{ ft}}\right) \left[\left(\frac{L_{ps}}{L_{g_ps}}\right)^2 - \frac{1}{3}\right] = 1.04 \text{ in}$$

Cantilever overhang during lift

$$a_{ps} := \frac{(L_{g_ps} - L_{ps})}{2} = 5 \text{ ft}$$

Weight of beam per unit length (use weighted average of BT-78 and BT-180 and BT-90 to represent haunched section)

Gross area of concrete $A_{g_ps} := \frac{1261 \text{ in}^2 + 2 \cdot 2179 \text{ in}^2 + 1369 \text{ in}^2}{4} = 1747 \text{ in}^2$

$$w_{ps} := A_{g_ps} \cdot 150 \text{ pcf} = 1.82 \frac{\text{kip}}{\text{ft}}$$

Lateral deflection of center of gravity of girder with full dead weight applied laterally

$$z_{o_ps} := w_{ps} \cdot \left(\frac{0.1 \cdot L_{ps}^5 - a_{ps}^2 \cdot L_{ps}^3 + 3 \cdot a_{ps}^4 \cdot L_{ps} + 1.2 \cdot a_{ps}^5}{12 \cdot E_c \cdot I_{y_ps} \cdot L_{g_ps}} \right) = 5.96 \text{ in}$$

Assuming straight strands, $x_{ps} := 0.5 \cdot L_{g_ps} - a_{ps} = 57$ ft measured from lift point to midspan

Prestress $P_{ps} := 1600$ kip (approximate)

Eccentricity of strands $e_{ps} := y_{t_ps} - \frac{[16 \cdot 2 \text{ in} + 10 \cdot (78 \text{ in} - 3 \text{ in} - 2 \text{ in} - 2 \text{ in})]}{38} = 44.11 \text{ in}$ (above centroid)

Gravity moment at midspan

$$M_{g_ps} := \frac{(w_{ps} \cdot x_{ps})}{2 \cdot L_{ps}} \cdot (L_{ps}^2 - a_{ps}^2 - x_{ps} \cdot L_{ps}) - \frac{w_{ps} \cdot a_{ps}^2 \cdot (L_{ps} - x_{ps})}{2 \cdot L_{ps}} = 2933.5 \text{ kip ft}$$

Check Pier Segment Girder for Lifting Stability, continued:

Compressive stress in bottom flange from gravity load and prestress

$$f_{b_ps} := \frac{-M_{g_ps} \cdot y_{b_ps}}{I_{x_ps}} + \frac{P_{ps}}{A_{g_ps}} - \frac{P_{ps} \cdot e_{ps} \cdot y_{b_ps}}{I_{x_ps}} = -552.16 \text{ psi}$$

Note: Value is negative, so stress is actually tensile, but still less than modulus of rupture $f_r = 699.71$ psi

Lateral moment capacity

$$M_{lat_ps} := \frac{(f_r + f_{b_ps}) I_{y_ps}}{\left(\frac{b_{b_ps}}{2}\right)} = 103.37 \text{ kip ft}$$

Midspan radius of curvature

(note prestress, acting above

$$R_{ps} := \frac{E_c \cdot I_{x_ps}}{(-P_{ps} \cdot e_{ps} - M_{g_ps})} = -2.43 \times 10^5 \text{ in}$$

centroid, causes downward deflection)

Camber with respect to beam ends:

$$\Delta_{ps} := \frac{L_{g_ps}^2}{8R_{ps}} = -1.14 \text{ in.}$$

Height of roll axis above centroid

modified to include camber

$$y_{r_ps} := y_{t_ps} - \Delta_{ps} \left[\left(\frac{L_{ps}}{L_{g_ps}} \right)^2 - \frac{1}{3} \right] = 65.48 \text{ in}$$

Initial roll angle

$$\Theta_{i_ps} := \frac{e_{i_ps}}{y_{r_ps}} = 0.016 \text{ rad}$$

Assuming a small angle, $\Theta = \sin \Theta = \tan \Theta$

$$\Theta_{ps} := \Theta_{i_ps} \cdot \left[\frac{1}{1 - \left(\frac{z_{o_ps}}{y_{r_ps}} \right)} \right] = 0.018 \text{ rad}$$

Equilibrium roll angle

Maximum roll angle for cracking

$$\Theta_{max_ps} := \frac{M_{lat_ps}}{M_{g_ps}} = 0.035 \text{ rad}$$

$$\Theta_{max.p_ps} := \sqrt{\frac{e_{i_ps}}{2.5 \cdot z_{o_ps}}} = 0.265 \text{ rad}$$

Maximum roll angle for failure

Theoretical lateral deflection of center of gravity of girder including rotation effect

$$z_{o.p_ps} := z_{o_ps} \cdot (1 + 2.5 \cdot \Theta_{max.p_ps}) = 9.91 \text{ in}$$

Factor of safety for cracking

$$FS_{c_ps} := \frac{1}{\frac{z_{o_ps}}{y_{r_ps}} + \frac{\Theta_{i_ps}}{\Theta_{max_ps}}} = 1.84 \quad (\text{Manual Eq. D-4.3b})$$

Factor of safety for failure

$$FS_{f_ps} := \frac{y_{r_ps} \cdot \Theta_{max.p_ps}}{z_{o.p_ps} \cdot \Theta_{max.p_ps} + e_{i_ps}} = 4.73 \quad (\text{Manual Eq. D-4.3a})$$

Since factor of safety for cracking is above the recommended 1.0 minimum, girder is adequate.

Since factor of safety for failure is above the recommended 1.5 minimum, girder is adequate.

Check Girder End Segment for Beam Rollover:

The standard sized BT-78 girder will be used. Therefore exposed height $h := 78$ in

Use Figure 26.5-1A in ASCE 7-10 to determine basic wind speed for Risk Category II.

$$V := 120 \text{ miles per hour}$$

Use Manual Appendix D Table D-3.2 to determine design wind speed reduction based on construction duration.

Wind Velocity Modification Factor

$$V_m := 0.65 \quad \text{Assume less than 6 weeks duration for casting concrete splices.}$$

Design Wind Speed

$$DWS := V_m \cdot V \quad DWS = 78 \text{ mph}$$

Incorporate modified Design Wind Speed into pressure equation

Velocity Pressure Exposure Coefficient (ASCE 7-10, Sect. 26.7.3)

Assume Surface Roughness B – Wooded and/or suburban areas with numerous closely spaced obstructions. Therefore, Exposure Category B
Manual Table D-3.1, for Height = 30 feet, $K_z = 0.70$

Take wind directionality factor as 0.85 and topographic factor as 1.0.

$$K_z := 0.7 \quad K_{zt} := 1 \quad K_d := 0.85$$

Velocity Pressure (Manual Eq. D-3.4b)

$$q_z := 0.00256 \text{ psf} \cdot K_z \cdot K_{zt} \cdot K_d \cdot DWS^2 \quad q_z = 9.3 \text{ psf}$$

Gust Effect Factor

$$G := 0.85$$

Net Force Coefficient (Assuming deck forms not in place)

$$C_f := 2.2 \quad \text{since ratio of girder spacing to depth, } S/d < 2$$

Net Pressure

$$Q_z := G \cdot C_f \cdot q_z \quad Q_z := 17.3 \text{ psf}$$

Line Load from Wind

$$w_w := Q_z \cdot h = 112.64 \text{ plf}$$

Length between supports (from abutment to first splice) $L_s := 146$ ft

Moment of inertia about minor axis

$$I_y := 115158 \text{ in}^4$$

Lateral deflection at midspan due to wind on uncracked section $\Delta_w := \frac{5w_w \cdot L_s^4}{384E_c \cdot I_y} = 1.88$ in

Wind overturning moment acting on girder end segment $M_o := w_w \cdot L_s \cdot \frac{h}{2} = 53.45$ kip ft

Applied wind moment at midspan of girder end segment $M_w := \frac{w_w \cdot L_s^2}{8} = 300.14$ kip ft

Check Girder End Segment for Beam Rollover:

The standard sized BT-78 girder will be used. Calculate factor of safety for girder cracking and rollover.

Note that wind load assumes a short exposure duration (< 6 weeks) prior to casting of splices.

Gross area of concrete $A_g := 1261 \text{ in}^2$

Weight per length of girder $w := A_g \cdot 150 \text{ pcf} = 1.314 \text{ klf}$

Total weight of girder $W := w \cdot L_s = 191.78 \text{ kip}$

$$e_w := \frac{M_o}{W} = 3.34 \text{ in}$$

Eccentricity from wind overturning moments
Initial eccentricity of center of gravity of girder

(Based on 1 inch plus PCI
sweep tolerance of 1/8" per 10 ft)

$$e_i := \Delta_w + e_w + 1 \text{ in} + \left(\frac{1}{8} \text{ in}\right) \cdot \left(\frac{L_s}{10 \text{ ft}}\right) \cdot \left(1 - \frac{1}{3}\right) = 7.45 \text{ in} \quad (\text{includes wind effects})$$

Sum of rotational spring constants at supports $K_\theta := 100000 \text{ in kip per radian}$ (conservatively assumed)

Note that per the figures in Hurff (2010, a lower bound on the bearing pad rotational stiffness might be around 100000 in-kip/rad for a 24 in wide pad and around 50000 in-kip/rad for an 18 in wide pad. The bottom flange in this example is 30 in wide, so 100000 in-kip/rad should be conservative. Lateral restraint (friction) from strongback system is conservatively ignored. A creep reduction factor should also be applied to the rotational stiffness if bracing is not installed immediately after crane release (ignored for simplicity due to other conservatism).

Applied gravity moment $M_g := \frac{w \cdot L_s^2}{8} = 3499.93 \text{ kip ft}$

Distance from centroid to bottom fiber $y_b := 40.21 \text{ in}$

Distance from centroid to top fiber $y_t := 37.79 \text{ in}$

Prestress $P := 700 \text{ kip}$ (approximate)

Eccentricity of strands $e_s := y_b - \frac{(10 \cdot 3 \text{ in} + 6 \cdot 5 \text{ in})}{16} = 36.46 \text{ in}$ (below centroid)

Gross moment of inertia about the x-axis $I_{gx} := 1014899 \text{ in}^4$

Width of top flange $b_t := 62 \text{ in}$

Width of bottom flange $b_b := 30 \text{ in}$

Compressive stress at top fiber due to gravity
load and prestress and wind

$$f_t := \frac{M_g \cdot y_t}{I_{gx}} + \frac{P}{A_g} - \frac{P \cdot e_s \cdot y_t}{I_{gx}} - \frac{M_w \cdot \left(\frac{b_t}{2}\right)}{I_y} = 199.11 \text{ psi}$$

Modulus of rupture; recall $f_r = 0.7 \text{ ksi}$, so Lateral moment capacity $M_{\text{lat}} := \frac{(f_r + f_t) \cdot I_y}{\left(\frac{b_t}{2}\right)} = 278.24 \text{ kip ft}$

Midspan radius of curvature $R := \frac{E_c \cdot I_{gx}}{(P \cdot e_s - M_g)} = -3.27 \times 10^5 \text{ in}$

Camber with respect to beam ends

(note gravity deflection not overcome by prestress): $\Delta_{\text{ps}} := \frac{L_s^2}{8 \cdot R} = -1.17 \text{ in}$

Height of the center of gravity above roll axis
modified to include camber

(assuming roll axis at bearing pad mid-depth) $y := y_b + 1.5 \text{ in} + \Delta \cdot \left(1 - \frac{1}{3}\right) = 40.93 \text{ in}$

Check Girder End Segment for Beam Rollover, continued:

Tilt angle at which cracking begins $\Theta_{\max} := \frac{M_{\text{lat}}}{M_g} = 0.079 \text{ rad}$

Superelevation, or tilted angle of support (Based on PCI flatness tolerance of 1/16" over bearing pad width bottom flange width and DOT flatness tolerance of 1/16" over bearing seat width)

$$\alpha := \frac{2 \left(\frac{1}{16} \text{ in} \right)}{b_b} = 4.17 \times 10^{-3} \text{ rad}$$

Lateral deflection of center of gravity of girder $z_{\text{o.bar}} := \left(\frac{1}{120} \right) \cdot \left[\frac{(w \cdot L_s^4)}{(E_c \cdot I_y)} \right] = 14.07 \text{ in}$

Radius of stability $r := \frac{K_{\Theta}}{W} = 521.44 \text{ in}$

Width of bottom flange $b_b := 30 \text{ in}$

Maximum resisting moment arm $z_{\max} := \frac{b_b}{2} = 15 \text{ in}$ from edge of bottom flange

Height of roll center $h_r := 1.5 \text{ in}$ above bearing pad mid-depth (3 inch elastomeric bearing assumed)

Roll angle of major axis with respect to vertical $\Theta_{\text{major}} := \frac{(\alpha \cdot r + e_i)}{r - y - z_{\text{o.bar}}} = 0.021 \text{ rad}$

Title angle at maximum factor of safety against failure $\Theta_{\text{max.p}} := \frac{z_{\max} - h_r \cdot \alpha}{r} + \alpha = 0.033 \text{ rad}$

Lateral deflection of center of gravity of girder including rotation effects

$$z_{\text{o.bar.p}} := z_{\text{o.bar}} \cdot (1 + 2.5 \cdot \Theta_{\text{max.p}}) = 15.22 \text{ in}$$

Factor of safety for cracking

$$FS_c := \frac{[r \cdot (\Theta_{\max} - \alpha)]}{z_{\text{o.bar}} \cdot \Theta_{\max} + e_i + y \cdot \Theta_{\max}} = 3.31 \quad (\text{Manual Eq. D-5.3b})$$

Minimum factor of safety = 1.0, therefore OK.

Factor of safety for rollover

$$FS_f := \frac{[r \cdot (\Theta_{\text{max.p}} - \alpha)]}{z_{\text{o.bar.p}} \cdot \Theta_{\text{max.p}} + e_i + y \cdot \Theta_{\text{max.p}}} = 1.61 \quad (\text{Manual Eq. D-5.3a})$$

Minimum factor of safety = 1.5, therefore OK.

Note that buckling of the long span PPC girder at the end segment (simply supported prior to the splice pour) is not checked. This is because the inelastic lateral-torsional buckling failure mode and corresponding equations are not applicable for PPC bridge girders, as they do not crack under selfweight for normal designs.

Design Hanger Beams at Strongback Between Drop-In Girder Segment and Pier Segment:

Design load shall be one half of the weight of the drop-in girder segment with load factor of 1.25.

$$P_{di} := 1.25 \cdot 0.5 \cdot 0.5 \cdot w_{di} \cdot L_{g_di} = 67.74 \text{ kip} \quad (\text{load per channel})$$

Check pair of MC12x35 hanger beams on underside of drop-in segment

Unbraced length

$$L_b := 3.5 \text{ ft} \quad (\text{assumed})$$

Max moment

$$M_{u_mc} := \frac{(P_{di} \cdot L_b)}{4} = 59.27 \text{ kip ft}$$

Steel yield strength

$$F_y := 36 \text{ ksi}$$

Steel modulus of elasticity

$$E_s := 290000000 \text{ psi}$$

Section modulus

$$Z_x := 43.2 \text{ in}^3$$

Radius of gyration about y-axis

$$R_y := 1.11 \text{ in}$$

Flexural resistance based on yielding

$$M_p := F_y \cdot Z_x = 129.6 \text{ kip ft}$$

$$L_p := 1.76 \cdot r_y \cdot \sqrt{\left(\frac{E_s}{F_y}\right)} = 4.62 \text{ ft} \quad (\text{AASHTO 6.12.2.2.5-7})$$

Since L_p is greater than L_b than there is no need to check for lateral torsional buckling.

Factored flexural resistance

$$M_r := 1.0 \cdot M_p = 129.6 \text{ kip ft}$$

Since M_r is greater than M_u , section is satisfactory in flexure

$$M_r = 129.6 \text{ kip ft} > M_{u_mc} = 59.27 \text{ kip ft}$$

Max. shear

$$V_{u_mc} := 0.5 \cdot P_{di} = 33.87 \text{ kip}$$

Depth of web

$$D_{web} := 12 \text{ in}$$

Design Hanger Beams at Strongback Between Drop-In Girder Segment and Pier Segment, cont.:

Thickness of web

$$t_{web} := 0.465 \text{ in}$$

Plastic shear force

$$V_p := 0.58 \cdot F_y \cdot D_{web} \cdot t_{web} = 116.51 \text{ kip} \quad (\text{AASHTO 6.10.9.2-2})$$

$$\text{Ratio1} := \frac{D_{web}}{t_{web}} = 25.81$$

Transverse stiffener spacing

$$d_o := L_b = 42 \text{ in} \quad (\text{assumed})$$

Shear buckling coefficient

$$k := 5 + \left[\frac{5}{\left(\frac{d_o}{D_{web}} \right)^2} \right] = 5.41 \quad (\text{AASHTO 6.10.9.3.2-7})$$

$$\text{Ratio2} := 1.12 \cdot \sqrt{\frac{E_s \cdot k}{F_y}} = 73.92$$

Ratio of shear buckling resistance to yield strength

$$C_{mc} := 1.0 \quad (\text{AASHTO 6.10.9.3.2-4})$$

Nominal shear resistance

$$V_n := C_{mc} \cdot V_p = 116.51 \text{ kip}$$

factored shear resistance

$$v_r := 1.0 V_n = 116.51 \text{ kip}$$

Since V_r is greater than V_u , section is satisfactory in shear

$$V_n = 116.51 \text{ kip} > V_{u_{mc}} = 33.87 \text{ kip}$$

Design Hanger Bars at Strongback Between Drop-In Girder Segment and Pier Segment

Check pair of 1 5/8" diameter steel threaded bars (AASHTO 6.8.2.1)

Tension in bar (one at each end of the pair of channels)

$$T_{bar} := 2 \cdot V_{u_{mc}} = 67.74 \text{ kip}$$

Design Hanger Bars at Strongback Between Drop-In Girder Segment and Pier Segment, cont.:

Gross and net area of bar

$$A_g := \left(\frac{1.625 \text{ in}}{2} \right)^2 \cdot \pi = 2.07 \text{ in}^2 \quad A_n := 0.7854 \left(1.625 \text{ in} - \frac{0.9743 \text{ in}}{5} \right)^2 = 1.61 \text{ in}^2$$

Yield strength of bar

$$F_{y,\text{bar}} := 36 \text{ ksi}$$

Ultimate strength of bar

$$F_{u,\text{bar}} := 58 \text{ ksi}$$

Reduction factor for holes

$$R_p := 1.0$$

Reduction factor to account for shear lag

$$U := 1.0$$

Resistance factor for yielding

$$\Phi_y := 0.95$$

Resistance factor for fracture

$$\Phi_u := 0.80$$

Tensile resistance based on yielding

$$P_{r,y} := \Phi_y \cdot A_g \cdot F_{y,\text{bar}} = 70.93 \text{ kip}$$

Tensile resistance based on fracture

$$P_{r,u} := \Phi_u \cdot F_{u,\text{bar}} \cdot A_n \cdot R_p \cdot U = 74.54 \text{ kip}$$

Since P_r is greater than T_{bar} , section is satisfactory in tension.

$$P_r := \min (P_{r,y}, P_{r,u}) = 70.93 \text{ kip} > T_{\text{bar}} = 67.74 \text{ kip}$$

Design Strongback Between Drop-In Girder Segment and Pier Segment:

Try twin W14x120 strongback girders (this example uses AISC 13th ed. Flexure section F2 for simplicity).

Unbraced Length

$$L_{b,w14} := 10.5 \text{ ft (assumed)}$$

Max. moment

$$M_{u,w14} := P_{di} \cdot L_{b,w14} = 711.24 \text{ kip ft}$$

Steel yield strength

$$F_{y,w14} := 50 \text{ ksi}$$

Design Strongback Between Drop-In Girder Segment and Pier Segment, continued:

Section modulus

$$Z_{x.w14} := 212 \text{ in}^3$$

Flexural resistance based on yielding

$$M_{p.w14} := F_{y.w14} \cdot Z_{x.w14} = 883.33 \text{ kip ft}$$

Flexural resistance using lateral torsional buckling

$$L_{p.w14} := 13.2 \text{ ft}$$

$$L_{r.w14} := 52.0 \text{ ft}$$

Since L_{bw14} is less than L_{pw14} lateral torsional buckling does not need to be checked.

Flexural resistance

$$\phi M_{n.w14} := 0.9 \cdot M_{p.w14} = 795 \text{ kip ft}$$

Since ϕM_n is greater than M_u , section is satisfactory to flexure.

$$\phi M_{n.w14} = 795 \text{ kip ft} > M_{u.w14} = 711.24 \text{ kip ft}$$

Max. shear

$$V_{u.w14} := P_{di} = 67.74 \text{ kip}$$

Depth of web

$$D_{web.w14} := 14.5 \text{ in}$$

Thickness of web

$$t_{web.w14} := 0.590 \text{ in}$$

Plastic shear force

$$V_{p.w14} := 0.58 \cdot F_{y.w14} \cdot D_{web.w14} \cdot t_{web.w14} = 249.09 \text{ kip} \quad (\text{AASHTO 6.10.9.2-2})$$

$$\text{Ratio}_{w14} := \frac{D_{web.w14}}{t_{web.w14}} = 24.58$$

Transverse stiffener spacing

$$d_{o.w14} := L_{b.w14} = 126 \text{ in}$$

Design Strongback Between Drop-In Girder Segment and Pier Segment, continued:

Shear buckling coefficient

$$k_{w14} := 5 + \left[\frac{5}{\left(\frac{d_{o,w14}}{D_{web,w14}} \right)^2} \right] = 5.07 \quad (\text{AASHTO 6.10.9.3.2-7})$$

$$\text{Ratio2}_{w14} := 1.12 \sqrt{\frac{(E_s \cdot k_{w14})}{F_{y,w14}}} = 60.71$$

Ratio of shear buckling resistance to yield strength

$$C_{w14} := 1.0 \quad (\text{AASHTO 6.10.9.3.2-4})$$

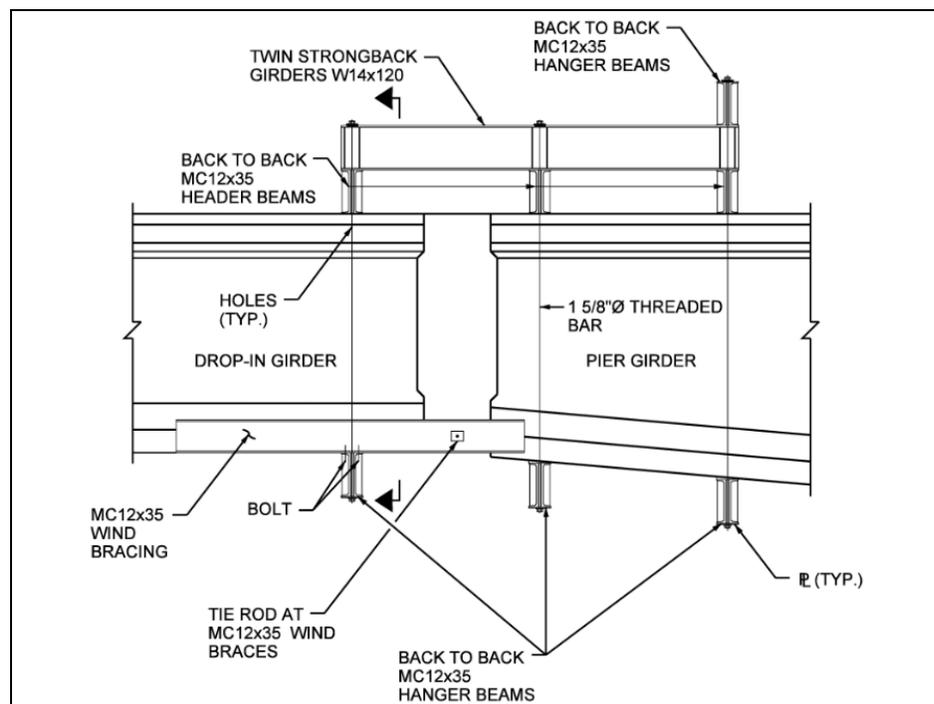
Nominal shear resistance

$$V_{n,w14} := C_{w14} \cdot V_{p,w14} = 248.09 \text{ kip}$$

Since V_n is greater than V_u , section is satisfactory in shear.

$$V_{n,w14} = 248.09 \text{ kip} > V_{u,w14} = 67.74 \text{ kip}$$

See below for a schematic of the complete strongback / hanger bar / hanger beam system.

**Figure B3-6: Typical Strongback System**

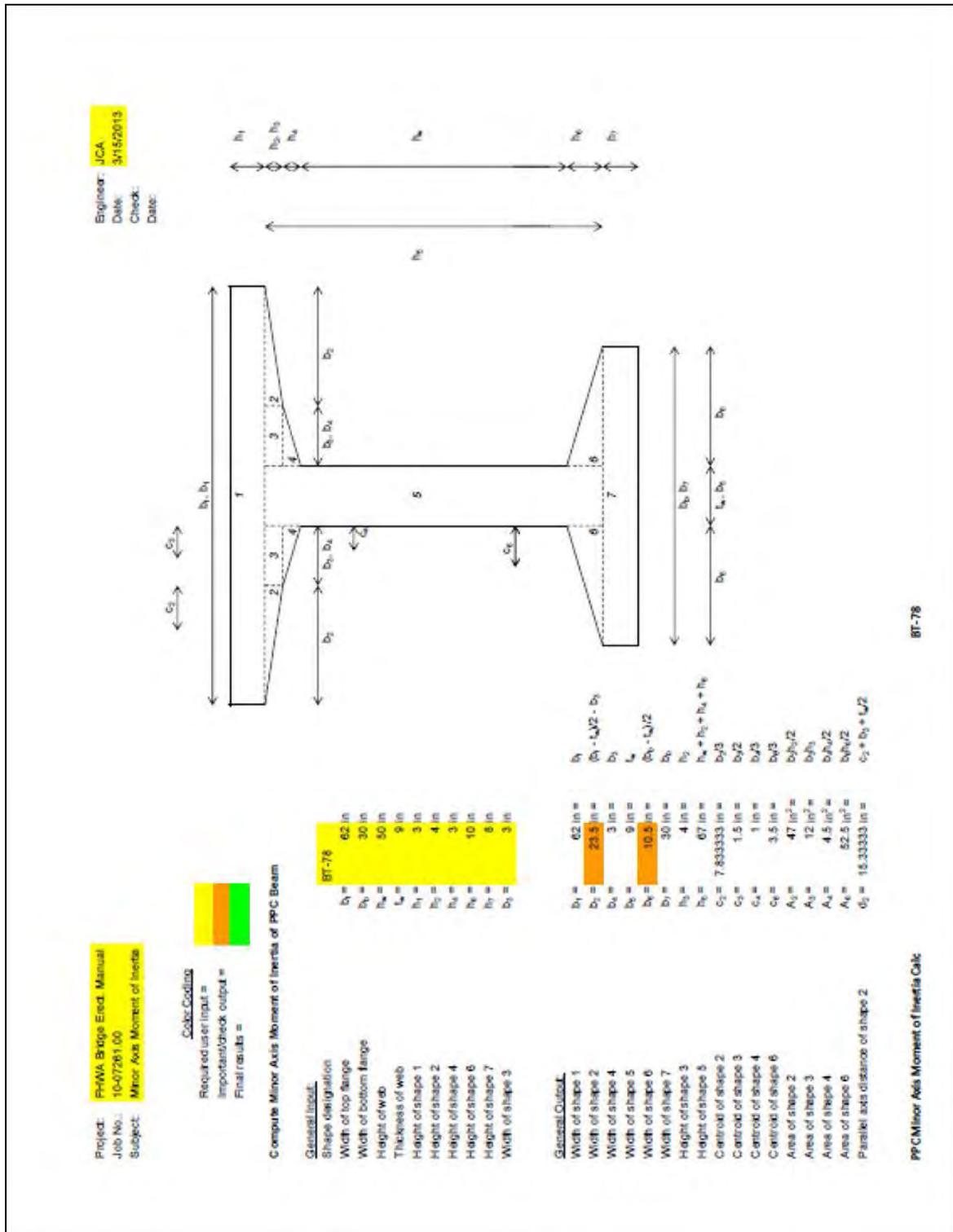


Figure B3-7: Minor Axis Moment of Inertia Calculation for BT-78

Parallel axis distance of shape 3	$d_3 = 6 \text{ in} = c_3 = L/2$	
Parallel axis distance of shape 4	$d_4 = 5.5 \text{ in} = c_4 = L/2$	
Parallel axis distance of shape 6	$d_6 = 8 \text{ in} = c_6 = L/2$	
Total depth	$b = 78 \text{ in} = h_1 + h_2 + h_3$	
Final Results:		
Minor axis moment of inertia of shape 1	$I_1 = 58582 \text{ in}^4 = h_1 b_1^3/12$	
Minor axis moment of inertia of shape 2	$I_2 = 12492.21 \text{ in}^4 = h_2 b_2^3/12 + A_2 d_2^2$	
Minor axis moment of inertia of shape 3	$I_3 = 441 \text{ in}^4 = h_3 b_3^3/12 + A_3 d_3^2$	
Minor axis moment of inertia of shape 4	$I_4 = 138,375 \text{ in}^4 = h_4 b_4^3/12 + A_4 d_4^2$	
Minor axis moment of inertia of shape 5	$I_5 = 4070.25 \text{ in}^4 = h_5 b_5^3/12$	
Minor axis moment of inertia of shape 6	$I_6 = 3681,563 \text{ in}^4 = h_6 b_6^3/12 + A_6 d_6^2$	
Minor axis moment of inertia of shape 7	$I_7 = 18000 \text{ in}^4 = h_7 b_7^3/12$	
Total (major axis) moment of inertia	$I_x = 2 I_2 + 2 I_3 + 2 I_4 + I_5 + 2 I_6 + I_7$	
	$I_x = 113,128.5 \text{ in}^4$	
PPC/Minor Axis Moment of Inertia Calc		BT-78

Figure B3-8: Minor Axis Moment of Inertia Calculation for BT-78, continued

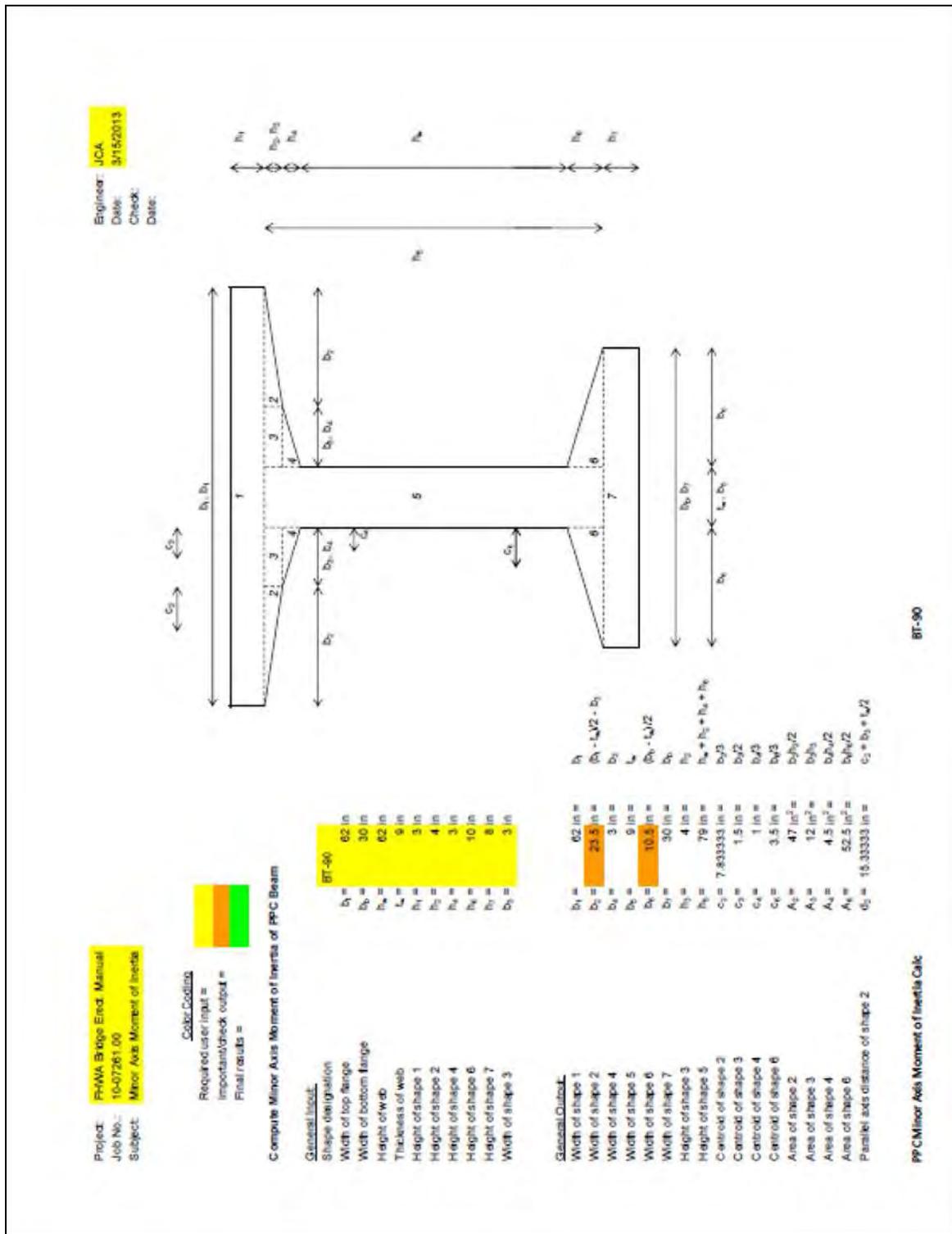


Figure B3-9: Minor Axis Moment of Inertia Calculation for BT-90

Parallel axis distance of shape 3	$d_3 = 6 \text{ in} = c_3 + t/2$
Parallel axis distance of shape 4	$d_4 = 5.6 \text{ in} = c_4 + t/2$
Parallel axis distance of shape 6	$d_6 = 8 \text{ in} = c_6 + t/2$
Total depth	$t = 90 \text{ in} = t_1 + t_2 + t_3$
Final Results:	
Minor axis moment of inertia of shape 1	$I_1 = 69432 \text{ in}^4 = h_b^3 t/12$
Minor axis moment of inertia of shape 2	$I_2 = 12422.21 \text{ in}^4 = h_b^3 t/36 + A_2 d_2^2$
Minor axis moment of inertia of shape 3	$I_3 = 441 \text{ in}^4 = h_b^3 t/12 + A_3 d_3^2$
Minor axis moment of inertia of shape 4	$I_4 = 138,375 \text{ in}^4 = h_b^3 t/36 + A_4 d_4^2$
Minor axis moment of inertia of shape 5	$I_5 = 4769.25 \text{ in}^4 = h_b^3 t/12$
Minor axis moment of inertia of shape 6	$I_6 = 3681.563 \text{ in}^4 = h_b^3 t/36 + A_6 d_6^2$
Minor axis moment of inertia of shape 7	$I_7 = 18000 \text{ in}^4 = h_b^3 t/12$
Total minor axis moment of inertia	$I_y = 173387.3 \text{ in}^4 = I_1 + 2I_2 + 2I_3 + 2I_4 + 2I_5 + 2I_6 + I_7$

Figure B3-10: Minor Axis Moment of Inertia Calculation for BT-90, continued

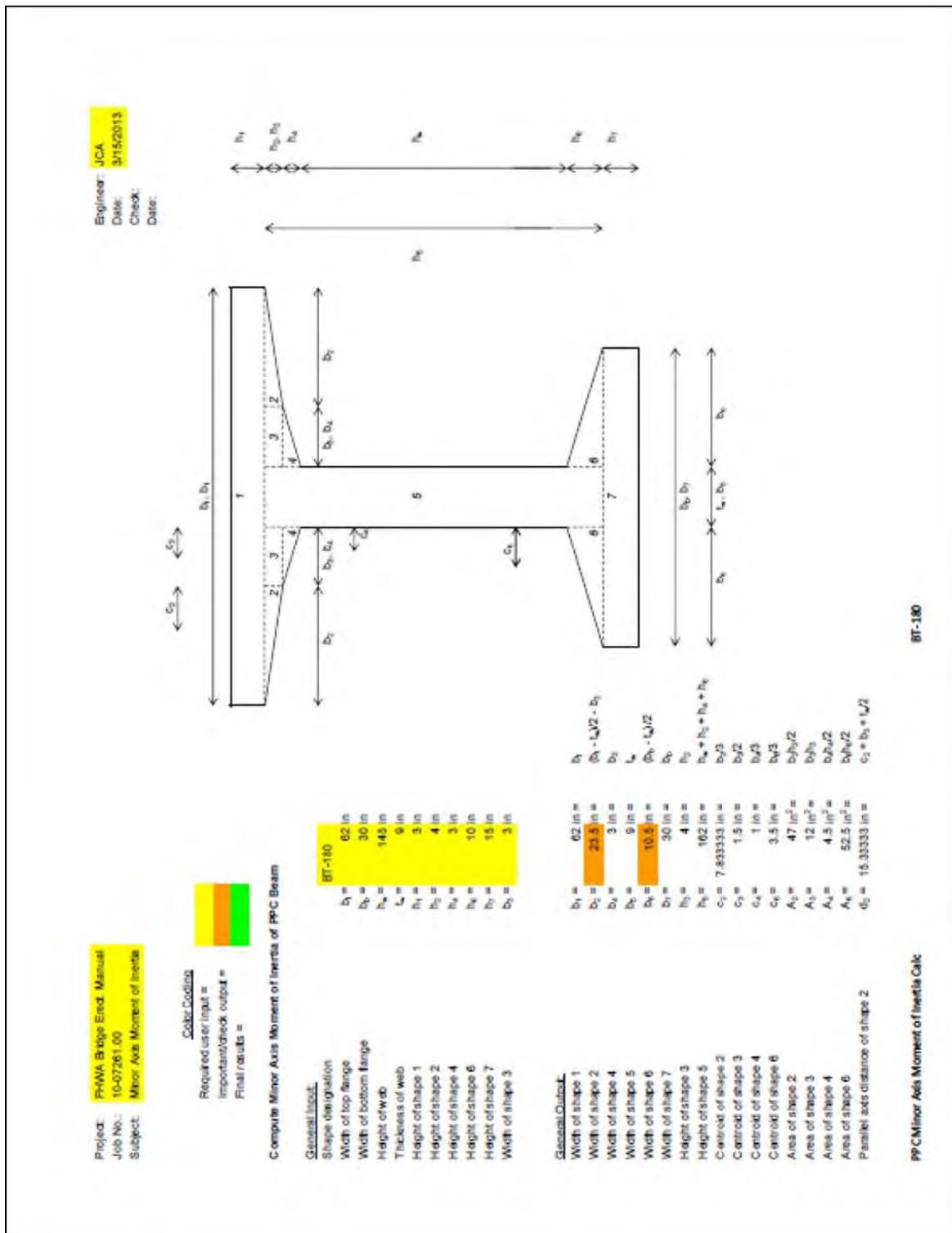


Figure B3-11: Minor Axis Moment of Inertia Calculation for BT-180

Parallel axis distance of shape 3	$c_3 = 6 \text{ in}$	$c_3 + L/2$
Parallel axis distance of shape 4	$c_4 = 5.5 \text{ in}$	$c_4 + L/2$
Parallel axis distance of shape 6	$c_6 = 8 \text{ in}$	$c_6 + L/2$
Total depth	$h = 180 \text{ in}$	$h_1 + h_2 + h_3$
Final Results:		
Minor axis moment of inertia of shape 1	$I_1 = 89482 \text{ in}^4$	$h_1^3/12$
Minor axis moment of inertia of shape 2	$I_2 = 12492.21 \text{ in}^4$	$h_2^3/36 + A_2 d_2^2$
Minor axis moment of inertia of shape 3	$I_3 = 441 \text{ in}^4$	$h_3^3/12 + A_3 d_3^2$
Minor axis moment of inertia of shape 4	$I_4 = 138,375 \text{ in}^4$	$h_4^3/36 + A_4 d_4^2$
Minor axis moment of inertia of shape 5	$I_5 = 9841.5 \text{ in}^4$	$h_5^3/12$
Minor axis moment of inertia of shape 6	$I_6 = 3651,563 \text{ in}^4$	$h_6^3/36 + A_6 d_6^2$
Minor axis moment of inertia of shape 7	$I_7 = 33750 \text{ in}^4$	$h_7^3/12$
Total minor axis moment of inertia	$I_y = 596779.5 \text{ in}^4$	$I_y = 2 I_1 + 2 I_2 + 2 I_3 + I_4 + 2 I_5 + 2 I_6 + I_7$

Figure B3-12: Minor Axis Moment of Inertia Calculation for BT-180, continued

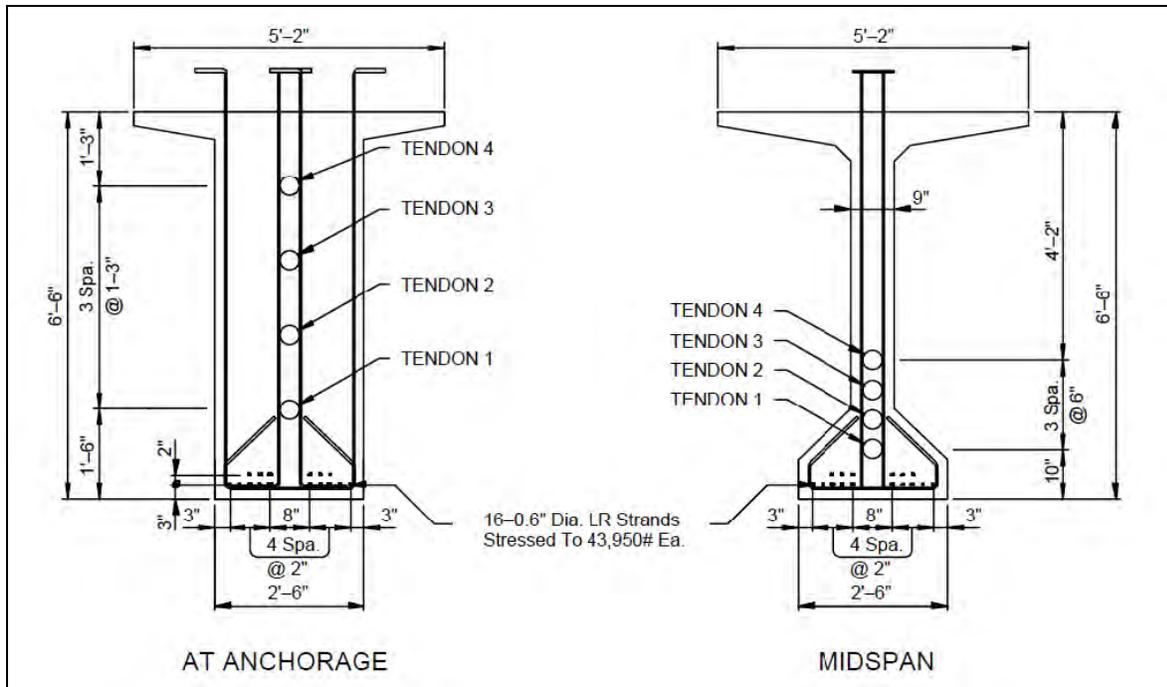


Figure B3-13: Prestress Layout for End Girder Segments

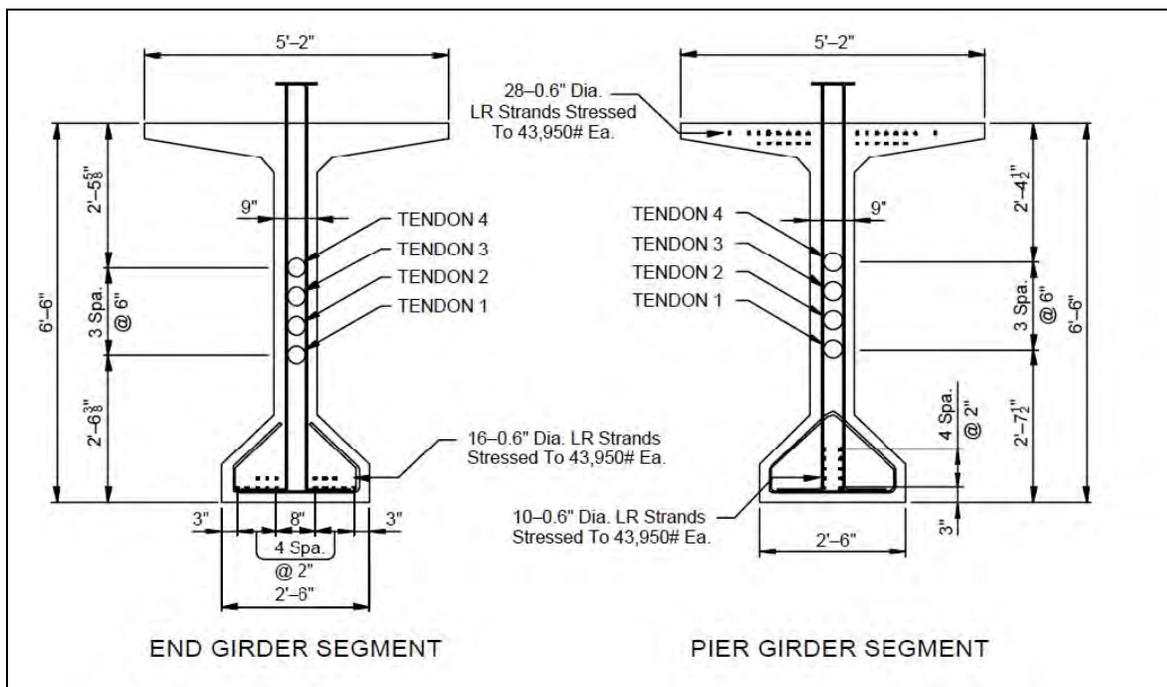


Figure B3-14: Prestress Layout for Girder Segments at Splices in Spans 1 & 3

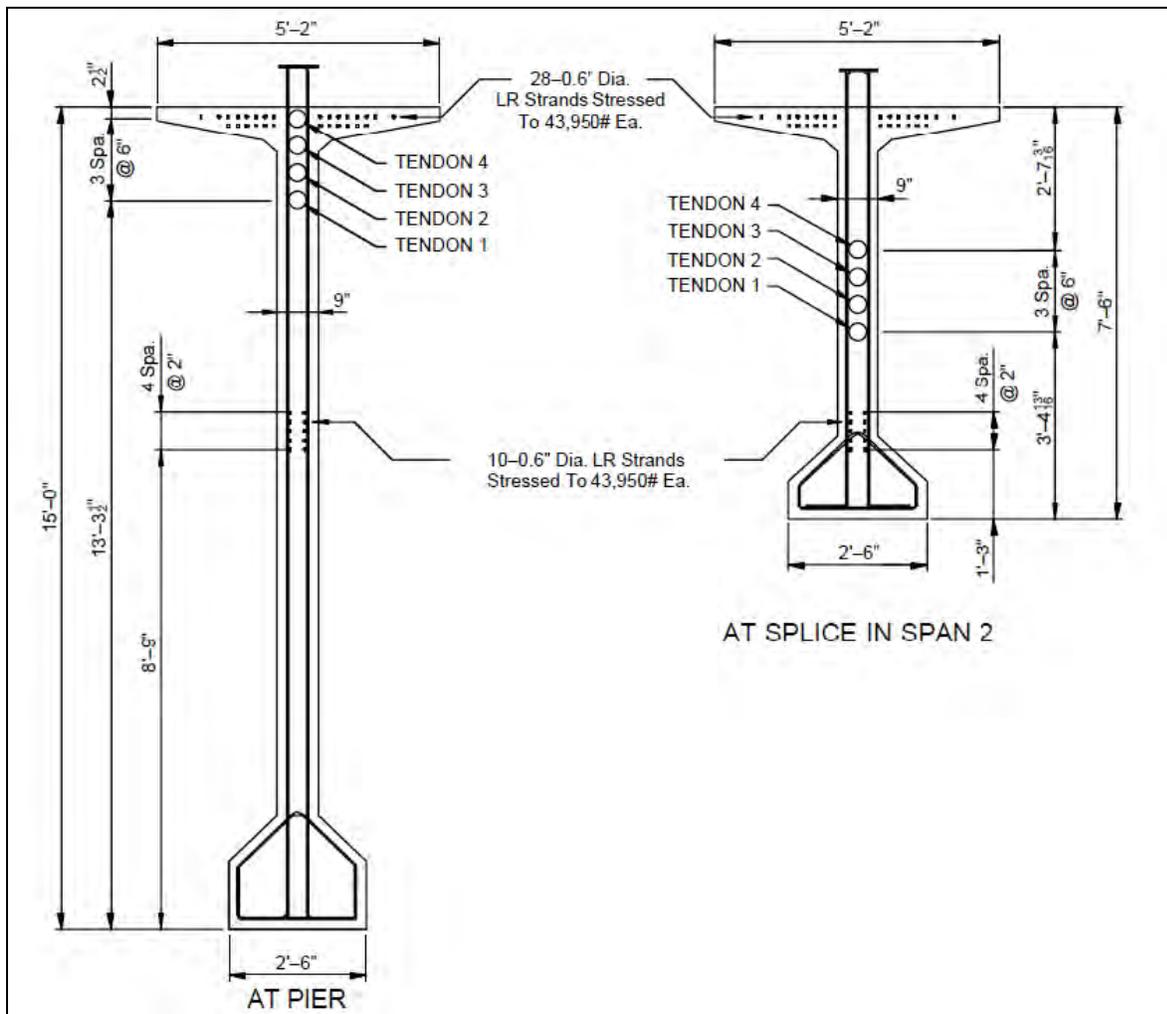


Figure B3-15: Prestress Layout for Pier Girder Segments

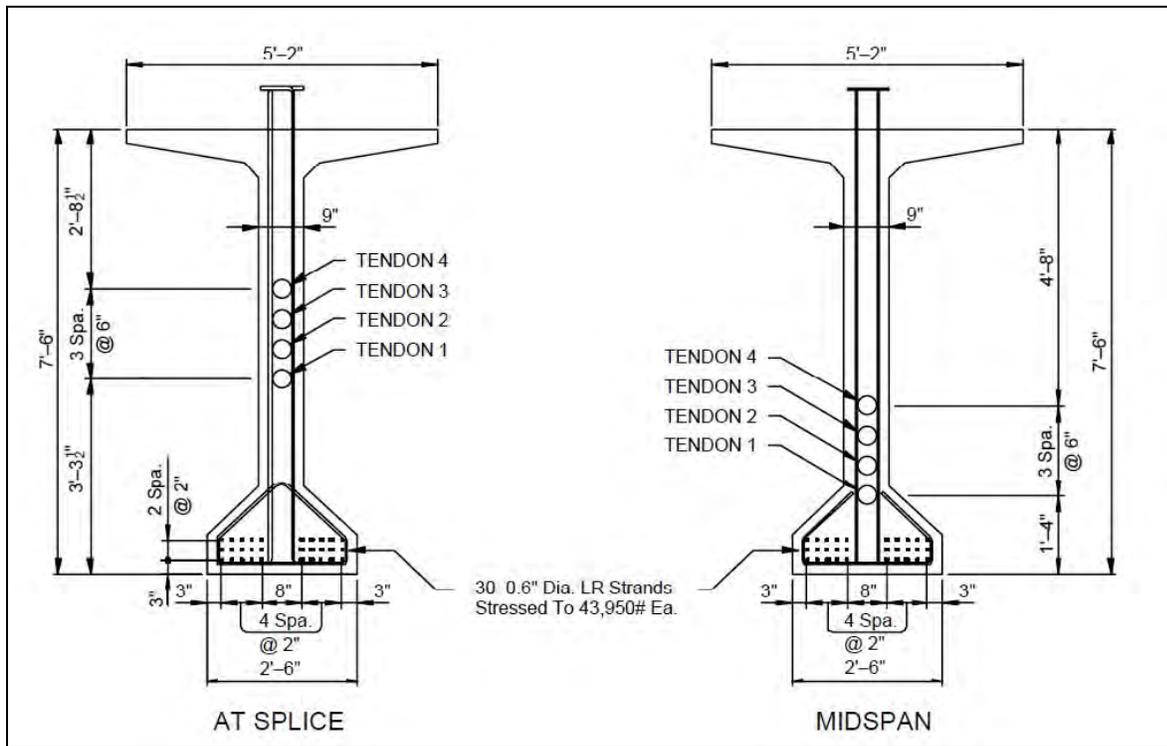
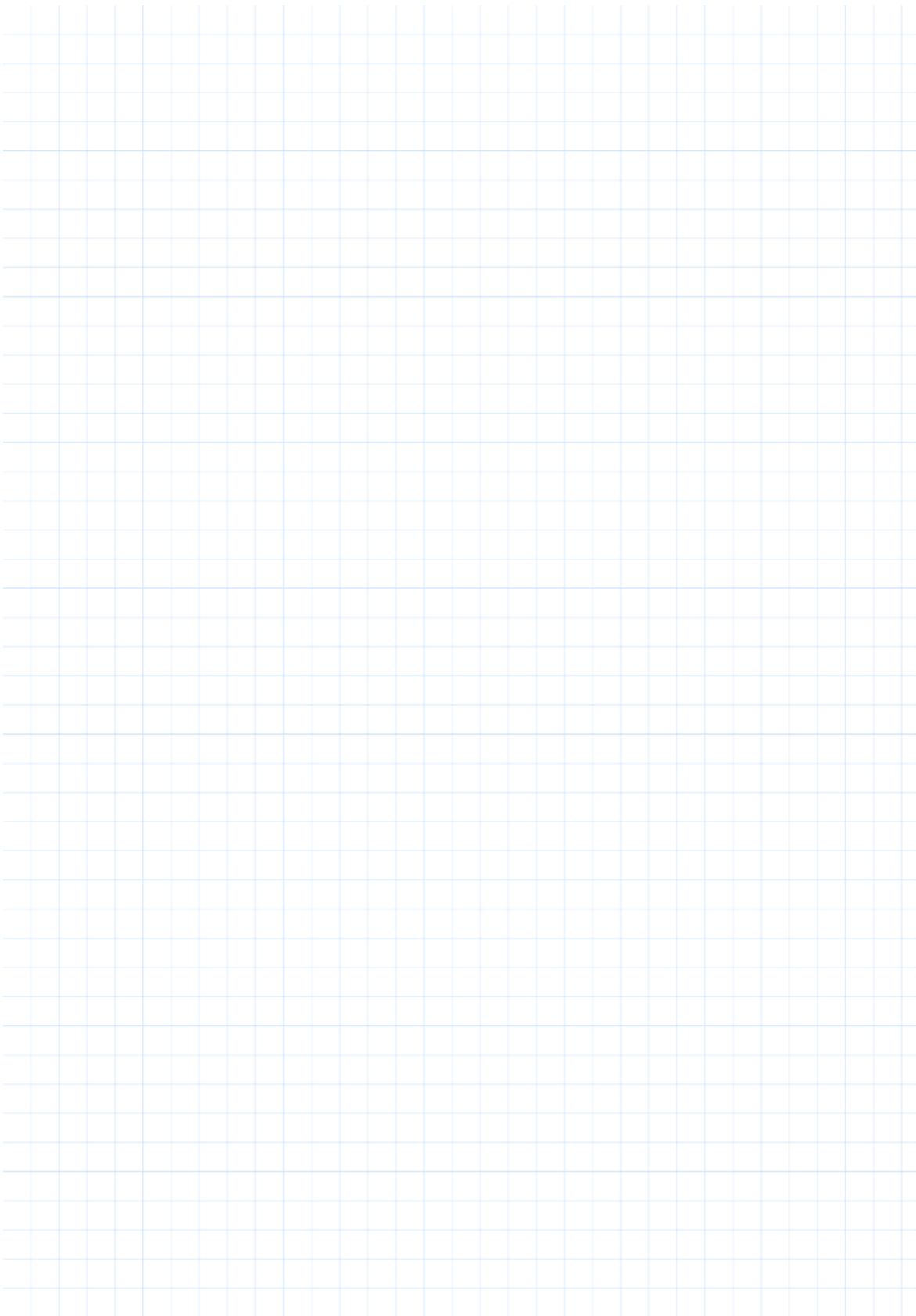


Figure B3-16: Prestress Layout for Drop-In Girder Segments

Reader Notes



Reader
Notes



Example Problem #4: 3-Span Box Girder Curved Steel Structure

The following table of contents illustrates the general categories into which this example problem is subdivided, and the relevant page number at which the start of each category, and its specific component analysis or calculation tasks, may be found.

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Analysis with Missing Bracing		Eigenvalue Buckling Analysis in UTrAp	B.191

Bridge Data:

Continuous 3 span curved steel box girder bridge, 2 girders total. Radius of curvature is 700 ft at centerline of roadway. Out-to-out deck width is 40.5 ft. Girders are spaced at 22.5 ft on centers. Structural deck slab thickness is 9.5 in. with no integral wearing surface.

The spans are symmetric, with Spans 1 and 3 at 160 ft long and Span 2 at 210 ft long measured along the centerline of structure.

The field splices are also symmetric, with Field Section 1 at 96 ft long in Spans 1 and 3 (at Abutments 1 and 2), Field Section 2 at 109 ft long (centered on Piers 1 and 2), and Field Section 3 at 120 ft long in the middle of Span 2. Since these measurements are taken at the centerline of structure, where there is no physical girder, the actual field section lengths vary depending on which girder is considered.

The general plan, cross-section, and various details are presented in the following figures.

Deck Pour Analysis:

For this example, it is assumed the deck pour will run in four casts. The first cast is in Span 1 starting at Abutment 1 and ending at the field splice. The second cast is in Span 2 between field splices. The third cast is in Span 3 from the field splice to Abutment 2. The fourth cast is over Piers 1 and 2. The worst case stage for the Span 1 girder segment is when the deck has been poured over Span 1 up to the splice (first cast). This stage will be examined for girder adequacy. The concrete weight is taken as 150 pcf and treated as permanent dead load, while construction dead load, the weight of the forms, is taken at 10 psf. Construction live load is taken as 20 psf in Spans 2 and 3. The pour sequence is modeled in the 3D finite element program UTrAp. Wind load is assumed to be negligible during the pour and is ignored; thus the Strength III load combination is not considered. The factored results used in the calculations are from the Strength I: $1.25 DC + 1.50 CDL + 1.50 CLL$ load combination. The Strength VI: $1.40 DC + 1.40 CDL + 1.40 CLL$ load combination should also be analyzed; however, it can be seen from the Strength I results that the design will still be adequate for Strength VI, since at most there will be a 12% increase in applied stresses ($1.40 / 1.25$) and all the allowable stresses are significantly greater than the applied stresses.

Evaluation of Stages:

The girders will be checked for adequacy according to AASHTO 2012 LRFD Specifications. The G2 girder (Field Section 1) will be checked for the deck pour loading. Note that many other checks (e.g., erection staging) are not performed here as they are conceptually similar to the calculations shown in Example 1 for straight plate girders. Only the first cast of the deck pour is analyzed here; if the entire cast sequence were analyzed, all of the girder field sections would have to be checked for both positive and negative moment. For example, in the negative moment regions, the welded bottom flange longitudinal T-stiffener, which is located over the piers and terminated at the splices, would need a design check, along with the box girder itself.

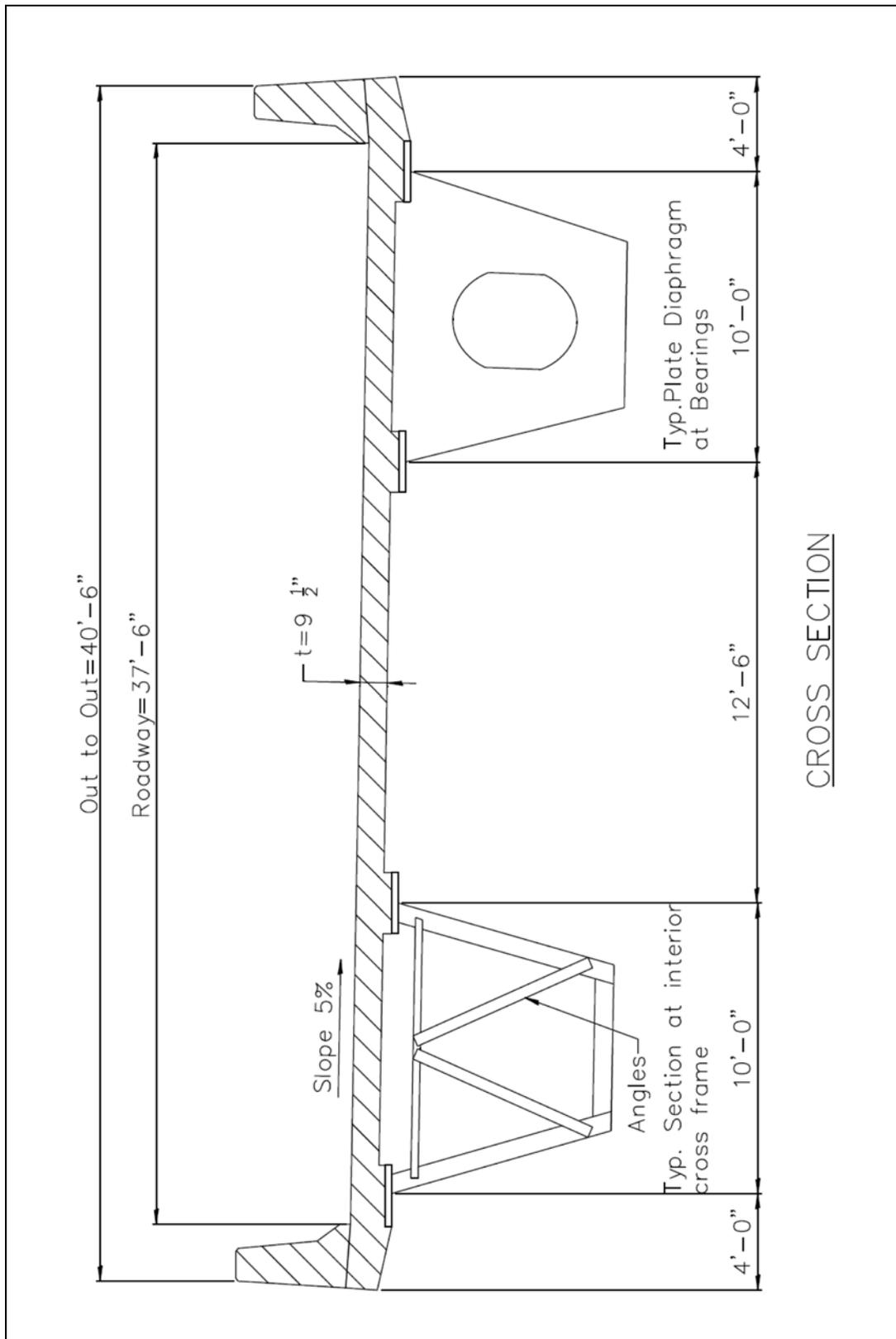


Figure B4-1: Example 4 Cross-section

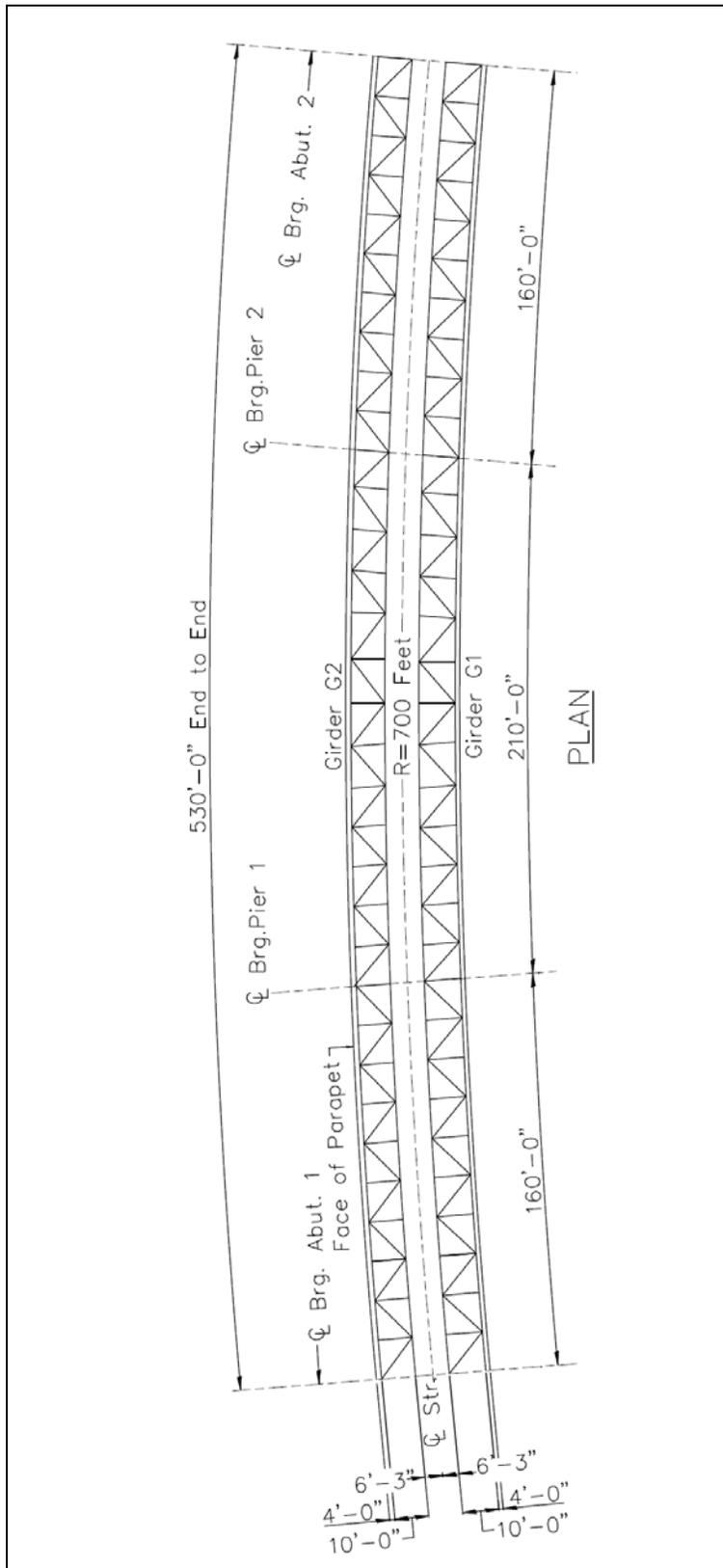


Figure B4-2: Example 4 Framing Plan

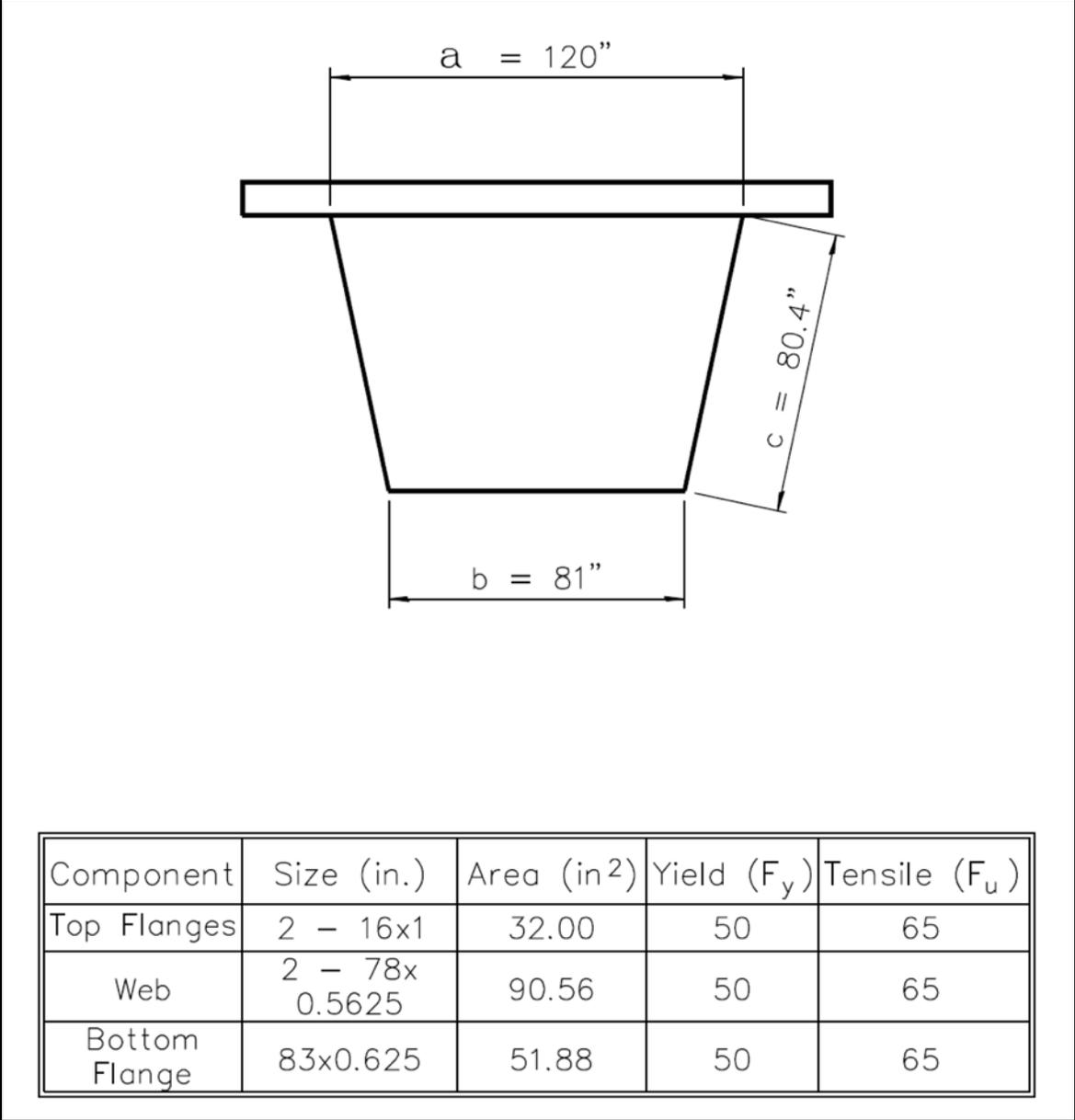


Figure B4-3: Example 4 Composite Box Cross-section, Girder 2

Resistance of Girder G2, Field Section 1 for First Cast During Concrete Deck Pour

Analysis and Methodology:

Refer to Figure B4-4, which shows the UTrAp setup for this problem, with the girder plate sizes along the three spans in elevation view, a plan view of the three spans with the cross-frames and top lateral bracing, and the concrete deck pour strips in plan view.

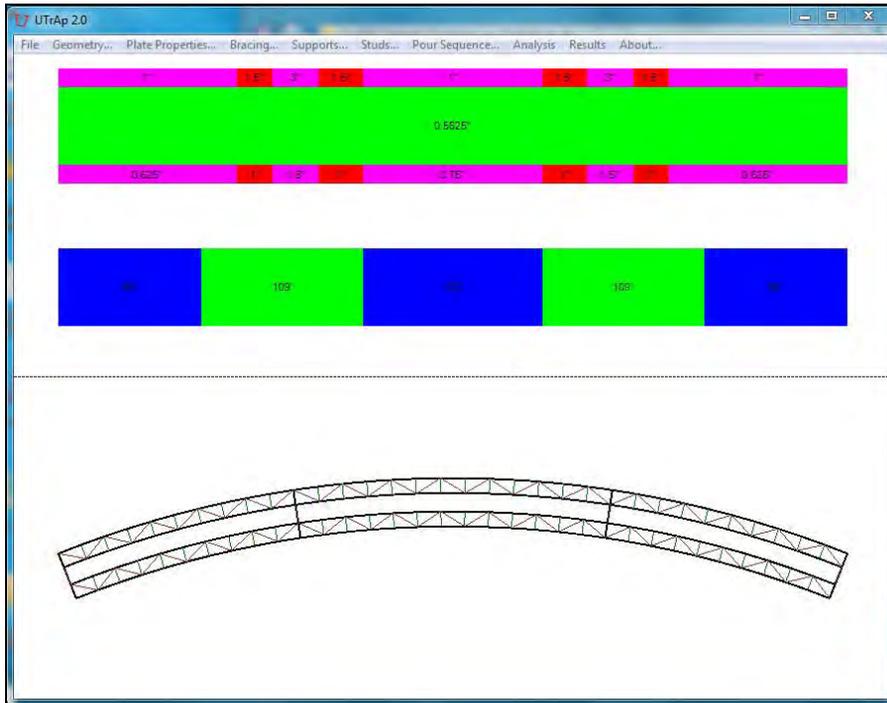


Figure B4-4: Problem Setup

Refer to Figure B4-5 for some of the input for the analysis program UTrAp.

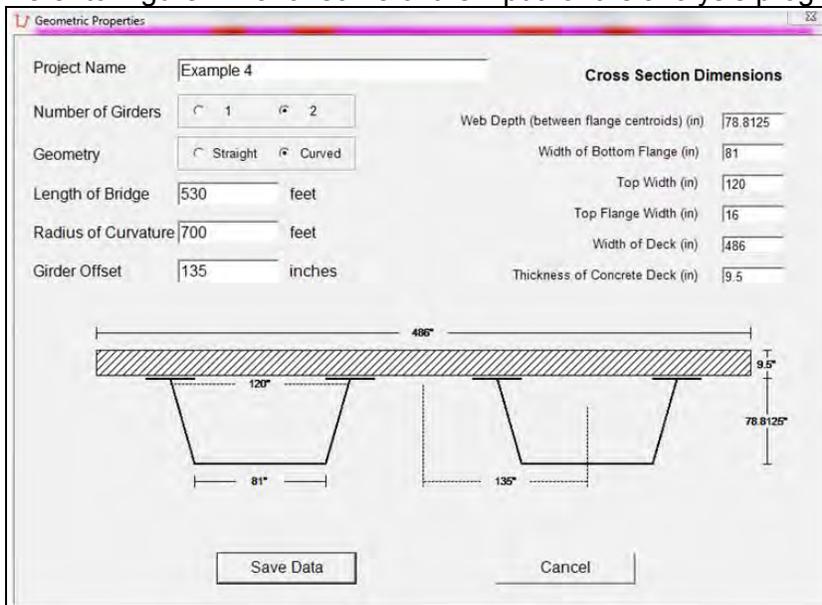
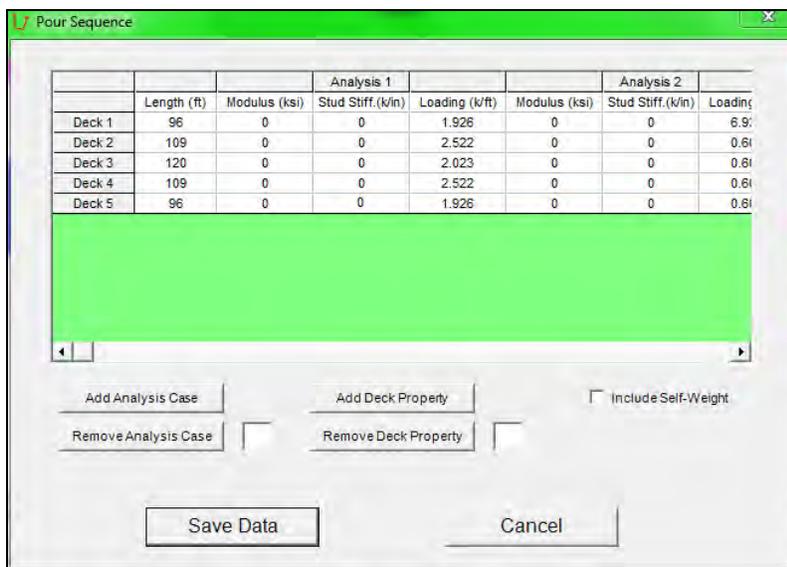


Figure B4-5: Problem Input

Resistance of Girder G2, Field Section 1 for First Cast During Concrete Deck Pour, continued

The first cast of the deck pour is modeled in UTrAp via three incremental analysis cases. The permanent dead load (steel bridge elements in all spans) corresponds to Analysis Case 1. The permanent dead load (wet concrete in Span 1) and construction dead load (formwork in all spans) correspond to Analysis Case 2.

The construction live load in Spans 2 and 3 corresponds to Analysis Case 3. Refer to Figures B4-6 and B4-7 below for the loading data for these three analysis cases. The computation of the factored line loads (DC, CDL, and CLL) input into UTrAp for each analysis case may be found at the end of this example problem.



	Length (ft)	Modulus (ksi)	Analysis 1		Analysis 2		Loading (k/ft)
			Stud Stiff. (k/in)	Loading (k/ft)	Modulus (ksi)	Stud Stiff. (k/in)	
Deck 1	96	0	0	1.926	0	0	6.9
Deck 2	109	0	0	2.522	0	0	0.61
Deck 3	120	0	0	2.023	0	0	0.61
Deck 4	109	0	0	2.522	0	0	0.61
Deck 5	96	0	0	1.926	0	0	0.61

Figure B4-6: Pour Sequence Loads (Analysis Case 1)

	Analysis 2			Analysis 3		
	Modulus (ksi)	Stud Stiff. (k/in)	Loading (k/ft)	Modulus (ksi)	Stud Stiff. (k/in)	Loading (k/ft)
Deck 1	0	0	6.932	0	0	0
Deck 2	0	0	0.608	0	0	1.215
Deck 3	0	0	0.608	0	0	1.215
Deck 4	0	0	0.608	0	0	1.215
Deck 5	0	0	0.608	0	0	1.215

Figure B4-7: Pour Sequence Loads (Analysis Cases 2 and 3)

Resistance of Girder G2, Field Section 1 for First Cast During Concrete Deck Pour, continued

Figure B4-8 shows the magnitude and location of the maximum total positive moment during the first concrete cast, which occurs for the cumulative Analysis Case 2 (i.e., Analysis Case 1 + Analysis Case 2) at 60 feet into Span 1.

Quantity	Analysis 2		Analysis 3	
	Value	Location (ft)	Value	Location (ft)
Incremental Upward Deflection (in)	2.06	230	0.34	92
Incremental Downward Deflection (in)	-5.13	68	-1.33	256
Total Upward Deflection (in)	1.14	204	0.5600001	186
Total Downward Deflection (in)	-6.2	68	-5.9	68
Incremental Rotation (rad)	0.003422	64	0.000858	256
Total Rotation (rad)	0.004148	64	0.003957	64
Incremental Shear (kips)	410.7	0	135.6	370
Incremental Positive Moment (kip-ft)	12421.7	60	2367.9	258
Incremental Negative Moment (kip-ft)	-9517.6	160	-5279.6	370
Incremental Torque (kip-ft)	873.5	0	148.7	160
Total Shear (kips)	526.5	160	609.8	160
Total Positive Moment (kip-ft)	15471.4	60	15112.3	58
Total Negative Moment (kip-ft)	-18040.8	160	-21509.2	160
Total Torque (kip-ft)	1069.5	0	1029.7	0
Incremental Positive Normal Stress (ksi)	17.53	61	4.59	387
Incremental Negative Normal Stress (ksi)	-22.95	49	-4.2	261
Incremental Shear Stress (ksi)	3.26	3	0.92	373
Total Positive Normal Stress (ksi)	21.81	61	21.27	61
Total Negative Normal Stress (ksi)	-28.6	49	-27.94	49
Total Shear Stress (ksi)	4.09	3	4.33	157
		Brace #		Brace #
Incremental Positive Top Lateral Axial Force (kips)	64.63	11	15.85	10

Figure B4-8: Maximum Positive Moment Magnitude and Location for Cumulative Analysis Case 2

Refer to Figure B4-9 below, which shows that the maximum axial stress in the exterior cross-frames is a fraction of 1 ksi. Although a design check of the interior cross-frames and the top flange lateral bracing is included in this example problem, a check of the exterior cross-frames would be superfluous.

Analysis Summary

Summary of Maximum Quantities

Quantity	Analysis 2		Analysis 3	
	Value	Location (ft)	Value	Location (ft)
Incremental Negative Top Lateral Axial Force (kips)	-83.97	1	-13.83	20
Total Positive Top Lateral Axial Force (kips)	80.3	11	77.53	11
Total Negative Top Lateral Axial Force (kips)	-103.12	1	-99.52	1
Incremental Positive Top Lateral Axial Stress (ksi)	4.616428	11	1.132143	10
Incremental Negative Top Lateral Axial Stress (ksi)	-5.997857	1	-0.9878572	20
Total Positive Top Lateral Axial Stress (ksi)	5.735714	11	5.537857	11
Total Negative Top Lateral Axial Stress (ksi)	-7.385715	1	-7.108572	1
Incremental Positive Internal Brace Axial Force (kips)	37.57	4	8.27	22
Incremental Negative Internal Brace Axial Force (kips)	-37.57	4	-8.27	22
Total Positive Internal Brace Axial Force (kips)	46.66	4	45.83	3
Total Negative Internal Brace Axial Force (kips)	-46.66	4	-45.42	4
Incremental Positive Internal Brace Axial Stress (ksi)	7.514	4	1.654	22
Incremental Negative Internal Brace Axial Stress (ksi)	-7.514	4	-1.654	22
Total Positive Internal Brace Axial Stress (ksi)	9.332	4	9.165999	3
Total Negative Internal Brace Axial Stress (ksi)	-9.332	4	-9.084	4
Incremental Positive External Brace Axial Force (kips)	0.92	1	0.18	3
Incremental Negative External Brace Axial Force (kips)	-0.92	1	-0.17	2
Total Positive External Brace Axial Force (kips)	1.13	1	1.13	2
Total Negative External Brace Axial Force (kips)	-1.13	1	-1.13	2
Incremental Positive External Brace Axial Stress (ksi)	0.115	1	0.0225	3
Incremental Negative External Brace Axial Stress (ksi)	-0.115	1	-0.02125	2
Total Positive External Brace Axial Stress (ksi)	0.14125	1	0.14125	2
Total Negative External Brace Axial Stress (ksi)	-0.14125	1	-0.14125	2

Figure B4-9: Max. External Brace Axial Stress for Cumulative Analysis Case 2

Resistance of Girder G2, Field Section 1 for First Cast During Concrete Deck Pour, continued

Refer to Figures B4-10 and B4-11, which visually represent the normal stresses adjacent to the location in Span 1 where the maximum positive moment occurs. Note that for this case no construction live load is present.

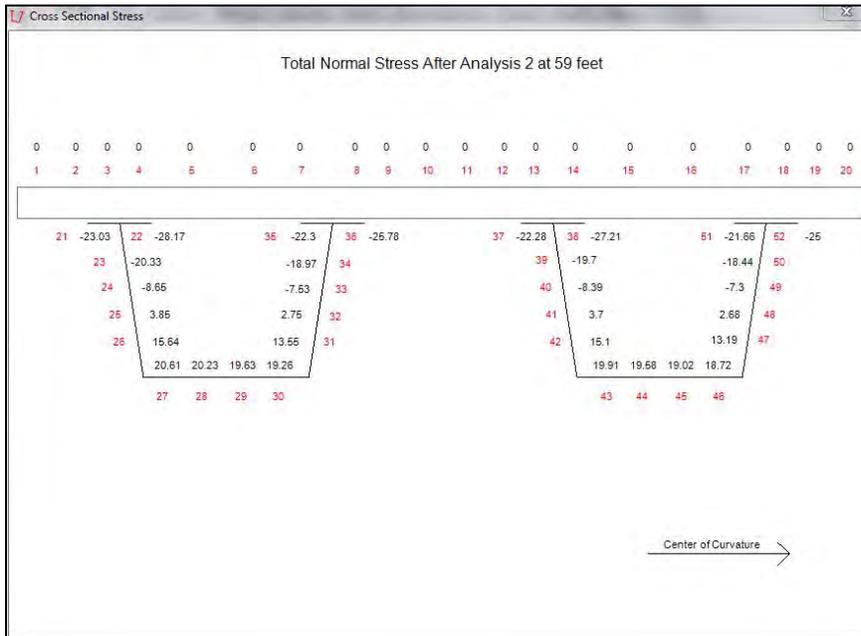


Figure B4-10: Normal Stresses Before Location of Maximum Positive Moment for Cumulative Analysis Case 2

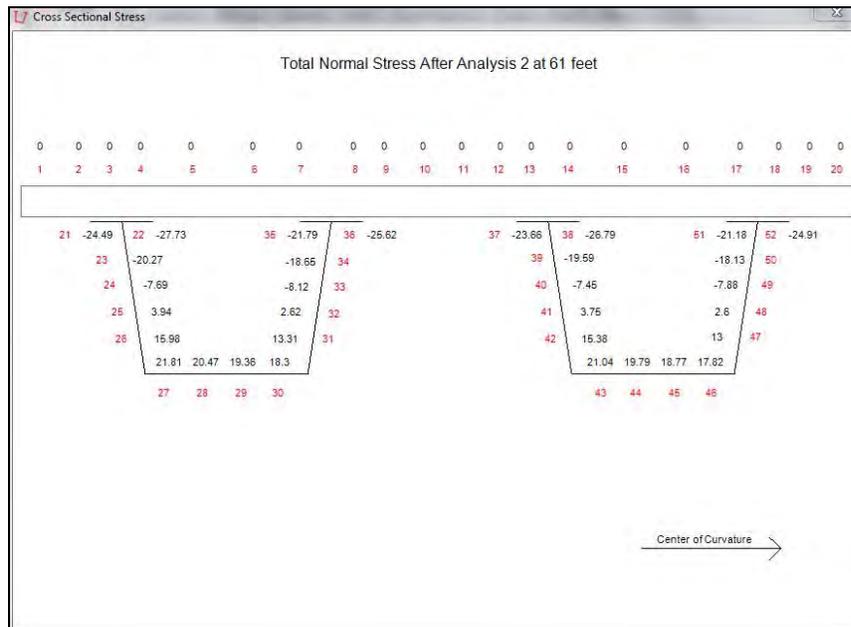


Figure B4-11: Normal Stresses Beyond Location of Maximum Positive Moment for Cumulative Analysis Case 2

Resistance of Girder G2, Field Section 1 for First Cast During Concrete Deck Pour, continued

The AASHTO equations require the stresses in the flanges to be broken up into components (due only to lateral bending, as well as without lateral bending), but most analysis programs do not provide output in this fashion. Therefore, it is necessary to either make some assumptions or otherwise manipulate the data. One approach would be to look at the normal stresses adjacent to the location where the maximum positive moment in Span 1 occurs and average the sum of the flange tip stresses (from Figures B4-10 and B4-11 above) to compute the bending stress component. This approach was adopted in Example #2 for the curved girder bridge analyzed in UT Bridge. Another approach, which is adopted in this example, would be to take the bending moment for the bridge, divide by two to get an average bending moment for the tub girders and thus (with the major axis section modulus) an average bending stress in the flanges. Based on comparing the stresses in the two girders (see Figure B4-10 for stresses at 59 feet in Span 1 or Figure B4-11 for stresses at 61 feet in Span 1), this appears to be fairly reasonable as the stress at any given location in one girder is only about 1 ksi different at the corresponding location in the other girder.

The warping stresses due to lateral bending could be computed by taking the average of the difference of the flange tip stresses; refer to Figure B4-12 for a diagram illustrating the decomposition of stresses into normal and warping. Another approach for the warping stresses, adopted in this example below, is to use approximations to account for lateral bending due to the curvature and the deck overhang brackets (this approach was also adopted for Example #2).

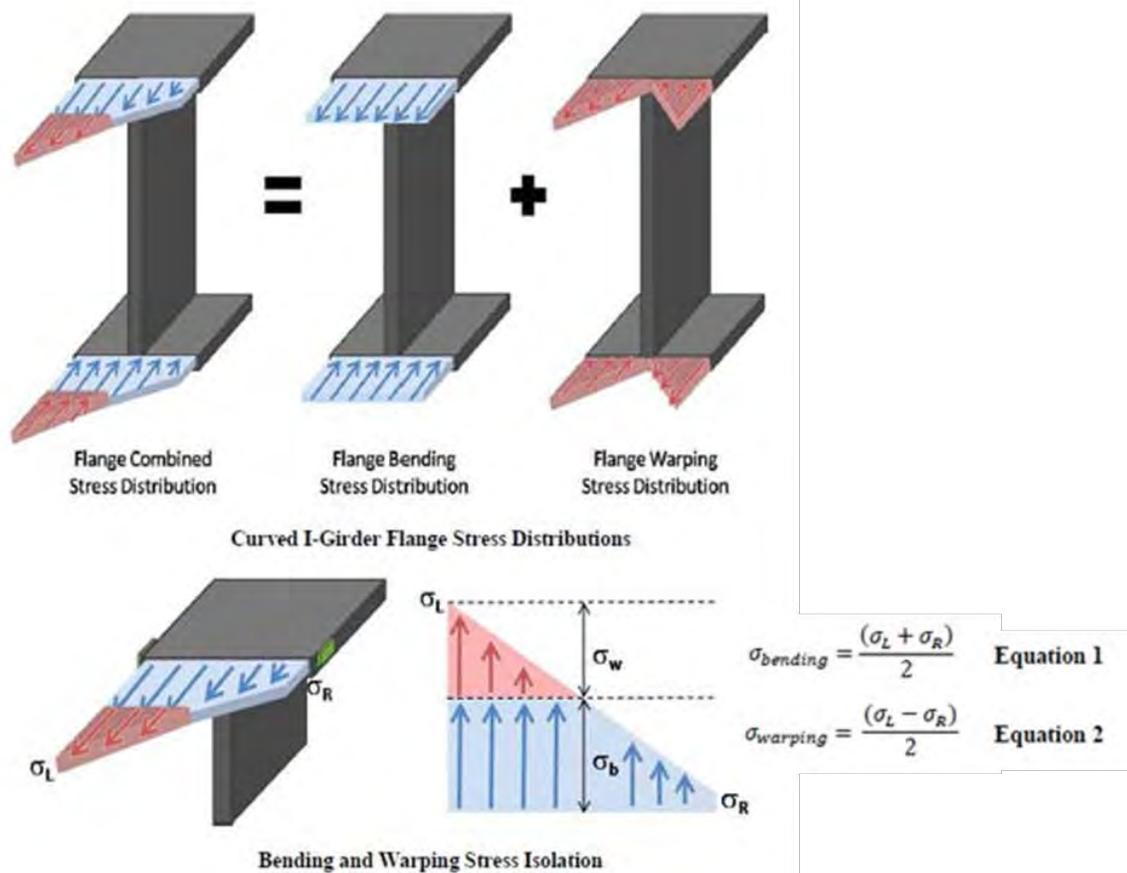


Figure B4-12: Normal And Warping Stresses In Curved I-Girders

Resistance of Girder G2, Field Section 1 for First Cast During Concrete Deck Pour, continued**Section Properties:**

$$E_s := 29000 \text{ ksi} \quad F_y := 50 \text{ ksi}$$

Flanges:

$$t_{tf} := 1 \text{ in} \quad b_{bf} := 16 \text{ in} \quad t_{bf} := \frac{5}{8} \text{ in} \quad b_{bf} := 81 \text{ in} \quad a := 10 \text{ ft} \quad \text{center-to-center of top flanges}$$

Webs:

$$t_w := \frac{9}{16} \text{ in} \quad D := 78 \text{ in} \quad \theta := \text{atan} \left[\frac{0.5(a - b_{bf})}{D} \right] = 14.04 \text{ deg inclined} \quad \text{Overall depth: } d := D + t_{tf} + t_{bf} = 79.6 \text{ in}$$

Calculated properties: Note that web inclination is not used in all inertia calculations (error is minor)

$$A_{bf} := t_{bf} \cdot b_{bf} = 50.63 \text{ in}^2 \quad y_{bf} := 0.5 \cdot t_{bf} = 0.31 \text{ in} \quad \text{Note: Ignore 1" lip on ea. side of bot. flange}$$

$$A_w := t_w \frac{D}{\cos(\theta)} = 45.23 \text{ in}^2 \quad y_w := t_{bf} + 0.5 \cdot D = 39.63 \text{ in}$$

$$A_{tf} := t_{tf} \cdot b_{bf} = 16 \text{ in}^2 \quad y_{tf} := t_{bf} + D + 0.5 \cdot t_{tf} = 79.13 \text{ in}$$

$$A_g := A_{bf} + 2A_w + 2A_{tf} = 173.08 \text{ in}^2 \quad \text{Area of girder}$$

$$Y_b := \frac{A_{bf} \cdot y_{bf} + 2A_w \cdot y_w + 2A_{tf} \cdot y_{tf}}{A_{bf} + 2A_w + 2A_{tf}} = 35.43 \text{ in} \quad \text{Centroid measured from bottom}$$

$$Y_t := d - Y_b = 44.2 \text{ in} \quad \text{Centroid measured from top}$$

Major axis moment of inertia:

$$I_x := \frac{b_{bf} \cdot t_{bf}^3}{12} + \frac{2t_w \cdot D^3}{12} + \frac{2b_{bf} \cdot t_{bf}^3}{12} + A_{bf} \cdot (Y_b - y_{bf})^2 + 2A_w \cdot (Y_b - y_w)^2 + 2A_{tf} \cdot (Y_b - y_{tf})^2 = 169614.1 \text{ in}^4$$

$$S_{bx} := \frac{I_x}{Y_b} = 4787.4 \text{ in}^3 \quad \text{Major axis elastic section modulus to bottom}$$

$$S_{tx} := \frac{I_x}{Y_t} = 3837.8 \text{ in}^3 \quad \text{Major axis elastic section modulus to top}$$

$$I_y := \frac{t_{bf} \cdot b_{bf}^3}{12} + \frac{2D \cdot t_w^3}{12} + \frac{2t_{tf} \cdot b_{bf}^3}{12} + 2A_w \left[\frac{0.5(a + b_{bf})}{2} \right]^2 + 2A_{tf} \left(\frac{a}{2} \right)^2 = 371957.7 \text{ in}^4 \quad \text{Minor axis moment of inertia}$$

$$S_y := \frac{I_y}{0.5 \cdot \max(b_{bf}, a + b_{bf})} = 5470 \text{ in}^3 \quad \text{Minimum minor axis elastic section modulus}$$

Applied Factored Forces:

$$\text{Major-axis Bending: } M_{ux} := \frac{15471.4 \text{ kip ft}}{2} = 7735.7 \text{ kip ft} \quad \text{per tub girder}$$

$$\text{Unbraced Length: } L_b := \frac{162.57 \text{ ft}}{10} = 16.257 \text{ ft} \quad \text{with cross-frame spacing of approximately 16 ft}$$

Lateral Flange Bending:

$$\text{Radius of curvature } R := 700 \text{ ft} + \frac{22.5 \text{ ft}}{2} - \frac{a}{2} = 706.25 \text{ ft} \quad \text{at Girder G2 top flange} \quad \text{Constant } N := 10$$

$$\text{Moment in each top flange due to lateral bending from curvature } M_{lat} := \frac{(0.5M_{ux}) \cdot L_b^2}{N \cdot R \cdot D} \quad (\text{Eq. C4.6.1.2.4b-1})$$

Note: moment in bot. flange will be approx. twice this value (R is more)

Resistance of Girder G2, Field Section 1 for First Cast During Concrete Deck Pour, continued

$M_{lat} = 22.3$ kip ft Note that this value is already factored since the major-axis moment was factored
 Minor-axis Bending: $M_{uy} := 0$ kip ft Assume wind load on girders is negligible during deck pour

Overhang Bracket Forces at Fascia Side of Girder:

Assume bracket loads are applied uniformly along girder although actual bracket spacing = 3 ft

$$\text{overhang} := \frac{40.5 \text{ ft} - 22.5 \text{ ft} - a}{2} = 4 \text{ ft} \text{ at top flange fascia } x := \frac{(a - b_{bf})}{2} = 19.5 \text{ in} \text{ more at bot. flange}$$

Assume 10" thick deck at fascia overhang; $\frac{1}{2}$ of weight goes to bracket and $\frac{1}{2}$ directly to girder

$$w_{fascia} := 0.5 \text{ overhang } 10 \text{ in} \cdot 150 \text{ pcf} \quad w_{fascia} = 250 \text{ plf} \quad \text{Concrete weight to bracket}$$

$$\text{Assume } 200 \text{ plf for screed rail at fascia overhang } w_{rail} := 200 \text{ plf} \quad \text{Rail weight to bracket}$$

Assume 10 psf for forms and another 5 psf for bracket components and miscellaneous at fascia overhang; $\frac{1}{2}$ of weight goes to bracket and $\frac{1}{2}$ directly to girder

$$w_{forms} := 0.5 \cdot \text{overhang} \cdot (10 \text{ psf} + 5 \text{ psf}) \quad w_{forms} = 30 \text{ plf} \quad \text{Form weight to bracket}$$

$$w_{bracket} := w_{fascia} + w_{rail} + w_{forms} \quad w_{bracket} = 480 \text{ plf} \quad \text{Total uniform load to bracket}$$

$$\alpha := \text{atan} \left(\frac{D}{\text{overhang} + x} \right) \quad \alpha = 49.13 \text{ deg} \quad \text{Angle of bracket relative to horizontal}$$

$$F_L := \frac{w_{bracket}}{\tan(\alpha)} = 415.4 \text{ plf} \quad \text{Lateral force on each flange from bracket}$$

$$\text{Lat. Moment in Ea. Flange from Overhang Bracket at Fascia Side of Girder: } M_L := \frac{F_L \cdot L_b^2}{12} \text{ (Eq. C6.10.3.4-2)}$$

Factored Lat. Moment in Ea. Girder Flange from Overhang Bracket at Fascia: $M_{Lu} := 1.50M_L = 13.7$ kip ft
 (conservatively assume highest load factor corresponding to all construction dead load)

Flexural Resistance

$$\text{Top flange is in compression} \quad b_{fc} := b_{tf} \quad t_{fc} := t_{tf}$$

$$\text{Bottom flange is in tension} \quad b_{ft} := b_{bf} \quad t_{ft} := t_{bf}$$

Applied Stresses:

$$\text{Top flange is in compression: } S_{yc} := \frac{t_{fc} \cdot b_{fc}^2}{6} \quad S_{yc} = 42.7 \text{ in}^3$$

$$\text{Bottom flange is in tension: } S_{yt} := \frac{t_{ft} \cdot b_{ft}^2}{6} \quad S_{yt} = 683.4 \text{ in}^3$$

Stress in compression flange without consideration of lateral bending:

$$f_{buc} := \frac{M_{ux}}{S_{tx}} \quad f_{buc} = 24.2 \text{ ksi}$$

First-order stress due to lateral bending

in compression flange:

$$f_{L1c} := \frac{0.5 \left(\frac{M_{uy}}{2} \right) + M_{lat} + M_{Lu}}{S_{yc}} \quad f_{L1c} = 10.1 \text{ ksi}$$

Stress in tension flange without consideration of lateral bending:

$$f_{but} := \frac{M_{ux}}{S_{bx}} \quad f_{but} = 19.4 \text{ ksi}$$

First-order stress due to lateral bending

in tension flange:

$$f_{Lt} := \frac{0.5M_{uy} + 2M_{lat} + M_{Lu}}{S_{yt}} \quad f_{Lt} = 1 \text{ ksi}$$

Resistance of Girder G2, Field Section 1 for First Cast During Concrete Deck Pour, continued*Flange Strength Reduction Factors:*

Hybrid Factor (AASHTO 6.10.1.10.1)

Since the flexural member is a homogenous built-up section, the hybrid factor shall be taken as unity

$$R_h = 1$$

Web Load Shedding Factor (AASHTO 6.10.1.10.2)

$$\text{Depth of web in compression} \quad D_c := \frac{(Y_t - t_{tf})}{\cos[\theta]} = 44.53 \text{ in measured along incline}$$

$$\lambda_{rw} := \left(5.7 \cdot \sqrt{\frac{E_s}{F_y}} \right) \quad \lambda_{rw} = 137.3 \quad \frac{2 \cdot D_c}{t_w} = 158.3$$

$$a_{wc} := \frac{2D_c \cdot t_w}{b_{fc} \cdot t_{fc}}$$

$$R_b := \begin{cases} 1.0 & \text{if } \frac{2 \cdot D_c}{t_w} \leq \lambda_{rw} \\ \left[1.0 - \left(\frac{a_{wc}}{1200 + 300 \cdot a_{wc}} \right) \cdot \left(\frac{2 \cdot D_c}{t_w} - \lambda_{rw} \right) \right] & \text{otherwise} \end{cases} \quad R_b = 0.97$$

Web is slender, so Eq. 6.10.3.2.1-3 (web bend-buckling) must be checked per AASHTO 6.10.3.2.1. For constructability checks, per AASHTO 6.10.3.2.1, use $R_b := 1$

Local Buckling Resistance (AASHTO 6.10.8.2.2):

Slenderness ratio of the compression flange

$$\lambda_f := \frac{b_{fc}}{2 \cdot t_{fc}} \quad \lambda_f = 8 \quad (\text{Eq. 6.10.8.2.2-3})$$

$$\lambda_{pf} := 0.38 \cdot \sqrt{\frac{E_s}{F_y}} \quad \lambda_{pf} = 9.2 \quad (\text{Eq. 6.10.8.2.2-4})$$

$$\lambda_{rf} := 0.56 \cdot \sqrt{\frac{E_s}{0.7F_y}} \quad \lambda_{rf} = 16.1 \quad (\text{Eq. 6.10.8.2.2-5})$$

Local Buckling Resistance

$$F_{nc1} := \begin{cases} (R_b \cdot R_h \cdot F_y) & \text{if } \lambda_f \leq \lambda_{pf} \\ \left[1 - \left(1 - \frac{0.7F_y}{R_h \cdot F_y} \right) \cdot \frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}} \right] \cdot R_b \cdot R_h \cdot F_y & \text{otherwise} \end{cases} \quad (\text{Eq. 6.10.8.2.2-1})$$

$$F_{nc1} = 50 \text{ ksi}$$

$$(\text{Eq. 6.10.8.2.2-2})$$

Resistance of Girder G2, Field Section 1 for First Cast During Concrete Deck Pour, continued**Lateral Torsional Buckling Resistance (AASHTO 6.10.8.2.3):**Unbraced length $L_b = 16.257$ ft with cross-framing spacing of approximately 16 ft

Effective Radius

$$\text{of Gyration} \quad r_t := \frac{b_{fc}}{\sqrt{12 \cdot \left(1 + \frac{1}{3} \cdot \frac{D_c \cdot t_w}{b_{fc} \cdot t_{fc}}\right)}} \quad r_t = 3.7 \text{ in.} \quad (\text{Eq. 6.10.8.2.3-9})$$

Limiting Unbraced Length Calculations:

$$L_p := r_t \cdot \sqrt{\frac{E_s}{F_y}} \quad L_p = 7.5 \text{ ft.} \quad (\text{Eq. 6.10.8.2.3-4})$$

$$L_r := \pi r_t \cdot \sqrt{\frac{E_s}{0.7 F_y}} \quad L_r = 28.2 \text{ ft.} \quad (\text{Eq. 6.10.8.2.3-5})$$

Per D-4, Moment gradient modifier $C_b := 1$ since, $\frac{f_{mid}}{f_2} > 1$

Lateral Torsional Buckling Resistance

$$F_{nc2} := \begin{cases} (R_b \cdot R_h \cdot F_y) & \text{if } L_b \leq L_p \\ \left[C_b \cdot \left[1 - \left(1 - \frac{0.7 F_y}{R_h \cdot F_y} \right) \cdot \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \cdot R_b \cdot R_h \cdot F_y \right] & \text{if } L_p < L_b \leq L_r \\ \frac{C_b \cdot R_b \cdot \pi^2 \cdot E_s}{\left(\frac{L_b}{r_t} \right)^2} & \text{otherwise} \end{cases} \quad (\text{Eq. 6.10.8.2.3-1})$$

$$F_{nc2} = 43.7 \text{ ksi}$$

$$(\text{Eq. 6.10.8.2.3-2})$$

$$(\text{Eq. 6.10.8.2.3-3})$$

Controlling Nominal Flexure Resistance:

$$F_{nc} := \min(F_{nc1}, F_{nc2}) \quad F_{nc} = 43.7 \text{ ksi}$$

$$\text{Resistance factor} \quad \Phi_f := 1.0 \quad (\text{AASHTO 6.5.4.2})$$

$$\Phi_f \cdot F_{nc} = 43.7 \text{ ksi}$$

$$M_{rx} := \Phi_f \cdot F_{nc} \cdot S_{tx} \quad M_{rx} = 13964.8 \text{ kip ft}$$

$$\frac{M_{ux}}{M_{rx}} = 0.55 \quad \text{Adequate resistance for lateral-torsional buckling}$$

Resistance of Girder G2, Field Section 1 for First Cast During Concrete Deck Pour, continued

Determine Stress due to Lateral Bending:

First-order lateral bending stress (from previous): $f_{L1c} = 10.1$ ksi

Limiting unbraced length for first-order lateral bending stress

$$1.2 \cdot L_p \cdot \sqrt{\frac{C_b \cdot R_b}{\frac{f_{buc}}{F_y}}} = 13 \text{ ft} \quad (\text{Eq. 6.10.1.6-2})$$

Lateral bending stress:
elastic lateral torsional buckling stress

$$F_{cr} := \frac{C_b \cdot R_b \cdot \pi^2 \cdot E_s}{\left(\frac{L_b}{r_t}\right)^2} \quad F_{cr} = 105.4 \text{ ksi} \quad (\text{Eq. 6.10.8.2.3-8})$$

Check if the first-order stress needs to be amplified:

Approximated second-order lateral bending stress=

$$f_{Lc} = 11.2 \text{ ksi}$$

$$f_{Lc} := \begin{cases} f_{Lc1} & \text{if } L_b \leq 1.2 \cdot L_p \cdot \sqrt{\frac{C_b \cdot R_b}{\frac{f_{buc}}{F_y}}} \\ \left(\frac{0.85}{1 - \frac{f_{buc}}{F_{cr}}} \right) \cdot f_{L1c} & \text{otherwise} \end{cases} \quad (\text{Eq. 6.10.1.6-4})$$

Lateral bending check:

Lateral_Bending_Resistance := $\begin{cases} \text{"Lateral Bending Requirements Satisfied"} & \text{if } f_{Lc} \leq 0.6F_y \\ \text{"Girder is NOT Adequate in Lateral Flexure"} & \text{otherwise} \end{cases}$

Lateral_Bending_Resistance = "Lateral Bending Requirements Satisfied"

Overall Flexural Resistance Check

The following must be satisfied:

$$f_{buc} + \frac{1}{3} \cdot f_{Lc} \leq \phi_f \cdot F_{nc} \quad (\text{Eq. 6.10.3.2.1-2})$$

$$f_{buc} + \frac{1}{3} \cdot f_{Lc} = 27.9 \text{ ksi} \quad \phi_f \cdot F_{nc} = 43.7 \text{ ksi}$$

Resistance_Check := $\begin{cases} \text{"Girder Meets AASHTO Flexural Requirements"} & \text{if } f_{buc} + \frac{1}{3} \cdot f_{Lc} \leq \phi_f \cdot F_{nc} \\ \text{"Girder is NOT Adequate in Lateral Flexure"} & \text{otherwise} \end{cases}$

Resistance_Check = "Girder Meets AASHTO Flexural Requirements"

Resistance of Girder G2, Field Section 1 for First Cast During Concrete Deck Pour, continued**Constructibility Check:**

The following must be satisfied:

$$f_{\text{buc}} + f_{\text{Lc}} \leq \Phi_f \cdot R_h \cdot F_y \quad (\text{Eq. 6.10.3.2.1-1})$$

$$f_{\text{buc}} + f_{\text{Lc}} = 35.4 \text{ ksi} < \Phi_f \cdot R_h \cdot F_y = 50 \text{ ksi}$$

Adequate resistance in compression flange

The following must be satisfied:

$$f_{\text{but}} + f_{\text{Lt}} \leq \Phi_f \cdot R_h \cdot F_y \quad (\text{Eq. 6.10.3.2.2-1})$$

$$f_{\text{but}} + f_{\text{Lt}} = 20.4 \text{ ksi} < \Phi_f \cdot R_h \cdot F_y = 50 \text{ ksi}$$

Adequate resistance in tension flange

Web Bend-Buckling Resistance without Longitudinal Stiffeners (AASHTO 6.10.1.9.1):

$$\text{Web Depth } D_i := \frac{D}{\cos(\theta)} = 80.4 \text{ in} \quad \text{measured along incline}$$

$$k := \frac{9}{\left(\frac{D_c}{D}\right)^2} = 29.35 \quad (\text{Eq. 6.10.1.9.1-2})$$

$$F_{\text{crw}} := \frac{0.9 \cdot E_s \cdot k}{\left(\frac{D}{t_w}\right)^2} = 37.5 \text{ ksi} \quad (\text{Eq. 6.10.1.9.1-1})$$

$$F_{\text{crw}} := \min\left(F_{\text{crw}}, R_h \cdot F_y, \frac{F_y}{0.7}\right) = 37.5 \text{ ksi}$$

Constructability Check:

$$F_{\text{buc}} \leq \Phi_f \cdot F_{\text{crw}} \quad (\text{Eq. 6.10.3.2.1-3})$$

$$F_{\text{buc}} = 24.2 \text{ ksi} < \Phi_f \cdot F_{\text{crw}} = 37.5 \text{ ksi}$$

Adequate resistance in web

Resistance of the Brace for Internal K-Frame

Axial Tension Resistance of the Brace L6x6x7/16 (AASHTO 6.8.2):

$P_{ut} := 46.66$ kip Maximum tension force in any internal brace member from analysis model.

Section Properties (Taken from AISC manual) $F_y := 36$ ksi $F_u := 58$ ksi $A_g := 5.08$ in²
From Figure B2-19, it appears that the brace is welded rather than bolted to the conn. Plate, so $A_n := A_g$

For shear lag reduction factor U, lacking weld details, assume $U := 0.60$ (0.5 is worst case per 6.8.2.2)

Resistance factors $\Phi_y := 0.95$ $\Phi_u := 0.80$ (AASHTO 6.5.4.2)

Tension resistance for yielding $P_{ry} := \Phi_y \cdot F_y \cdot A_g = 173.7$ kip (Eq. 6.8.2.1-1)

Tension resistance for fracture $P_{ru} := \Phi_u \cdot F_u \cdot A_n \cdot U = 141.4$ kip (Eq. 6.8.2.1-2)

Axial Resistance Check $P_{ut} = 46.7$ kip $<$ $P_{rt} := \min(P_{ry}, P_{ru}) = 141.4$ kip Tension resistance is adequate

Axial Compression Resistance of the Brace L6x6x7/16 (AASHTO 6.9.4):

$P_{uc} := 46.66$ kip Maximum compression force in any internal brace member from analysis model.

Section Properties (Taken from AISC manual) $E_s := 29000$ ksi $F_y := 36$ ksi $A_g := 5.08$ in²

Check Slenderness of the Member (Sect. 6.9.4.2)

Following requirements needs to be satisfied for the element to qualify as nonslender:

$$\frac{b}{t} \leq k \sqrt{\frac{E}{F_y}} \quad (\text{Eq. 6.9.4.2.1-1}) \quad k := 0.45 \quad \text{from Table 6.9.4.2.1-1}$$

angle leg:

$$b := 6 \text{ in} \quad t := \frac{7}{16} \text{ in} \quad \frac{b}{t} = 13.7 \quad k \sqrt{\frac{E_s}{F_y}} = 12.8$$

leg check:

Find Q if element is slender:

$$Q_s := \begin{cases} 1.34 - 0.76 \left(\frac{b}{t} \right) \cdot \sqrt{\frac{F_y}{E_s}} & \text{if } \frac{b}{t} \leq 0.91 \cdot \sqrt{\frac{E_s}{F_y}} \\ \frac{0.53 \cdot E_s}{F_y \cdot \left(\frac{b}{t} \right)^2} & \text{otherwise} \end{cases} \quad (\text{Eq. 6.9.4.2.2-5})$$

(Eq. 6.9.4.2.2-6)

$$Q_s = 0.97$$

$$Q_s := \begin{cases} 1.0 & \text{if } \frac{b}{t} \leq 0.45 \cdot \sqrt{\frac{E_s}{F_y}} \\ Q_s & \text{otherwise} \end{cases}$$

$$Q = 0.97$$

Resistance of the Brace for Internal K-Frame, continued

Determine Effective Slenderness Ratio $(KL/r)_{\text{eff}} = \lambda_{\text{eff}}$ (Sect. 6.9.4.4)

$$\text{Recall } a = 120 \text{ in and } b_{\text{br}} = 81 \text{ in and } D = 78 \text{ in} \quad L := \max \left[a, \sqrt{\left(\frac{b_{\text{br}}}{2} \right)^2 + D^2} \right]$$

Strut rather than diagonal governs the brace length, so $L = 120$ in

$$R_x := 1.86 \text{ in} \quad \text{so} \quad \frac{L}{r_x} = 64.5$$

$$\lambda_{\text{eff}} := \begin{cases} \left(72 + 0.75 \frac{L}{r_x} \right) & \text{if } \frac{L}{r_x} \leq 80 \\ \left(32 + 1.25 \frac{L}{r_x} \right) & \text{otherwise} \end{cases} \quad (\text{Eq. 6.9.4.4-1})$$

$$\lambda_{\text{eff}} = 120.4 \quad (\text{Eq. 6.9.4.4-2})$$

Limiting KL/r for secondary compression members $\lambda_{\text{limit}} := 120$ (Sect. 6.9.3)

Maximum actual slenderness corresponds to minor principal axis buckling $r_z := 1.18$ in

$K := 1$ for single angles (Sect 4.6.2.5)

$$\frac{K \cdot L}{r_z} = 101.7 < \lambda_{\text{limit}} = 120 \quad \text{Therefore, actual maximum slenderness ratio is adequate}$$

Flexural Buckling Resistance

$$P_e := \frac{\pi^2 \cdot E_s}{(\lambda_{\text{eff}})^2} \cdot A_g \quad P_e = 100.3 \text{ kip} \quad (\text{Eq. 6.9.4.1.2-1})$$

Since the various conditions for single-angle members are satisfied as enumerated in AASHTO LRFD Sect. 6.9.4.4, the effective slenderness ratio can be calculated per that section; therefore, only flexural buckling resistance will be used to determine nominal compressive resistance of the brace. The effect of the eccentricities can be neglected when evaluated in this manner.

Equivalent Nominal Yield Resistance

$$P_o := Q \cdot F_y \cdot A_g \quad P_o = 177.9 \text{ kip} \quad (\text{Sect. 6.9.4.1.1})$$

$$\frac{P_e}{P_o} = 0.56$$

Nominal Compressive Resistance

$$P_n := \begin{cases} \left[0.658 \left(\frac{P_o}{P_e} \right) \right] \cdot P_o & \text{if } \frac{P_e}{P_o} \geq 0.44 \\ (0.877 P_e) & \text{otherwise} \end{cases} \quad (\text{Eq. 6.9.4.1.1-1})$$

$$P_n = 84.7 \text{ kip} \quad (\text{Eq. 6.9.4.1.1-2})$$

Resistance factor $\Phi_c := 0.9$ (AASHTO 6.5.4.2)

$$\text{Factored Axial Resistance} \quad P_{\text{rc}} := \Phi_c \cdot P_n \quad P_{\text{rc}} = 76.2 \text{ kip} \quad (\text{Eq. 6.9.2.1-1})$$

$$\text{Axial Resistance Check} \quad P_{\text{uc}} = 46.7 \text{ kip} < P_{\text{rc}} = 76.2 \text{ kip} \quad \text{Compression resistance is adequate}$$

Resistance of the Brace for Top Flange Laterals**Axial Tension Resistance of the Brace WT9x48.5 (AASHTO 6.8.2):**

$P_{ut} := 80.30$ kip Maximum tension force in any top flange lateral brace member from analysis model.

Section Properties (Taken from AISC manual) $F_y := 50$ ksi $F_u := 65$ ksi $A_g := 14.3$ in² $b_f := 11.1$ in

Assume that the brace is bolted rather than welded to the girder top flange, so assume $A_n := 0.85 A_g$

For shear lag reduction factor U, assume WT flange has 3+ fasteners, so $U := 0.9$ (Table 6.8.2.2-1)

Resistance factors $\Phi_y := 0.95$ $\Phi_u := 0.80$ (AASHTO 6.5.4.2)

Tension resistance for yielding $P_{ry} := \Phi_y \cdot F_y \cdot A_g = 679.3$ kip (Eq. 6.8.2.1-1)

Tension resistance for fracture $P_{ru} := \Phi_u \cdot F_u \cdot A_n \cdot U = 568.9$ kip (Eq. 6.8.2.1-2)

Axial Resistance Check $P_{ut} = 80.3$ kip $<$ $P_{rt} := \min(P_{ry}, P_{ru}) = 568.9$ kip Tension resistance is adequate

Axial Compression Resistance of the Brace WT9x48.5 (AASHTO 6.9.4):

$P_{uc} := 103.12$ kip Max. compression force in any top flange lateral brace member from analysis model.

Section Properties (Taken from AISC manual)

$E_s := 29000$ ksi $F_y := 50$ ksi $A_g := 14.3$ in² $d := 9.3$ in

Check Slenderness of the Member (Sect 6.9.4.2)

Following requirement needs to be satisfied for the element to qualify as nonslender:

$$\frac{b}{t} \leq k \sqrt{\frac{E}{F_y}} \quad (\text{Eq. 6.9.4.2.1-1})$$

flange: $k_{\text{flange}} := 0.56$ from Table 6.9.4.2.1-1

stem: $k_{\text{stem}} := 0.75$ from Table 6.9.4.2.1-1

flange: $b_{\text{flange}} := \frac{b_f}{2} = 5.6$ in $t_{\text{flange}} := 0.870$ $\frac{b_{\text{flange}}}{t_{\text{flange}}} = 6.4$ $k_{\text{flange}} \cdot \sqrt{\frac{E_s}{F_y}} = 13.5$

stem: $b_{\text{stem}} := d = 9.3$ in $t_{\text{stem}} := 0.535$ in $\frac{b_{\text{stem}}}{t_{\text{stem}}} = 17.4$ $k_{\text{stem}} \cdot \sqrt{\frac{E_s}{F_y}} = 18.1$

flange check:

Find Q if element is slender:

$$Q_{s_flange} := \begin{cases} \left[1.415 - 0.74 \left(\frac{b_{\text{flange}}}{t_{\text{flange}}} \right) \cdot \sqrt{\frac{F_y}{E_s}} \right] & \text{if } \frac{b_{\text{flange}}}{t_{\text{flange}}} \leq 1.03 \cdot \sqrt{\frac{E_s}{F_y}} \\ \frac{0.69 \cdot E_s}{F_y \cdot \left(\frac{b_{\text{flange}}}{t_{\text{flange}}} \right)^2} & \text{otherwise} \end{cases} \quad (\text{Eq. 6.9.4.2.2-1})$$

$$Q_{s_flange} = 1.22 \quad (\text{Eq. 6.9.4.2.2-2})$$

$$Q_{\text{flange}} := \begin{cases} 1.0 & \text{if } \frac{b_{\text{flange}}}{t_{\text{flange}}} \leq 0.56 \cdot \sqrt{\frac{E_s}{F_y}} \\ Q_{s_flange} & \text{otherwise} \end{cases}$$

$$Q_{\text{flange}} = 1$$

Resistance of the Brace for Top Flange Laterals, continued

$$Q_{s_stem} := \begin{cases} \left[1.908 - 1.22 \left(\frac{b_{stem}}{t_{stem}} \right) \cdot \sqrt{\frac{F_y}{E_s}} \right] \text{ if } \frac{b_{stem}}{t_{stem}} \leq 1.03 \cdot \sqrt{\frac{E_s}{F_y}} \\ \frac{0.69 \cdot E_s}{F_y \cdot \left(\frac{b_{stem}}{t_{stem}} \right)^2} \text{ otherwise} \end{cases} \quad (\text{Eq. 6.9.4.2.2-3})$$

(Eq. 6.9.4.2.2-4)

$$Q_{s_stem} = 1.03$$

$$Q_{stem} := \begin{cases} 1.0 \text{ if } \frac{b_{stem}}{t_{stem}} \leq 0.75 \cdot \sqrt{\frac{E_s}{F_y}} \\ Q_{stem} \text{ otherwise} \end{cases}$$

$$Q_{stem} = 1$$

$$Q := \min(Q_{flange}, Q_{stem}) = 1 \quad (\text{Sect. 6.9.4.2.2})$$

Determine Slenderness Ratio $(KL/r) = \lambda$

$$\text{Recall } L_b = 195.1 \text{ in and } a = 120 \text{ in} \quad \text{Therefore} \quad L := \sqrt{a^2 + L_b^2} = 229 \text{ in}$$

$$r_x := 2.56 \text{ in} \quad \text{and} \quad r_y := 2.65 \text{ in}$$

$$K := 0.75 \quad \text{for bolted or welded ends} \quad (\text{Sect. 4.6.2.5})$$

$$\lambda := \frac{K \cdot L}{\min(r_x, r_y)} = 67.1$$

$$\text{Limiting } KL/r \text{ for primary compression members} \quad \lambda_{limit} := 120 \quad (\text{Sect. 6.9.3})$$

$$\lambda = 67.1 < \lambda_{limit} = 120 \quad \text{Therefore, maximum slenderness ratio is adequate}$$

$$P_{e_FB} := \frac{\pi^2 \cdot E_s}{(\lambda)^2} \cdot A_g \quad P_{e_FB} = 909 \text{ kip} \quad (\text{Eq. 6.9.4.1.2-1})$$

Flexural Torsional Buckling Resistance

$$K_z := 1 \quad \text{for torsional buckling (Commentary Sect. C6.9.4.1.3)}$$

$$L_z := L = 229 \text{ in} \quad \text{for torsional buckling (Commentary Sect. C6.9.4.1.3)}$$

$$K_y := 0.75 \quad \text{for bolted or welded ends} \quad (\text{Sect. 4.6.2.5})$$

$$L_y := L = 229 \text{ in}$$

$$\text{Shear modulus of elasticity} \quad G := 0.385 \cdot E_s = 11165 \text{ ksi}$$

Resistance of the Brace for Top Flange Laterals, continued

Additional Cross-Sectional Properties (Taken from AISC manual)

$$\text{Major axis moment of inertia } I_x := 93.8 \text{ in}^4$$

$$\text{Minor axis moment of inertia } I_y := 100 \text{ in}^4$$

$$\text{Warping torsional constant } C_w := 9.29 \text{ in}^6$$

$$\text{St. Venant torsional constant } J := 2.92 \text{ in}^4$$

Distance along y-axis between shear center and centroid

$$y_o := 1.91 \text{ in} - \frac{t_{\text{flange}}}{2} = 1.475 \text{ in}$$

Polar radius of gyration about shear center

$$r_o := \sqrt{y_o^2 + \frac{I_x + I_y}{A_g}} = 3.97 \text{ in} \quad (\text{Eq. 6.9.4.1.3-6})$$

$$H := 1 - \frac{y_o^2}{r_o^2} = 0.86 \quad (\text{Eq. 6.9.4.1.3-3})$$

$$P_{ey} := \frac{\pi^2 \cdot E_s}{\left(\frac{K_y \cdot L_y}{r_y}\right)^2} \cdot A_g = 974.1 \text{ kip} \quad (\text{Eq. 6.9.4.1.3-4})$$

$$P_{ez} := \left[\frac{\pi^2 \cdot E_s \cdot C_w}{(K_z \cdot L_z)^2} + G \cdot J \right] \cdot \frac{1}{r_o^2} = 2076.1 \text{ kip} \quad (\text{Eq. 6.9.4.1.3-5})$$

$$P_{e_FTB} := \left(\frac{P_{ey} + P_{ez}}{2H} \right) \cdot \left[1 - \sqrt{1 - \frac{4 \cdot P_{ey} \cdot P_{ez} \cdot H}{(P_{ey} + P_{ez})^2}} \right] = 883.5 \text{ kip} \quad (\text{Eq. 6.9.4.1.3-2})$$

Governing Elastic Critical Buckling Resistance

$$P_e := \min(P_{e_FB}, P_{e_FTB}) = 883.5 \text{ kip}$$

Equivalent Nominal Yield Resistance

$$P_o := Q \cdot F_y \cdot A_g \quad P_o = 715 \text{ kip} \quad (\text{Sect. 6.9.4.1.1})$$

$$\frac{P_e}{P_o} = 1.24$$

Nominal Compressive Resistance

$$P_n := \begin{cases} 0.658 \left(\frac{P_o}{P_e}\right) \cdot P_o & \text{if } \frac{P_e}{P_o} \geq 0.44 \\ (0.877 P_e) & \text{otherwise} \end{cases} \quad (\text{Eq. 6.9.4.1.1-1})$$

$$P_n = 509.6 \text{ kip}$$

(Eq. 6.9.4.1.1-2)

Resistance of the Brace for Top Flange Laterals, continued

Resistance factor $\Phi_c := 0.9$ (AASHTO 6.5.4.2)

Factored Axial Resistance

$P_{rc} := \Phi_c \cdot P_n$ $P_{rc} = 458.6 \text{ kip}$ (Eq. 6.9.2.1-1)

Axial Resistance Check

$P_{uc} = 103.1 \text{ kip} < P_{rc} = 458.6 \text{ kip}$ Compression resistance is adequate

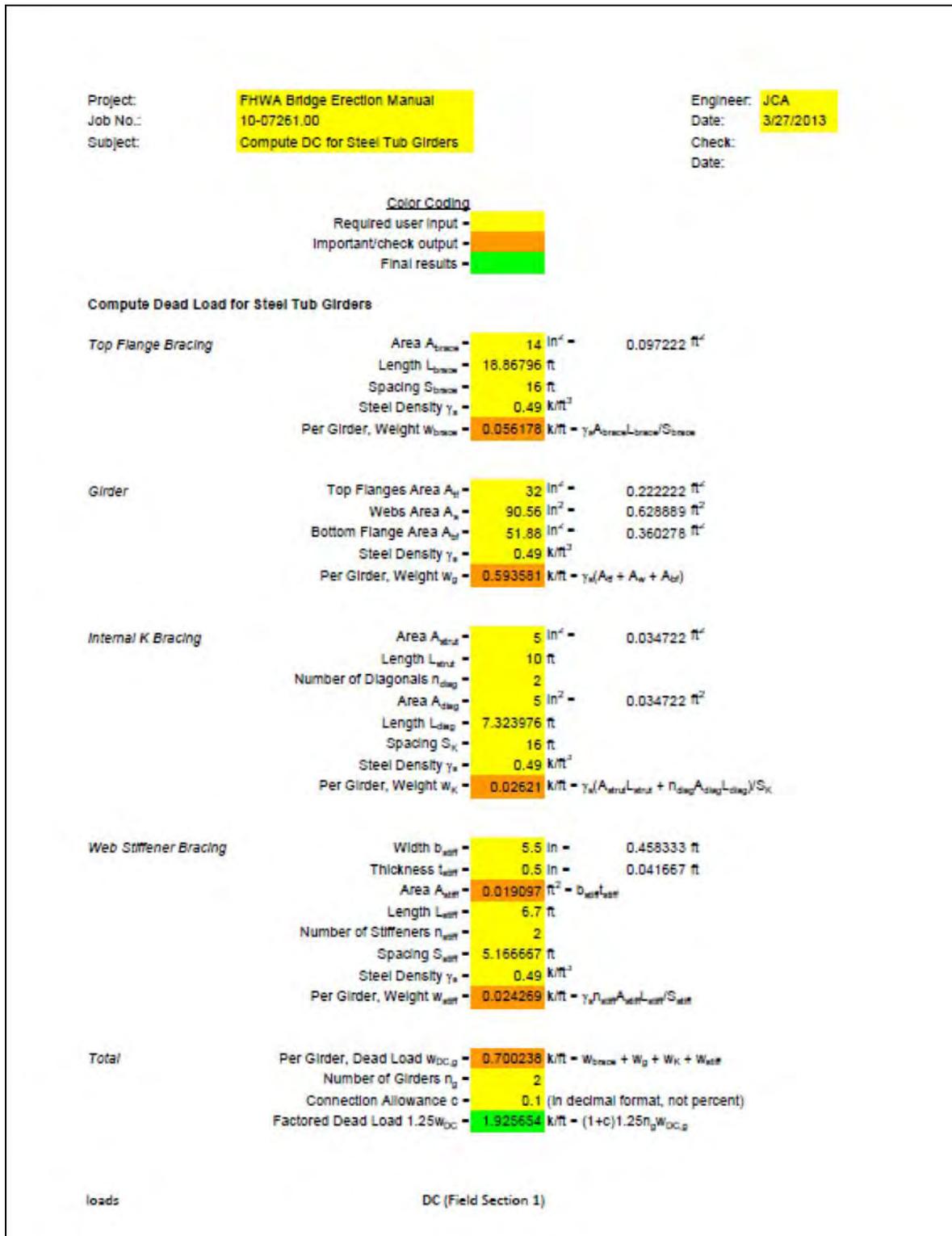


Figure B4-13: Loading Computation For Steel Dead Load (Dc) In Field Section 1

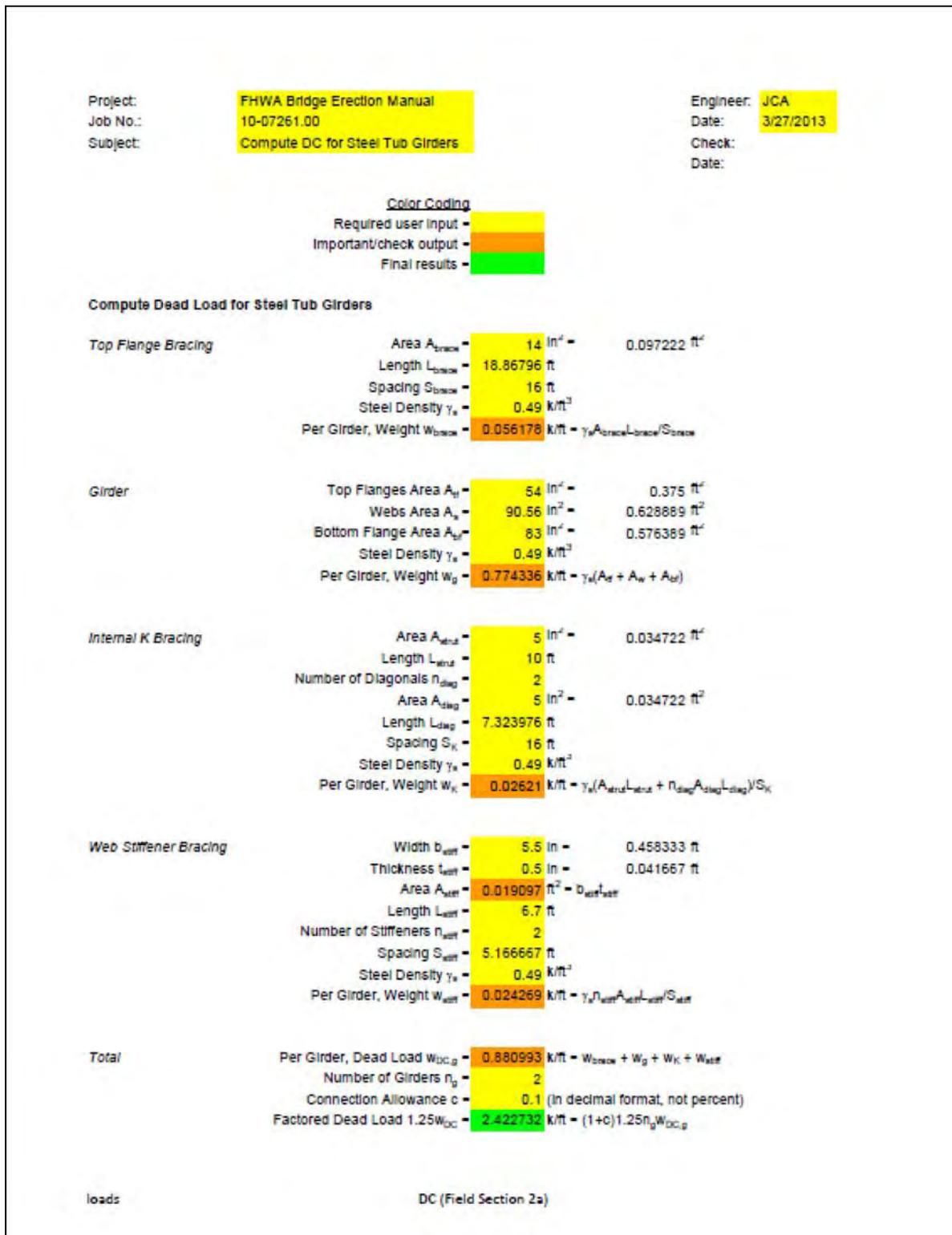


Figure B4-14: Loading Computation for Steel Dead Load (DC) in Field Section 2a

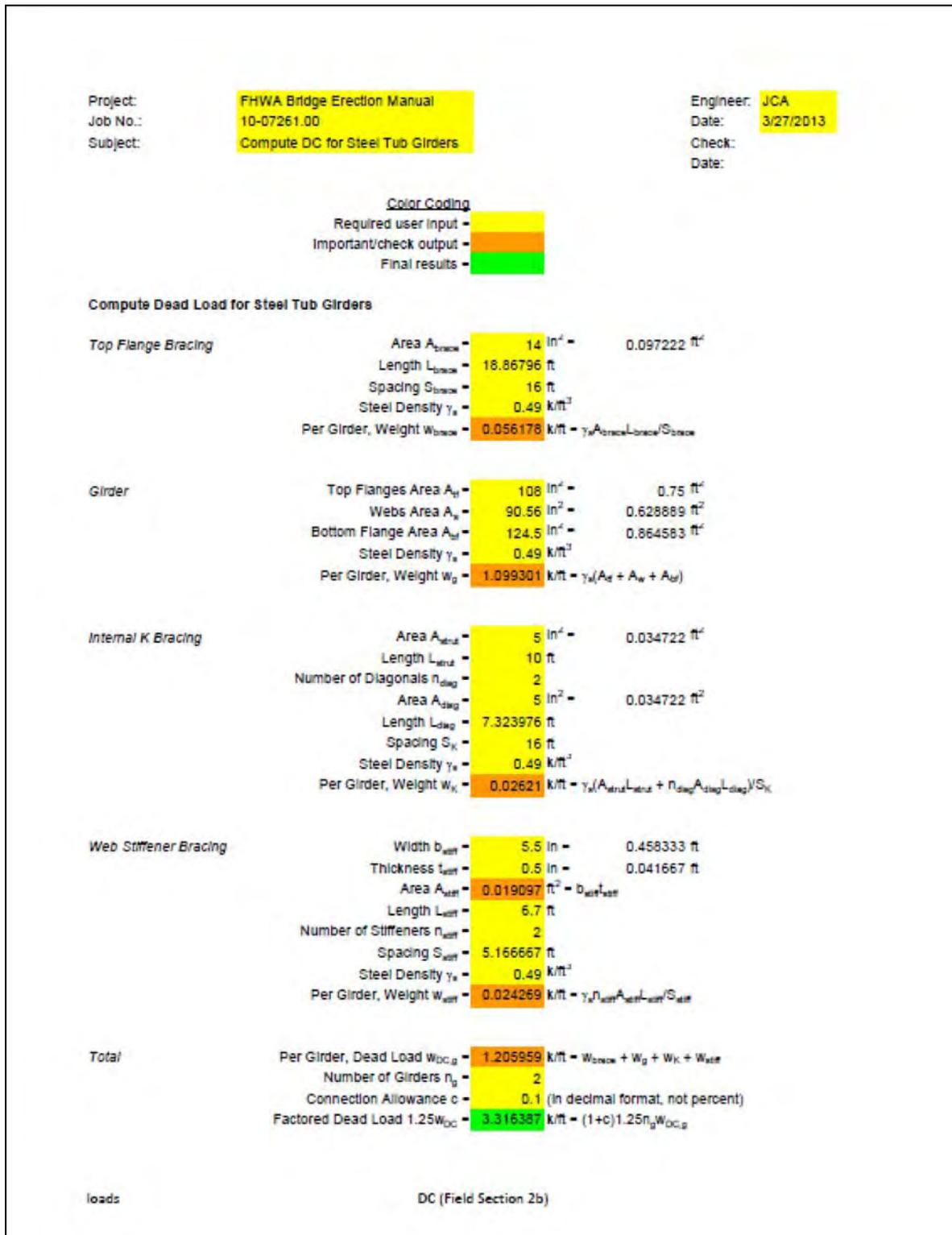


Figure B4-15: Loading Computation for Steel Dead Load (DC) in Field Section 2b



Figure B4-16: Loading Computation for Average Steel Dead Load (DC) in Field Section 2

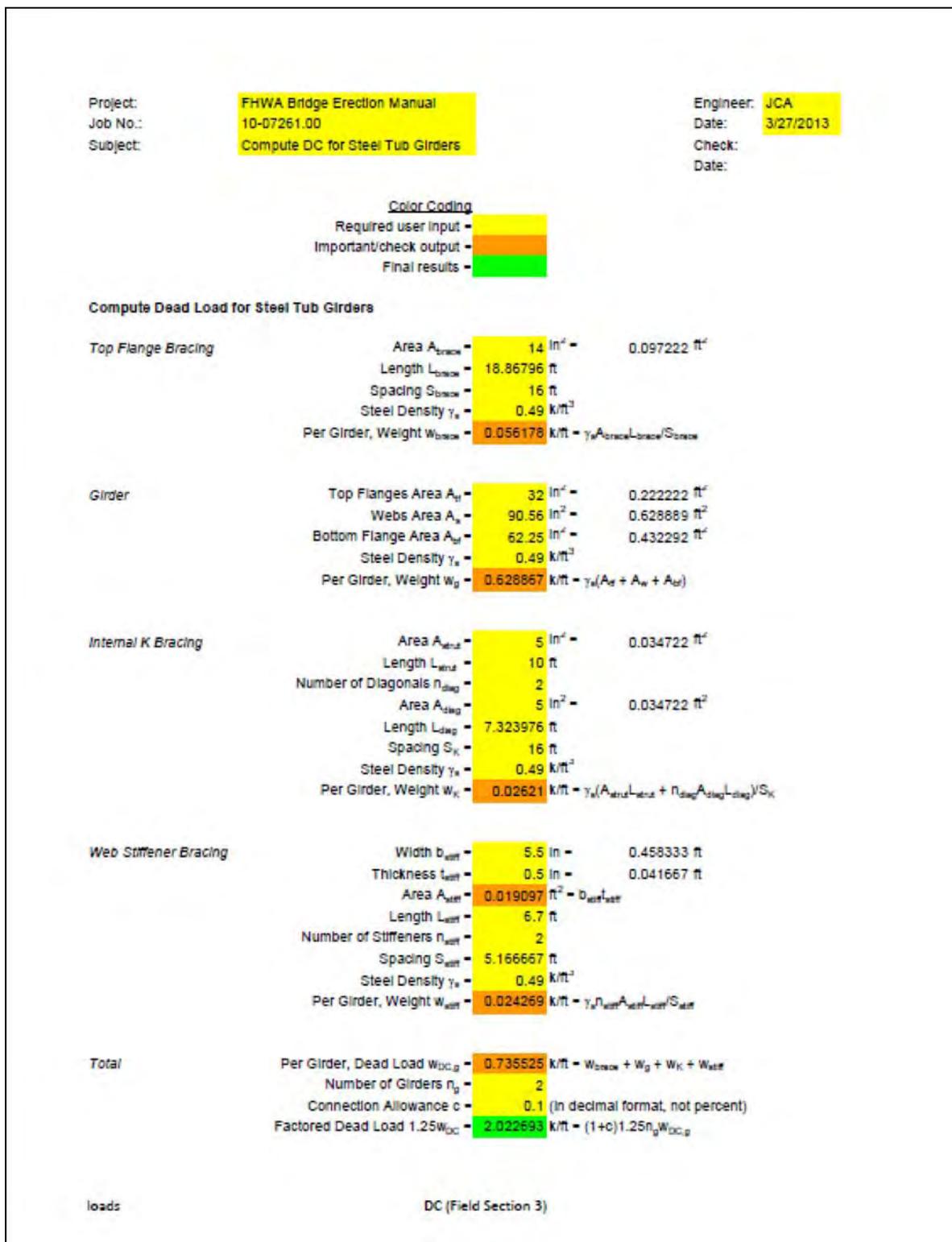


Figure B4-17: Loading Computation for Steel Dead Load (DC) in Field Section 3

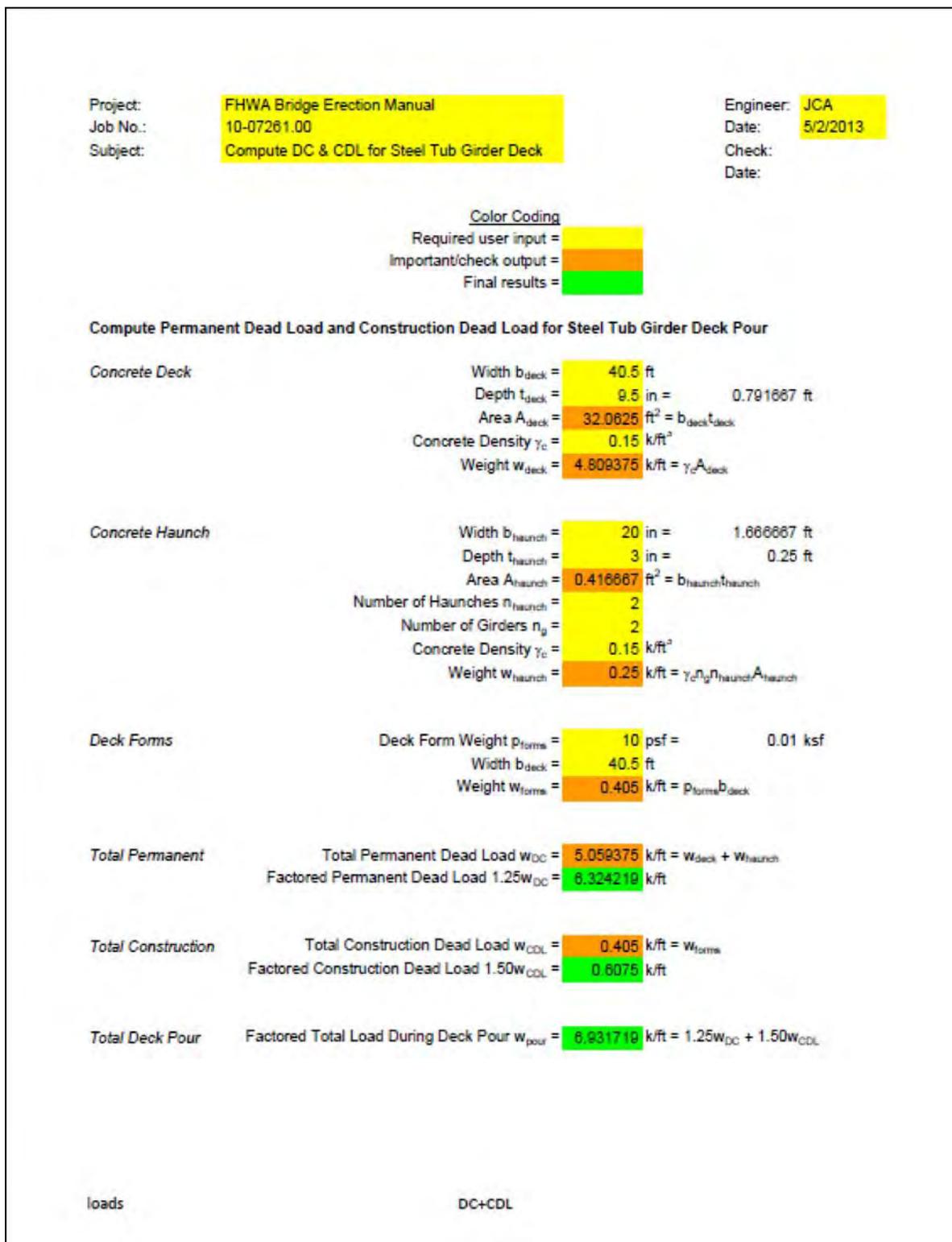


Figure B4-18: Loading Computation for Concrete Dead Load (DC) and Temporary Forms Dead Load (CDL)

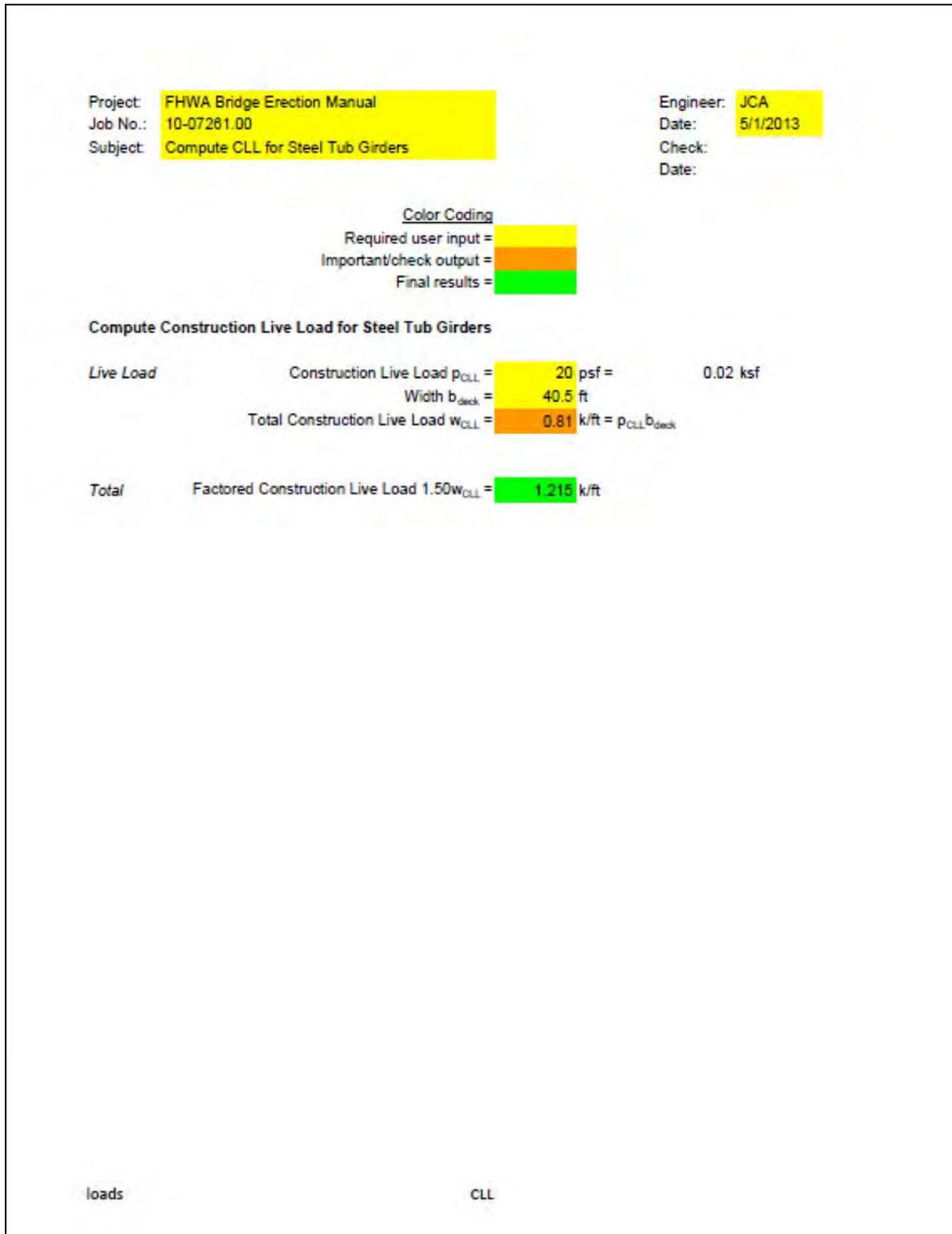


Figure B4-19: Loading Computation for Construction Live Load (CLL)

Eigenvalue Analysis for Cumulative Analysis Case 2

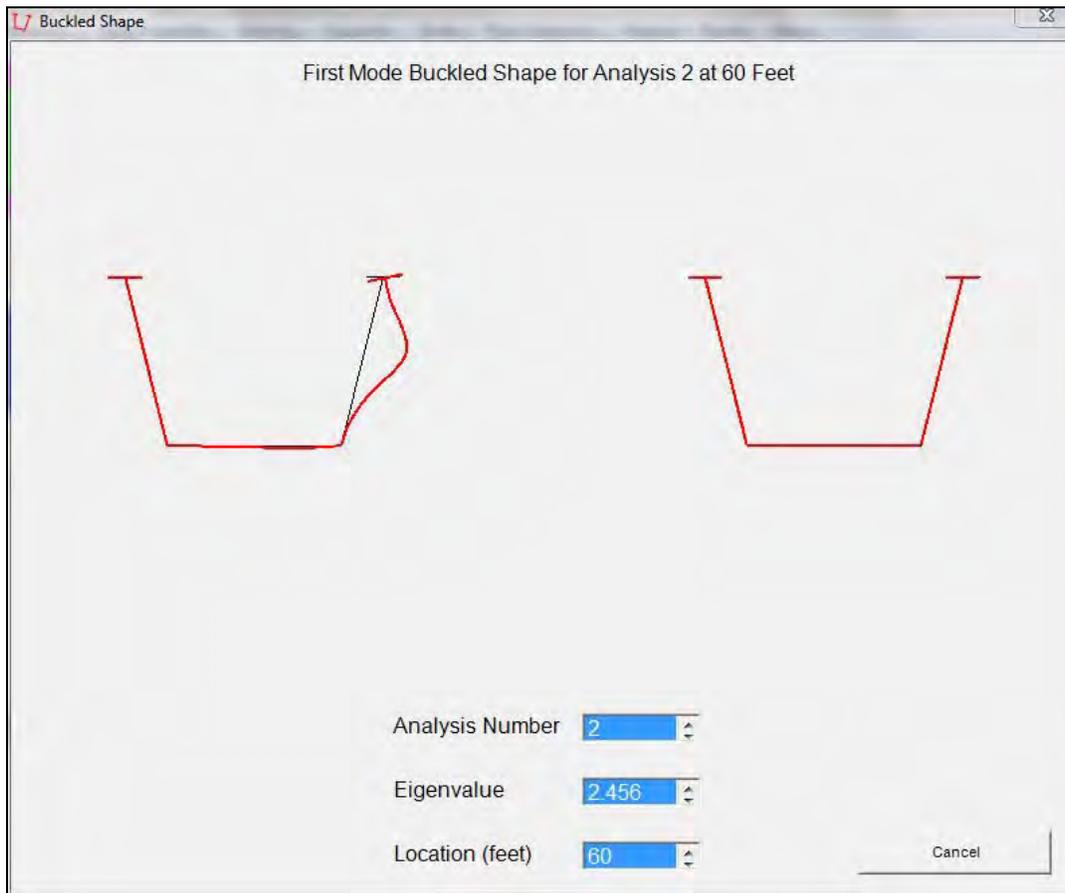


Figure B4-20: First Buckling Mode for Cumulative Analysis Case 2

As stated previously, the governing positive moment occurs at 60 feet into Span 1 and corresponds to cumulative Analysis Case 2, or the permanent dead load plus the construction dead load (Analysis Case 1 + Analysis Case 2). UTrAp can analyze up to 5 buckling modes. Figure B4-20 above shows the buckled shape for the first of the 5 modes, which occurs at an Eigenvalue of 2.456. As can be seen, it is a local web buckling phenomenon. Note that the cumulative Analysis Case 2 loads are factored, so the true buckling factor, relative to actual loads, is much higher: $2.456(1.25 \text{ DC} + 1.50 \text{ CDL})$.

The other 4 modes are not shown in the interest of brevity. However, they are all very similar. Modes 2 through 4 are also local web buckling, occurring at the other three girder webs. The Eigenvalue on the fifth mode is 2.636; it occurs at the same web as the first buckling mode and is a local web buckling in the opposite direction (mirror image) to that shown. No global buckling modes (e.g., girder lateral torsional buckling) are captured in the UTrAp buckling analysis for this example, meaning that they are higher modes than the first five (factored Eigenvalue must be greater than 2.636). Since post-buckling strength of the webs is incorporated in most design specifications, the local web buckling found here is not particularly noteworthy. At any rate, the factored design loads are far below the loads associated with these (or any other) buckling modes.

Eigenvalue Analysis for Cumulative Analysis Case 2 Without Bracing at Span 1 Splice

Sometime bracing is temporarily omitted to facilitate field splicing of the box girders. The UTrAp model was modified to reflect this by deleting the top flange bracing on either side of the first splice (starting and terminating at 96 ft in Span 1). The buckling analysis was rerun for this bridge to simulate the worst-case condition in which the deck pour was begun without this bracing in place (i.e., the contractor forgot to complete the brace installation after field splicing the girders). See Figure B4-21 for the UTrAp plan view of the revised model, illustrating the missing bracing in Span 1.

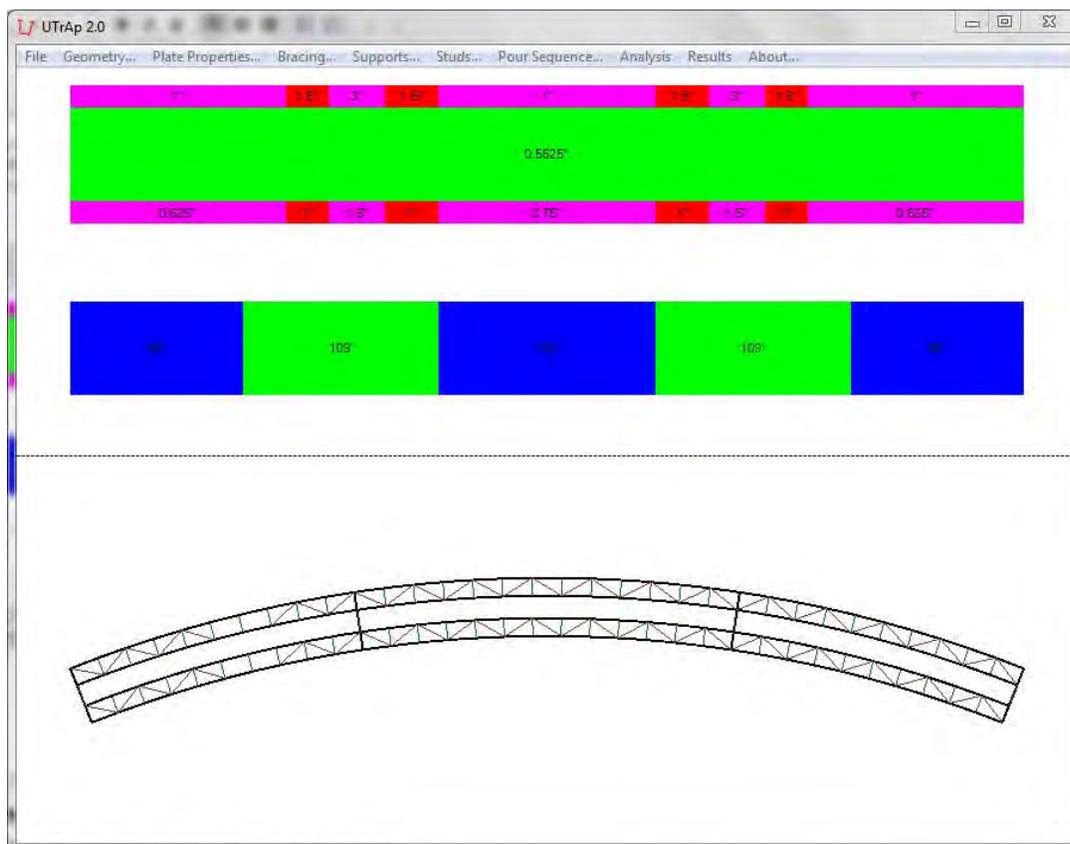
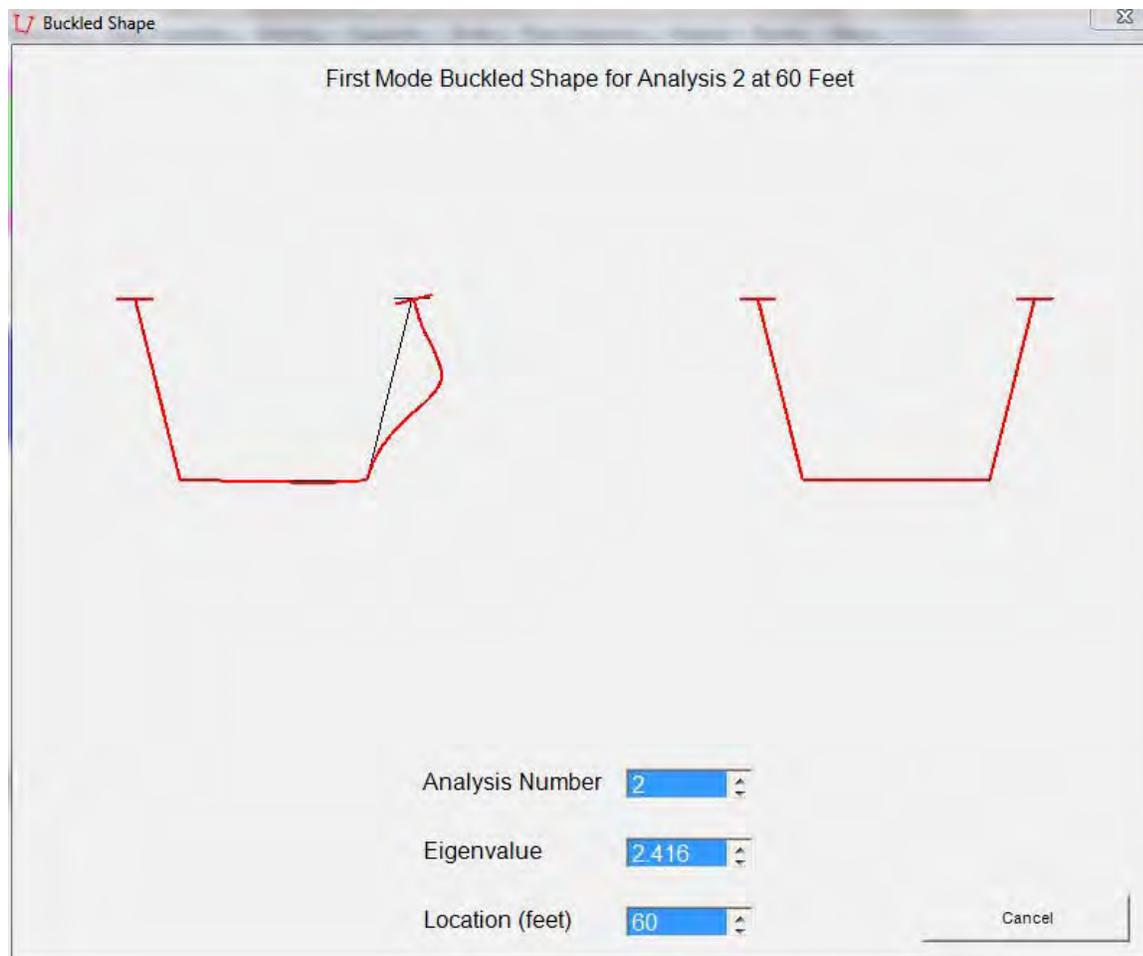


Figure B4-21: Problem Setup for Missing Brace Scenario

Again, at 60 feet into Span 1 the buckled shapes for cumulative Analysis Case 2, or the permanent dead load plus the construction dead load (Analysis Case 1 + Analysis Case 2), are examined. Figure B4-22 below shows the buckled shape for the first of the 5 modes analyzed by UTrAp, which occurs at an Eigenvalue of 2.416. As can be seen, it is still a local web buckling phenomenon. So the missing braces have, not surprisingly, weakened the bridge, but only minimally. The first buckling mode occurs at a slightly smaller Eigenvalue than before (2.416 vs. 2.456). Note that the cumulative Analysis Case 2 loads are factored, so the true buckling factor, relative to actual loads, is much higher: 2.416(1.25 DC + 1.50 CDL).

Eigenvalue Analysis for Cumulative Analysis Case 2 Without Bracing at Span 1 Splice, continued**Figure B4-22: First Buckling Mode for Cumulative Analysis Case 2 with Missing Brace**

The other 4 modes are not shown in the interest of brevity. However, they are all very similar to modes seen before. Modes 2 through 5 are local web buckling, occurring at various girder webs. The Eigenvalue on the fifth mode is 2.587.

Additionally, instead of contractor error, the much more common situation of the top braces temporarily being omitted to facilitate splicing can be examined. In this case, only Analysis Case 1 (steel permanent dead load) would be applicable for the temporary condition, post-splicing, where the box girders are temporarily supporting their own self-weight with some of the top bracing not yet installed. The Eigenvalue for Analysis Case 1, mode 1, is 12.21, corresponding to local web buckling at 60 feet into Span 1 (not shown here). With all bracing in place, the Eigenvalue for Analysis Case 1, mode 1, is 12.47 (not shown here). Thus, the impact of the omitted bracing is minimal. Furthermore, the Eigenvalue is more than twelve times the factored design loads, indicating a reasonable level of safety.

Reader Notes



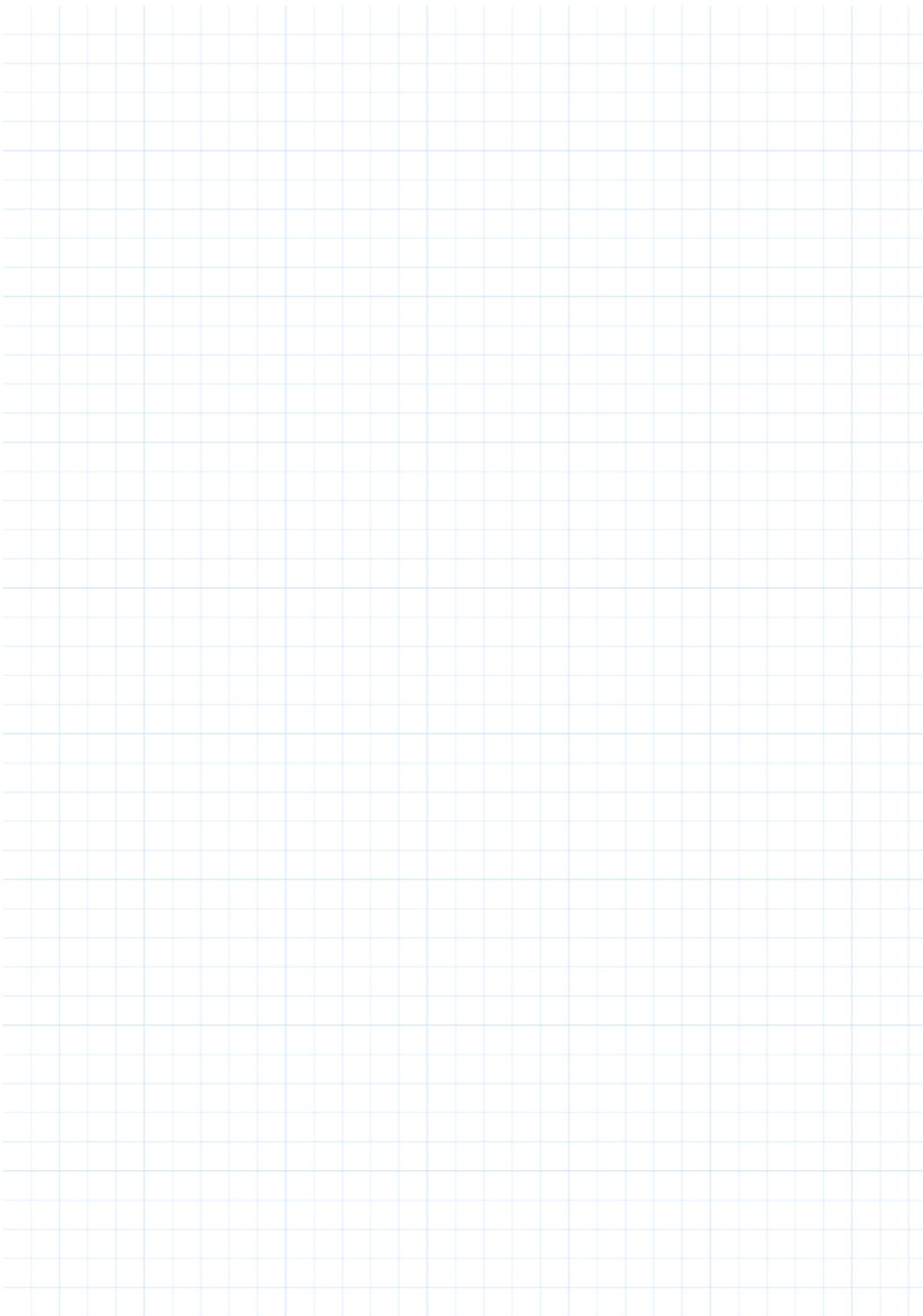
Reader Notes



Reader Notes



Reader Notes



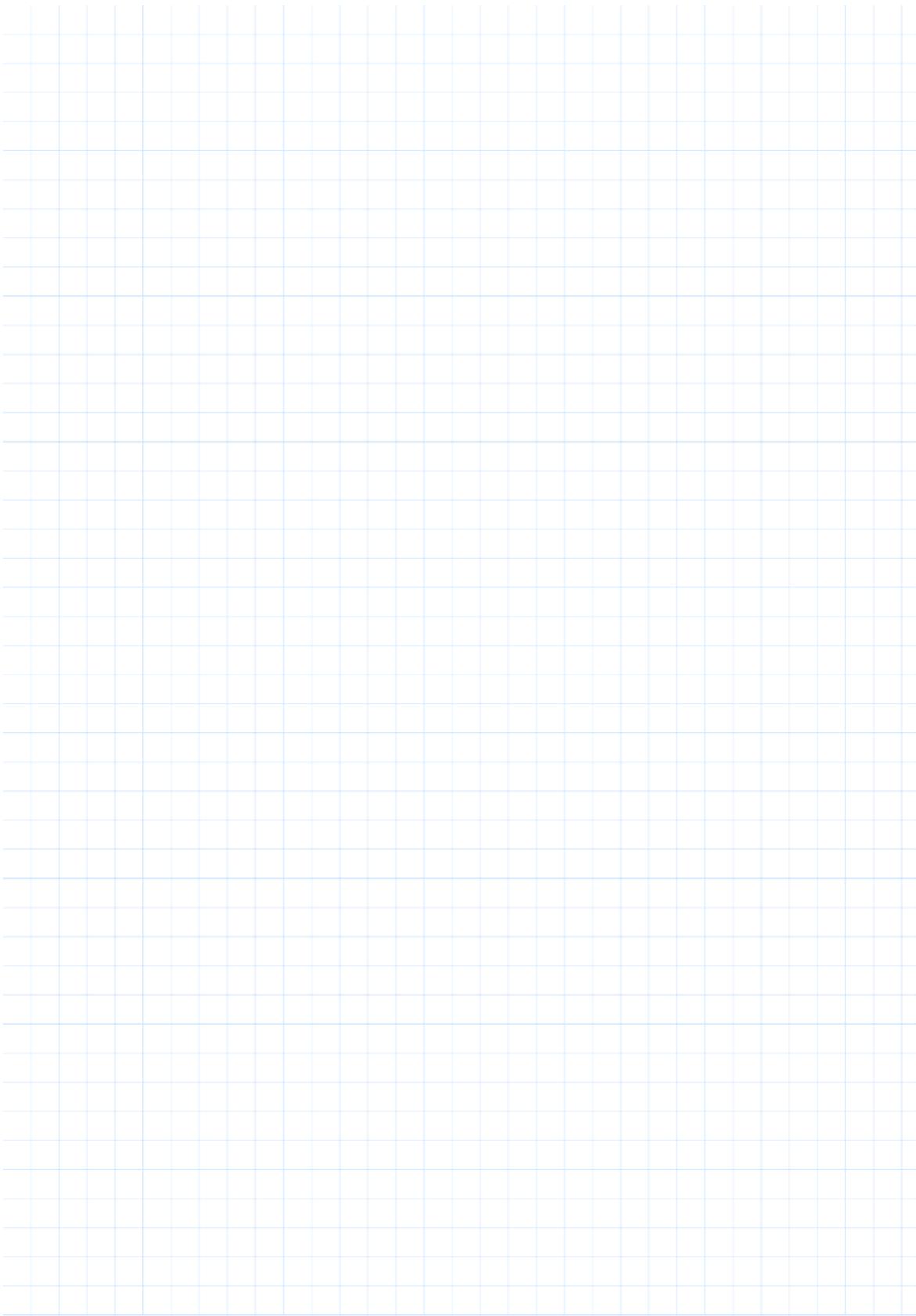
Reader Notes



Reader Notes



Reader Notes



Reader

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Notes

APPENDIX C

SURVEY ON ENGINEERING FOR STRUCTURAL SAFETY

In an effort to better understand the past experiences in bridge superstructure erection in individual states, and their requirements as they relate to superstructure erection, a survey was sent to all states that are AASHTO members. The survey included 18 questions, starting with any past problems related to girder erection (either steel or concrete), but concentrating on erection standards, erection design criteria, and submittal and review practices utilized. A summary of the survey answers is provided in Part 2 of Chapter 1. The entire survey is provided in this appendix.

STATES RESPONDING TO THE SURVEY

- 1. Arizona Department of Transportation**
- 2. Arkansas Department of Transportation**
- 3. CA Department of Transportation**
- 4. CDOT, Colorado Department of Transportation, Staff Bridge Branch**
- 5. Delaware DOT**
- 6. Georgia Department of Transportation**
- 7. Illinois Department of Transportation**
- 8. Iowa Department of Transportation**
- 9. Kansas department of Transportation**
- 10. Maryland SHA, Office of Structures**
- 11. MassDOT, Massachusetts Department of Transportation**
- 12. Michigan Department of Transportation**
- 13. Minnesota Department of Transportation**
- 14. Missouri Department of Transportation**
- 15. Nevada Department of Transportation**
- 16. New Jersey Department of Transportation**
- 17. New Mexico Department of Transportation**
- 18. North Carolina Department of Transportation**
- 19. North Dakota Department of Transportation**
- 20. NYSDOT, New York State Department of Transportation**
- 21. Ohio Department of Transportation**
- 22. Oklahoma Department of Transportation (ODOT)**
- 23. Oregon Department of Transportation**
- 24. Pennsylvania Department of Transportation**
- 25. SDDOT, South Dakota Department of Transportation**
- 26. State of Florida Department of Transportation**
- 27. Texas Department of Transportation**
- 28. Utah Department of Transportation**
- 29. Vermont Agency of Transportation**
- 30. Virginia Department of Transportation**
- 31. Washington State Department of Transportation**
- 32. Wisconsin DOT**
- 33. WYDOT, Wyoming Department of Transportation.**

1. How often have you experienced any collapses or near-miss events due to lifting, handling, or other temporary conditions due to construction of bridge superstructure members? Description of the problem(s) and lesson(s) learner or a contact person

Responding Agency	Frequency	Response
Arizona Department of Transportation	Rarely	
Arkansas Department of Transportation		
CA Department of Transportation	Occasionally	Falsework reduces clearance over traffic therefore truck without permit hit the falsework stringer or backhoe boom not ties properly hit the falsework stringer
CDOT, Staff Bridge Branch	Rarely	May of 2004 we had a girder collapse mostly due to not being able to erect a pair of girders as planned by the contractor, consequently only one girder was erected with inadequate bracings. Since then we have revised procedures for all Bridge Girder Erections specifications.
Delaware DOT	Rarely	
Georgia Department of Transportation	Rarely	Beams are occasionally dropped on construction
Illinois Department of Transportation	Often	On average, about twice a year we have new girders drop during erection or existing girders fail or collapse under construction loads
Iowa Department of Transportation	Rarely	Typically this was a result of a crane tipping and releasing the girder to save the crane. Crane tipping was the result of picks outside the recommended range, erection of girder during high wind, and coordination issues between two cranes walking a beam. One other situation involved a steel girder which was not adequately secured at a splice location (out of specifications) as a temporary situation.
Kansas Department of Transportation	Rarely	Contact: Travis Malone at 785.296.2066 or Malone@KSdot.org
Maryland SHA, Office of Structures	Never	

Responding Agency	Frequency	Response
MassDOT	Rarely	<p>Chelsea Street Bridge, the erection of the lift span truss. The truss span was being launched and the whole as had been assembled up to that time started to slide backwards and came off the temporary supports. The lesson learned was that restraints had not been installed to prevent the backwards slide.</p> <p>Contact: Rich DeSantis, D4 Construction</p>
Michigan Department of Transportation	Rarely	<p>In 2007, we had a pedestrian bridge approach slab pour fail the temporary shoring towers, resulting in collapse of the forms, and loss of the pour. This was a small pour of approximately 20 feet long by 14 feet wide; however, the falsework was over loaded, and failed halfway through the pour. Lessons learned were to further enforce our specifications which require all falsework to be designed and stamped by a registered professional engineer. This is not a failure; however, in 2005 we had a fascia beam rotate out of plane due to the wet load of the concrete during a deck pour. This was a deck replacement project for a ramp bridge on a superelevation. The previous superelevated shape was a rotated parabola, and our new design called for a straight line superelevation. The haunch depth this produced along with the additional overhang width to achieve standard shoulders added significant load to the fascia beam. Lessons learned were to consider bolstering beam in the future to avoid excessive haunch depths, or to transition the deck thickness back to our minimum of 9" on the outside of the fascia beam, as opposed to running the depth of the haunch out to the deck fascia.</p>
Minnesota Department of Transportation	Rarely	<p>This is very rare with 5-10 years between occurrences. Contact information for any of these questions is listed above.</p>
Missouri Department of Transportation	Rarely	
Nevada Dept of Transportation	Never	
New Jersey Department of Transportation	Rarely	<p>Prestressed Concrete Haunched Girder dropped during erection. Crane angle was not appropriate. Welded Steel girder Plate and Girders lifted in pairs dropped during erection. Only one crane and pick points not far enough apart and too much wind.</p>

Responding Agency	Frequency	Response
New Mexico Department of Transportation	Rarely	NMDOT had an exterior rolled steel girder fall over due to light wind during erection. The girder fell into the channel below. The Contractor failed to anchor the girder immediately after placement onto the substructure. Lesson learned: anchor girders (no matter what type) immediately after placing on substructure. NMDOT had a prestressed girder fall off the truck during transportation. The transporting truck drove up to the bridge site on an approximate slope of 10%. The truck and girder tipped over with the prestressed girder imploding. Lesson learned: be careful on steep slopes.
North Carolina Department of Transportation	Rarely	Lenoir County, Kinston: Left Lane Bridge on Crescent Road over NC Railroad A temporary shoring tower collapsed during erection of continuous plate girders, resulting in significant damage to all six lines of girders in the end span – August 2011. (See attached photos) The shoring towers legs were observed to be bowing or not vertical shortly before the collapse, and the contractor was making adjustments. The cause of the failure is undetermined. Brunswick County, Bridge on SR 1105 (North Middleton Avenue) over Intracoastal Waterway at Oak Island A strongback rod failed during erection of post-tensioned girders, resulting in a girder dropping to the ground – December 2008. The Department believes that welding of locknuts on high strength bars and reverse orientation of temporary cross-frames during their installation contributed to the collapse. Guilford County, Bridges on Left & Right Lanes Greensboro Western Urban Loop over West Friendly Avenue. When a shoring tower was removed, the two 310-ft. plate girders on that tower tilted over. The girder load was put back on the tower until the Department and Contractor could decide what to do. The Contractor's revised plan added four shoring towers. The size of the girder's top flange and the lack of top lateral bracing may have contributed to this behavior.
North Dakota Department of Transportation	Never	
NYSDOT	Rarely	Ensure adequacy of foundations of temporary bents. Follow erection plans
Ohio Department of Transportation	Never	ODOT is not aware of an incident involving near misses or collapses in any of its projects over the last decade. There was an accident involving a local agency project (Cuyahoga County) replacing a CSX railroad bridge (Eastland Road Overpass) on September 18, 2010 where the structural steel collapsed during erection of the floor beam units. One worker sustained non-life threatening injuries and no other injuries were reported.

Responding Agency	Frequency	Response
Oklahoma Department of Transportation (ODOT)	Rarely	Aware of one case where the Contractor bolted two spans together and tried to lift the girder, the girder twisted, and we required flame straightening to correct it.
Oregon Department of Transportation	Rarely	1- Loss of prestressed girder due to lack of proper support. 2- Loss of prestressed girder during hauling to the project site from casting plant.
Pennsylvania Department of Transportation	Occasionally	<p>Recently, a steel beam failed during removal due to lateral torsional buckling. The position of the lifting points resulted in an unbraced compression flange stress limit of 1.85 ksi. The applied bending stress was 7.8 ksi, and the beam failed.</p> <p>Lessons learned:</p> <ul style="list-style-type: none"> • Gravity, Bending, and Stability are related. Recognize and understand the stress state. • As a part of the failure investigation, obtain witness statements, capture videos and pictures, and coordinate with Central Office Bridge and Construction Staff. • Expect the contractor to revise the procedure. • For demolition, do not reduce the Factor of Safety to below 1.5. <p>Contact: Tom Macioce at 717.787.2881 or tmacioce@pa.gov</p>
SDDOT	Rarely	There was a slab bridge falsework collapse on an urban project where the end support transverse beams were hung from the bridge piers/abutments. The hanger rod/bolt diameter was changed (decreased) during construction and approved by the falsework designer but the larger hole diameter was left in the support beams without additional plate washers added. The standard washers covered the holes but were not sufficient to prevent them from being pulled through the oversize hole when concrete placement overloaded the detail.
State of Florida Department of Transportation	Rarely	<p>During the placement of long span Bulb-T superstructure, temporary bracing was not adequate and 7 beams that were placed and were waiting for deck form placement, collapsed because of high winds. Six beams were in place when the 7th and last beam was set after which a high wind gust caused the 7th beam to topple into the 6th beam and so on until all collapsed (see photo). There were no injuries or loss of life.</p> <p>During a hurricane like event concrete beams that were placed but not decked collapsed due to high winds. The reason for the collapse was inadequate temporary bracing. There were no injuries or loss of life. Because of these mishaps and other near miss events, the temporary bracing requirements for FDOT projects were strengthened dramatically. The following are links to FDOT Standard drawings covering bracing requirements.</p>

Responding Agency	Frequency	Response
Texas Department of Transportation	Occasionally	In most cases the cause has been equipment failures rather than a procedural issue. This occurs with both structural steel and prestressed girders. TxDOT has had long-standing design guidance for proportioning plate girders to avoid local instability or buckling issues. The old rule of thumb was that flanges widths needed to be 1/3 of the web depth. That has relaxed some due to recent research at UT-Austin but we are still much more conservative than the LRFD design code.
Utah Department of Transportation	Rarely	
Vermont Agency of Transportation	Rarely	Last year VTrans had a project to rehabilitate a historic metal through truss. While replacing steel on a built-up section of the top chord, there was a buckling failure when too many bolts were removed. For more information please contact Wayne Symonds.
Virginia Department of Transportation	Rarely	Superstructure/girder removal of highly skewed continuous bridge collapsed during removal. Problem was due to cutting /removal of most of the intermediate diaphragms prior to removing each segments/girders as was proposed/reviewed on the demolition plan.
Washington State Department of Transportation	Never	
Wisconsin DOT	Never	
WYDOT	Never	

2. How often have you experienced member deformation/stability/alignment problems during deck placement? Description of the problem(s) and lesson(s) learner or a contact person.

Responding Agency	Frequency	Response
Arizona Department of Transportation	Never	
Arkansas Department of Transportation	Rarely	
CA Department of Transportation	Occasionally	Falsework reduces clearance over traffic therefore truck without permit hit the falsework stringer or backhoe boom not ties properly hit the falsework stringer
CDOT, Staff Bridge Branch	Rarely	Steel Girders on one construction job showed some excess deformation, deflection under wet non-composite concrete loads. The pour sequence was revised and concrete was allowed to cure before other deck sections were placed. The lesson learned is to be more aware of these situations during design and use thicker web plates or adjust section areas and camber cuttings accordingly.
Delaware DOT	Never	
Georgia Department of Transportation	Rarely	For steel beams on bad skews, we have occasionally had the exterior beam twist while placing concrete. (The diaphragms are not fully welded until after deck placement for highly skewed bridges)
Illinois Department of Transportation	Often	About two or three times a year, fascia girders during the deck pour are not properly braced and cause thin decks to be poured. Sometimes girder deflection at staged construction joints behave abnormally causing the girders to not act identically.
Iowa Department of Transportation	Occasionally	Rotation of the exterior girder. We pay more attention to the cantilever overhang and securing the top flange. Loss of support of the beam at the abutment. The heavier beams required us to modify the bearing we were using.
Kansas Department of Transportation	Rarely	Contact: Travis Malone at 785-296-2066 or Malone@KSdot.org
Maryland SHA, Office of Structures	Rarely	During staged construction, there have been issues with installing diaphragms as one side of the bridge is completely loaded and the other is not resulting in misalignment of the bolt holes. Deflection of bridge deck slabs under traffic loads during stage construction has required changes in deck formwork design details to achieve a smooth riding surface.
MassDOT	Never	

Responding Agency	Frequency	Response
Michigan Department of Transportation	Occasionally	Staged or part width construction on skewed or curved structures often results in geometry or alignment issues on finished structures. We have had instances of the crown point not matching during the second stage of part width construction, or diaphragm rotation causing differential deflections on skewed deck pours.
Minnesota Department of Transportation	Rarely	This is extremely rare with more than 10 years between occurrences.
Missouri Department of Transportation	Rarely	
Nevada Dept of Transportation	Never	
New Jersey Department of Transportation	Rarely	Bearings shifted longitudinally after deck pour on a long-span-curve bridge with high camber. Reinforced elastomeric bearings deformed on steel girder bridge with high camber prior to pouring deck and secondary loads.
New Mexico Department of Transportation	Never	
North Carolina Department of Transportation	Often	Lateral girder deflections/rotations are often observed on bridges with skewed supports. Exacerbating factors include vertical curve grades, skew and superelevation (cross-slope).
North Dakota Department of Transportation	Rarely	Once. Rotation of exterior beam during deck placement.
NYSDOT	Occasionally	Inadequate shop assembly. Foundation location out of tolerance. Cumulative tolerances exceed practical limits
Ohio Department of Transportation	Rarely	ODOT has experienced excessive beam rotation on two steel girder projects within the past 10 years that required corrective action. In both instances, the lateral bracing was connected with bolts that were only snug tightened in slotted holes during the deck placement operation. The beams rotated until the bolts bottomed out in the slotted holes resulting in exaggerated beam rotations. This issue has been addressed from two fronts. First the Department prohibited the use of slotted holes and snug-tight connections in bracing members. Second, the Department construction specifications require all connections to be fully tightened before beginning deck placement operations.
Oklahoma Department of Transportation (ODOT)	Occasionally	Due to not using proper bracing, there have been some cases where we have twisted the outside beam.
Oregon Department of Transportation	Rarely	Overestimated camber in a steel plate girder superstructure resulted in a hump and not meeting the designed finished grade profile.

Responding Agency	Frequency	Response
Pennsylvania Department of Transportation	Occasionally	For steel girders, the deck placement is a critical condition. Every few years we experience an issue with distortion of the structural members or ride quality of the deck due to the deflection of the structural steel during deck placement. The issues occur most frequently on skewed steel bridges.
SDDOT	Rarely	There have been some instances of exterior beam rotation from overhang bracket (cantilever) loading that caused some separation of the exterior top flange and formwork. In one case this separation nearly caused the ends of transverse form joists to lose bearing and drop off the beam edge. An exterior beam torsional analysis is now typically performed to ensure adequate ties are placed to limit the top flange lateral deflection to a minimum and prevent future occurrences of this problem. There have also been some issues with expandable type joists deflecting excessively and not being of good condition/quality for the purpose intended.
State of Florida Department of Transportation	Rarely	A prestressed concrete fascia beam had positive camber (downward sag) after full dead load was applied. The beam was load tested and found to have acceptable load carrying capacity but there was no explanation for why it sagged.
Texas Department of Transportation	Rarely	The only issue in recent memory was uplift and rotation of a long single box girder HOV lane bridge. The designer did not include the potential for uplift in the plans. We now put required deck placing sequences in the plans if there is a potential for uplift.
Utah Department of Transportation	Occasionally	
Vermont Agency of Transportation	Rarely	A couple of years ago a girder bridge experienced significant rotation and lateral deflection of exterior girders during concrete replacement. This was due to insufficient temporary bracing by the Contractor. For more information contact Wayne Symonds.
Virginia Department of Transportation	Rarely	Multi- span continuous steel girders where some uplift was anticipated/noticed during stage concrete deck pouring. Sequence of pouring had to be changed or more stage pouring to be considered.
Washington State Department of Transportation	Rarely	Concrete girders on a precast simple span bridge rotated outward ($\frac{1}{2}$ " to $\frac{3}{4}$ ") during deck placement. The Contractor had not installed temporary deck bracing details (tension tie member at top, compression struts on bottom flange) to ensure girders did not rotate. The Contractor assumed the intermediate diaphragms were adequate to keep girders from rotating outward.
Wisconsin DOT	Rarely	
WYDOT	Never	

3. How often have you experienced problems in the final geometry/alignment of superstructures during or at the end of erection? Description of the problem(s) and lesson(s) learner or a contact person.

Responding Agency	Frequency	Response
Arizona Department of Transportation	Never	
Arkansas Department of Transportation	Rarely	
CA Department of Transportation	Rarely	Bridges (P/S CIP) girder with multiple frame has hinges in them. Due to post tensioning girder dead load and prestressing load are redistributed to falsework under the long span and combined with hinge curl the alignment is invariable. Contact: Rod Simmons.
CDOT, Staff Bridge Branch	Rarely	Pier cap was a bit shorter or a bit taller than elevation specified. The problem was corrected by shimming or shaving the concrete surface area and adjusting it for the right elevation. The lesson learned is to measure twice and cut once. We recommend to survey elevations more carefully.
Delaware DOT	Never	
Georgia Department of Transportation	Rarely	This occasionally happens but it is usually due to surveying errors.
Illinois Department of Transportation	Often	Problems cited in question 2 above cause thin decks making vertical grade changes necessary.
Iowa Department of Transportation	Never	
Kansas Department of Transportation	Rarely	
Maryland SHA, Office of Structures	Never	

Responding Agency	Frequency	Response
MassDOT	Rarely	<p>Wellesley, Route 16 over Route 9. When second stage beam erection began, it was found that the first stage beams were not erected according to the plans. Paul Maloy, D4 Construction</p> <p>Palmer, US 20 over Route 67 and CSX RR. Same problem as above. The details are still being investigated, but it may be that the substructures were not constructed as located on the plans. D2 Construction</p> <p>As it happens, both bridges were being built by the same contractor who was going out of business during the construction. It would appear that the problems encountered were associated with the contractor.</p>
Michigan Department of Transportation	Occasionally	As discussed above, staged or part width construction projects sometimes experience these issues. We have a skew policy that requires refined analysis on curved or severely skewed bridges (> 30°).
Minnesota Department of Transportation	Never	This essentially never happens
Missouri Department of Transportation	Rarely	
Nevada Dept of Transportation	Rarely	On curved structures, we have experienced alignment problems on instances where the field splice gap was wider than deigned. These have occurred despite shop assembly of the sections in question. In some cases, geometric errors of the substructures may have contributed to the problems encountered. In one instance, field splice connections we redesigned to accommodate the additional gap.
New Jersey Department of Transportation	Rarely	Contractor could not attach end diaphragms after erection of bridge with severe skew. Construction documents should specified plumb at erection or at final position after application of dead load.
New Mexico Department of Transportation	Rarely	We had a contractor pour a deck onto BT-54 prestressed girders. The exterior girders came out of plumb about 5° during the deck pour but were not noticed until after forms were stripped for the pier and abutment diaphragms. Lesson learned: better anchorage of intermediate diaphragms and temporary anchorage or cross-frames are needed near girder ends to keep girders plumb.
North Carolina Department of Transportation	Rarely	Occasionally diaphragms that are a welded frame, as opposed to individual angles or WT members, are difficult to install on curved structures.
North Dakota Department of Transportation	Never	

Responding Agency	Frequency	Response
NYSDOT	Rarely	
Ohio Department of Transportation	Never	
Oklahoma Department of Transportation (ODOT)	Rarely	
Oregon Department of Transportation	Rarely	<ul style="list-style-type: none"> - End bent of an eight span curved single cell post-tensioned box girder rotated and lifted off from bearing after post-tensioning. Fix required shimming the bearings and plating the finger plate joint at end bent. - End bent of a three span post-tension box girder were lifted from bearing not well adjusted post-tension path.
Pennsylvania Department of Transportation	Rarely	This has occurred with skewed steel bridges. One issue occurred because some of the end cross-frame bays were installed in the wrong bay location. This resulted in the lateral shift of the girders and difficult fit-up.
SDDOT	Rarely	There have been issues with bearing/anchor bolt alignment in the past. Plans now typically call for field welding of the sole plate to the bottom flange and block outs in the pier cap to allow for casting of anchor bolts after bearing placement. On a recent bridge widening project with a skewed and curved superstructure there were some alignment differences between old and new where the diaphragms had to be field adjusted
State of Florida Department of Transportation	Often	This is only a common occurrence during the erection of precast concrete segmental box girder superstructures where it happens on occasion during erection of almost every bridge
Texas Department of Transportation	Occasionally	This is more common than one might think but is usually the result of surveying errors in the field. The fabrication surveying has almost always been correct. The issue is usually bents that are out of place, at the wrong elevation, or at the wrong skew angle. We've noticed a general decline in the quality of field surveying with the increased use of GPS/Total Station surveying
Utah Department of Transportation	Often	
Vermont Agency of Transportation	Never	
Virginia Department of Transportation	Rarely	Wide width of bridge superstructure steel frames under stage construction, where some misalignment occurred on the exterior girders. Lateral bracing/diaphragms attachment/connections were made unevenly within the frame structure.

Responding Agency	Frequency	Response
Washington State Department of Transportation	Never	
Wisconsin DOT	Rarely	
WYDOT	Never	

4. When checking a girder for stability during handling and erection do you require AASHTO LRFD be used? If no, what other standard(s) are allowed?

Responding Agency	Y/N	Response
Arizona Department of Transportation	No	AASHTO Guide specifications for Temporary Works
Arkansas Department of Transportation	No	Contractor is responsible
CA Department of Transportation	No	Falsework is checked using ASD.
CDOT, Staff Bridge Branch	Yes	
Delaware DOT	Yes	
Georgia Department of Transportation	No	AASHTO 17 th edition or other acceptable analysis methods.
Illinois Department of Transportation	Yes	
Iowa Department of Transportation	No	This is the contractor's responsibility to assure stability using means and methods.
Kansas Department of Transportation	Yes	
Maryland SHA, Office of Structures	Yes	
MassDOT	No	AASHTO Standard Specifications
Michigan Department of Transportation	Yes	
Minnesota Department of Transportation	No	
Missouri Department of Transportation	No	MoDOT specifications are typically "end result". It is the contractor's responsibility to develop a plan for erection of structures, with assistance from the fabricator and/or the contractor's engineer.
Nevada Dept of Transportation	Yes	
New Jersey Department of Transportation	Yes	
New Mexico Department of Transportation	Yes	
North Carolina Department of Transportation	Yes	Bridge designers are advised to follow NCDOT's "Constructability Guidelines for Steel Plate Girder Bridges" (see attachment labeled "constructability guidelines")
North Dakota Department of Transportation	Yes	

Responding Agency	Y/N	Response
NYSDOT	Yes	
Ohio Department of Transportation	No	The Department requires the design for erection procedures in accordance with either the AASHTO Standard Specifications for Highway Bridges or the AASHTO LRFD Bridge Design Specifications.
Oklahoma Department of Transportation (ODOT)	No	
Oregon Department of Transportation	No	The contractor's engineer often uses AISC construction manual.
Pennsylvania Department of Transportation	No	
SDDOT	Yes	
State of Florida Department of Transportation	No	AASHTO LRFD, AASHTO Guide Design Specifications for temporary works, FDOT Structures Manual/Structures Design Guidelines (4.3.4)
Texas Department of Transportation	No	The University of Texas developed software (UTBridge and UTLift) that we use for both design and erection analyses. This is not currently a requirement but will likely become a required check when we re-write our specs for 2014.
Utah Department of Transportation	Yes	
Vermont Agency of Transportation	Yes	
Virginia Department of Transportation	Yes	
Washington State Department of Transportation	Yes	We also have a Standard Specification for Temporary Bracing:
Wisconsin DOT	Yes	
WYDOT	No	

5. Do you require an erection procedure to be submitted by the bridge erector?

Responding Agency	Y/N
Arizona Department of Transportation	Yes
Arkansas Department of Transportation	Yes
CA Department of Transportation	Yes
CDOT, Staff Bridge Branch	Yes
Delaware DOT	Yes
Georgia Department of Transportation	No
Illinois Department of Transportation	Yes
Iowa Department of Transportation	Yes
Kansas Department of Transportation	Yes
Maryland SHA, Office of Structures	Yes
MassDOT	Yes
Michigan Department of Transportation	No
Minnesota Department of Transportation	Yes
Missouri Department of Transportation	No
Nevada Dept of Transportation	Yes
New Jersey Department of Transportation	Yes
New Mexico Department of Transportation	Yes
North Carolina Department of Transportation	Yes
North Dakota Department of Transportation	No
NYSDOT	Yes
Ohio Department of Transportation	Yes
Oklahoma Department of Transportation	No

Responding Agency	Y/N
(ODOT)	
Oregon Department of Transportation	Yes
Pennsylvania Department of Transportation	Yes
SDDOT	Yes
State of Florida Department of Transportation	Yes
Texas Department of Transportation	Yes
Utah Department of Transportation	Yes
Vermont Agency of Transportation	Yes
Virginia Department of Transportation	Yes
Washington State Department of Transportation	Yes
Wisconsin DOT	No
WYDOT	Yes

6. If the answer to 5 is yes, are they required for all bridges?

Responding Agency	Y/N
Arizona Department of Transportation	Yes
Arkansas Department of Transportation	Yes
CA Department of Transportation	No
CDOT, Staff Bridge Branch	Yes
Delaware DOT	Yes
Georgia Department of Transportation	
Illinois Department of Transportation	No
Iowa Department of Transportation	No
Kansas Department of Transportation	Yes
Maryland SHA, Office of Structures	Yes
MassDOT	Yes
Michigan Department of Transportation	N/A
Minnesota Department of Transportation	No
Missouri Department of Transportation	
Nevada Dept of Transportation	Yes
New Jersey Department of Transportation	Yes
New Mexico Department of Transportation	No
North Carolina Department of Transportation	No
North Dakota Department of Transportation	
NYSDOT	Yes
Ohio Department of Transportation	Yes
Oklahoma Department of Transportation	N/A

Responding Agency	Y/N
(ODOT)	
Oregon Department of Transportation	No
Pennsylvania Department of Transportation	Yes
SDDOT	No
State of Florida Department of Transportation	Yes
Texas Department of Transportation	No
Utah Department of Transportation	Yes
Vermont Agency of Transportation	Yes
Virginia Department of Transportation	No
Washington State Department of Transportation	Yes
Wisconsin DOT	N/A
WYDOT	No

7. If the answer to question 6 is no, is there a size, span length, geometry feature or other threshold consideration that triggers the requirement? If yes, what size threshold or other consideration that triggers submittal of a procedure?

Responding Agency	Y/N	Response
Arizona Department of Transportation		
Arkansas Department of Transportation		
CA Department of Transportation	Yes	All steel girder & precast girder.
CDOT, Staff Bridge Branch	No	Some actions with in specs only apply when there is traffic below the structure.
Delaware DOT	N/A	
Georgia Department of Transportation	Yes	For bridges over railroads, erection procedures are required by the railroads now.
Illinois Department of Transportation	Yes	Steel girders only.
Iowa Department of Transportation	Yes	This is required when the design engineer determines the complexity of the erection dictates an erection plan
Kansas Department of Transportation	N/A	
Maryland SHA, Office of Structures		
MassDOT		
Michigan Department of Transportation	Yes	As discussed above, staged or part width construction projects sometimes experience these issues. We have a skew policy that requires refined analysis on curved or severely skewed bridges (> 30°). Type of construction (segmental, deck cast with beams, etc.), fracture critical, deep curved steel plate girders, girders with a field splice, cantilever construction
Minnesota Department of Transportation	No	<i>Engineering judgment is used to determine which bridges require an erection plan.</i>
Missouri Department of Transportation		
Nevada Dept of Transportation	N/A	

Responding Agency	Y/N	Response
New Jersey Department of Transportation		
New Mexico Department of Transportation	Yes	We require erection plans, design calculations and procedures for steel girder bridges in accordance with AASHTO/NSBA Steel Bridge Erection Guide Specification. We do not require it for prestressed girders.
North Carolina Department of Transportation	Yes	See NCDOT's "Constructability Guidelines for Steel Plate Girder Bridges".
North Dakota Department of Transportation		
NYS DOT		
Ohio Department of Transportation	N/A	Erection plans are required by ODOT Construction and Materials Specification (C&MS) Item 501.05 for every project that requires erection of steel or precast concrete structural members.
Oklahoma Department of Transportation (ODOT)	Yes	Curved girder or over traffic
Oregon Department of Transportation	No	
Pennsylvania Department of Transportation	N/A	
SDDOT	Yes	There is no set policy on erection plan submittal requirement. It is typically based on engineering judgment considering span lengths, girder depths, curvature, skew, etc.
State of Florida Department of Transportation		
Texas Department of Transportation	Yes	Our specs say "railroad underpasses, trusses, field-spliced (welded or bolted) girders, arches, or other members for which erection drawings are required on the plans. "Submit an additional copy of the drawings for railroad underpasses. Erection drawings are not required for rolled I-beam units unless otherwise noted on the plans
Utah Department of Transportation		All bridges require submittal, but only bridges specifically designated by the EOR require erection drawing approval.
Vermont Agency of Transportation	N/A	

Responding Agency	Y/N	Response
Virginia Department of Transportation	Yes	Depends on the type of structure given consideration to its span length, beam/girder types, bridge geometry (straight, high skewed, curved) simple or multi-span, and its behavior to type of substructure (i.e. boxed pier caps, prestressed/ post-tensioned)
Washington State Department of Transportation		
Wisconsin DOT	No	
WYDOT	Yes	Complexity of bridge (curved, span length < 200', large skews < 40 degrees), and bridge type [currently have an deck arch under construction, required an erection plan]

8. Do you have requirements for the erection procedure contents and format? If yes, provide a copy or link.

Responding Agency	Y/N	Response
Arizona Department of Transportation	No	
Arkansas Department of Transportation	No	
CA Department of Transportation	Yes	Standard Specification 2010: Section 55-1.03C, "Erection"
CDOT, Staff Bridge Branch	Yes	See Link on last page
Delaware DOT	Yes	http://deldot.gov/information/pubs_forms/manuals/standard_specifications/index.shtml
Georgia Department of Transportation	No	
Illinois Department of Transportation	Yes	For Curved Steel Girders http://www.dot.il.gov/bridges/GBSP55.pdf
Iowa Department of Transportation	No	
Kansas Department of Transportation	Yes	Special Provision 08-07004-R03 http://www.ksdot.org/burconsmain/specprov/2007/pdf/07-07004-r03.pdf
Maryland SHA, Office of Structures	No	
MassDOT	Yes	http://www.massdot.state.ma.us/Portals/8/docs/manuals/SSP022510MetEng.pdf Starting on page SUPPLEMENT C2010 - 141
Michigan Department of Transportation	No	
Minnesota Department of Transportation	No	
Missouri Department of Transportation	No	
Nevada Dept of Transportation	Yes	

Responding Agency	Y/N	Response
New Jersey Department of Transportation	Yes	www.nj.gov/transportation/eng/specs/2007/spec500.shtm
New Mexico Department of Transportation	Yes	ftp://ftp.mdt.mt.gov/research/LIBRARY/NSBASBEGS-1-OL-STEEL_BRIDGE-AASHTO.PDF
North Carolina Department of Transportation	Yes	http://www.ncdot.org/doh/preconstruct/highway/structur/psp/PSP2012/PSP033.pdf http://www.ncdot.org/doh/preconstruct/highway/structur/psp/PSP2012/PSP006.pdf
North Dakota Department of Transportation	No	
NYS DOT	Yes	https://www.dot.ny.gov/divisions/engineering/structures/manuals/scm
Ohio Department of Transportation	Yes	As mentioned above, C&MS 501.05 governs the requirements. The ODOT C&MS may be accessed at: http://www.dot.state.oh.us/Divisions/ConstructionMgt/OnlineDocs/Specifications/2013CMS/500/501.htm#A_501_05
Oklahoma Department of Transportation (ODOT)	No	
Oregon Department of Transportation	Yes	Web and bearing stiffener at end bents should be plumb at final condition.
Pennsylvania Department of Transportation	Yes	The erection drawings and calculations must be signed and sealed by a PE registered in Pennsylvania.
SDDOT	No	The erection procedure submittal information requirements are intended to be referenced to those found in the AASHTO/NSBA Steel Bridge Erection Guide Specification.
State of Florida Department of Transportation	Yes	ftp://ftp.dot.state.fl.us/LTS/CO/Specifications/SpecBook/2010Book/460.pdf and see spec 460-7.1.3 ftp://ftp.dot.state.fl.us/LTS/CO/Specifications/WorkBook/Jan2012/SS4520000.pdf and see spec 452-8.1
Texas Department of Transportation	Yes	Clearly indicate at least: procedures, sequence of work, equipment to be used, location of falsework, erection cranes and holding cranes, falsework design details, girder lifting points, adjacent structures loaded and requirements for releasing cranes during erection
Utah Department of Transportation	Yes	http://www.udot.utah.gov/main/uconowner.gf?n=7602520459537371

Responding Agency	Y/N	Response
Vermont Agency of Transportation	Yes	<a href="http://vtranscontracts.vermont.gov/sites/aot_contract_administrati
on/files/documents/2011specbook/2011Division500.pdf">http://vtranscontracts.vermont.gov/sites/aot_contract_administrati on/files/documents/2011specbook/2011Division500.pdf
Virginia Department of Transportation	No	
Washington State Department of Transportation	Yes	Copy Provided
Wisconsin DOT	No	
WYDOT	No	

9. Do you specify criteria for erectors for design wind load considerations during erection? If yes, what is the wind load or provide a copy or link.

Responding Agency	Y/N	Response
Arizona Department of Transportation	No	
Arkansas Department of Transportation	No	
CA Department of Transportation	Yes	Specifications for Precast Prestressed Concrete Bridge members require that bracing resist the following lateral pressures: (see Figure C-10)
CDOT, Staff Bridge Branch	No	
Delaware DOT	No	
Georgia Department of Transportation	No	
Illinois Department of Transportation	No	
Iowa Department of Transportation	No	
Kansas Department of Transportation	No	
Maryland SHA, Office of Structures	No	
MassDOT	Yes	See link above
Michigan Department of Transportation	No	This is done on a project by project basis depending on complexity. For example, for jacking loads for bearing replacement on a segmental bridge, the design specifies the wind load intensity, and the duration the jacks must with stand the load.
Minnesota Department of Transportation	No	
Missouri Department of Transportation	No	
Nevada Dept of Transportation	No	

Responding Agency	Y/N	Response
New Jersey Department of Transportation	No	
New Mexico Department of Transportation	No	We do not specify criteria but we do require all girders to be stabilized with falsework, temporary bracing, and/or holding cranes until a sufficient number of adjacent girders are erected with diaphragms and/or cross-frames connected to provide the necessary lateral stability and to make the structure self-supporting.
North Carolina Department of Transportation	Yes	AASHTO Guide Specification for Bridge Temporary Works, and as amended by NCDOT Special Provision entitled "Falsework and Formwork" (see attachment by that same name)
North Dakota Department of Transportation	No	
NYS DOT	No	
Ohio Department of Transportation	No	
Oklahoma Department of Transportation (ODOT)	No	
Oregon Department of Transportation	No	
Pennsylvania Department of Transportation	Yes	ftp://ftp.dot.state.pa.us/public/Bureaus/BOPD/Bridge/2013/bd/bd620m.pdf
SDDOT	No	
State of Florida Department of Transportation	Yes	FDOT requires Contractors to comply with the AASHTO Guide Design Specifications for Bridge Temporary Works and Construction Handbook for Bridge Temporary Works
Texas Department of Transportation	No	
Utah Department of Transportation	No	
Vermont Agency of Transportation	No	
Virginia Department of Transportation	No	

Responding Agency	Y/N	Response
Washington State Department of Transportation	No	
Wisconsin DOT	No	
WYDOT	No	

Structure Height H (ft)	Lateral Pressure (psf)
$0 < H \leq 30$	15
$30 < H \leq 50$	20
$50 < H \leq 100$	25
$H > 100$	30

Figure C-23 Lateral Pressure and Structure Height

10. Do you specify criteria to erectors for maximum lateral deflection of girders subjected to wind load during erection? If yes, what is the deflection criteria or provide a copy or link.

Responding Agency	Y/N	Response
Arizona Department of Transportation	No	
Arkansas Department of Transportation	No	
CA Department of Transportation	No	
CDOT, Staff Bridge Branch	No	
Delaware DOT	No	
Georgia Department of Transportation	No	
Illinois Department of Transportation	No	
Iowa Department of Transportation	No	
Kansas Department of Transportation	No	
Maryland SHA, Office of Structures	No	
MassDOT	No	
Michigan Department of Transportation	No	We follow the provision of the AASHTO LRFD Bridge Design Specification.
Minnesota Department of Transportation	No	
Missouri Department of Transportation	No	
Nevada Dept of Transportation	No	
New Jersey Department of Transportation	No	
New Mexico Department of Transportation	No	
North Carolina Department of Transportation	No	
North Dakota Department of Transportation	No	
NYSDOT	No	
Ohio Department of Transportation	No	
Oklahoma Department of Transportation (ODOT)	No	

Responding Agency	Y/N	Response
Oregon Department of Transportation	No	
Pennsylvania Department of Transportation	No	Pennsylvania does require the contractor to evaluate the effects of the deflections.
SDDOT	No	
State of Florida Department of Transportation	Yes	Deflection criteria are probably covered in the AASHTO Guide Design Specifications for Bridge Temporary Works and Construction Handbook for Bridge Temporary Works
Texas Department of Transportation	No	
Utah Department of Transportation	No	
Vermont Agency of Transportation	No	
Virginia Department of Transportation	No	
Washington State Department of Transportation	No	
Wisconsin DOT	No	
WYDOT	No	

11. Do you specify a minimum safety factor with regard to global stability of a partially erected or demolished structure for global structural stability?

Responding Agency	Y/N
Arizona Department of Transportation	No
Arkansas Department of Transportation	No
CA Department of Transportation	No
CDOT, Staff Bridge Branch	No
Delaware DOT	No
Department of Transportation	No
Georgia Department of Transportation	No
Illinois Department of Transportation	No
Iowa Department of Transportation	No
Maryland SHA, Office of Structures	No
MassDOT	No
Michigan Department of Transportation	No
Minnesota Department of Transportation	No
Missouri Department of Transportation	No
Nevada Dept of Transportation	No
New Jersey Department of Transportation	No
New Mexico Department of Transportation	No
North Carolina Department of Transportation	No
North Dakota Department of Transportation	No
NYS DOT	No
Ohio Department of Transportation	No

Responding Agency	Y/N
Oklahoma Department of Transportation (ODOT)	No
Oregon Department of Transportation	No
Pennsylvania Department of Transportation	No
SDDOT	No
State of Florida Department of Transportation	No
Texas Department of Transportation	No
Utah Department of Transportation	No
Vermont Agency of Transportation	No
Virginia Department of Transportation	No
Washington State Department of Transportation	No
Wisconsin DOT	No
WYDOT	No

12. Do you provide guidance or design criteria for the strength and stability checks for cantilever girder sections during lifting and placement?

Responding Agency	Y/N	Response
Arizona Department of Transportation	No	
Arkansas Department of Transportation	No	
CA Department of Transportation	No	
CDOT, Staff Bridge Branch	No	
Delaware DOT	No	
Georgia Department of Transportation	No	
Illinois Department of Transportation	No	
Iowa Department of Transportation	No	
Kansas Department of Transportation	Yes	KDOT Design Manual. $L/D \leq 85$
Maryland SHA, Office of Structures	No	
MassDOT	No	
Michigan Department of Transportation	No	
Minnesota Department of Transportation	No	
Missouri Department of Transportation	No	
Nevada Dept of Transportation	No	
New Jersey Department of Transportation	No	
New Mexico Department of Transportation	No	
North Carolina Department of Transportation	No	
North Dakota Department of Transportation	No	
NYSDOT	No	
Ohio Department of Transportation	No	

Responding Agency	Y/N	Response
Oklahoma Department of Transportation (ODOT)	No	
Oregon Department of Transportation	No	
Pennsylvania Department of Transportation	No	
SDDOT	No	
State of Florida Department of Transportation	No	
Texas Department of Transportation	No	
Utah Department of Transportation	No	
Vermont Agency of Transportation	No	
Virginia Department of Transportation	Yes	
Washington State Department of Transportation	No	
Wisconsin DOT	No	
WYDOT	No	

13. Do you require a bridge demolition procedure to be submitted for bridge removal?

Responding Agency	Y/N
Arizona Department of Transportation	Yes
Arkansas Department of Transportation	Yes
CA Department of Transportation	Yes
CDOT, Staff Bridge Branch	Yes
Delaware DOT	Yes
Georgia Department of Transportation	No
Illinois Department of Transportation	Yes
Iowa Department of Transportation	Yes
Kansas Department of Transportation	Yes
Maryland SHA, Office of Structures	Yes
MassDOT	Yes
Michigan Department of Transportation	No
Minnesota Department of Transportation	Yes
Missouri Department of Transportation	No
Nevada Dept of Transportation	Yes
New Jersey Department of Transportation	Yes
New Mexico Department of Transportation	Yes
North Carolina Department of Transportation	Yes
North Dakota Department of Transportation	Yes
NYSDOT	Yes
Ohio Department of Transportation	Yes
Oklahoma Department of Transportation (ODOT)	Yes

Responding Agency	Y/N
Oregon Department of Transportation	Yes
Pennsylvania Department of Transportation	Yes
SDDOT	No
State of Florida Department of Transportation	Yes
Texas Department of Transportation	Yes
Utah Department of Transportation	Yes
Vermont Agency of Transportation	No
Virginia Department of Transportation	Yes
Washington State Department of Transportation	Yes
Wisconsin DOT	No
WYDOT	Yes

14. If the answer to question 12 is yes, is there a size, span length, geometry feature or other threshold consideration that triggers the requirement?

Responding Agency	Response
Arizona Department of Transportation	
Arkansas Department of Transportation	
CA Department of Transportation	All bridge demolition requires demolition plan and procedure.
CDOT, Staff Bridge Branch	n/a
Delaware DOT	No. We make that determination based on the complexity of the structure and the conditions surrounding the bridge being removed.
Georgia Department of Transportation	
Illinois Department of Transportation	No but for certain structures we require a Structural Assessment Report (see link). http://www.idot.illinois.gov/Assets/uploads/files/Doing-Business/Manuals-Guides-&-Handbooks/Highways/Bridges/Bridge-Special-Provisions/qbsp67.pdf
Iowa Department of Transportation	Only for special cases as determined by the Engineer.
Kansas Department of Transportation	Yes. See Figure 736-1 in 07-070004-R03 http://www.ksdot.org/burconsmain/specprov/2007/pdf/07-07004-r03.pdf
Maryland SHA, Office of Structures	All bridge reconstruction and deck replacement contractions require a demolition plan
MassDOT	No
Michigan Department of Transportation	This would be on a case by case basis depending on the project, and if stability were an issue during lifting and placement, procedures would be shown in the plans.
Minnesota Department of Transportation	No
Missouri Department of Transportation	n/a
Nevada Dept of Transportation	No

Responding Agency	Response
New Jersey Department of Transportation	Information unknown – job to job basis.
New Mexico Department of Transportation	For existing steel girder bridges that are fracture critical and/or have fatigue cracks. We had a bridge collapse on a contractor during deck removal. The pin & hanger connections on the steel girders snapped and the bridge came down. Because of this, we now require a bridge removal plan for these types of bridges.
North Carolina Department of Transportation	Required when work is over the railway right-of-way
North Dakota Department of Transportation	If only a portion of the existing bridge is to be removed and traffic will be carried by the remaining portion.
NYS DOT	Designer specifies need
Ohio Department of Transportation	A plan submission is required when the demolition is over or adjacent to active traffic.
Oklahoma Department of Transportation (ODOT)	n/a
Oregon Department of Transportation	
Pennsylvania Department of Transportation	Any bridge with a span over 80 feet long and any bridge over live traffic.
SDDOT	n/a
State of Florida Department of Transportation	n/a
Texas Department of Transportation	Demolition plans are not normally required but can be for more complex structures (trusses, etc.).
Utah Department of Transportation	
Vermont Agency of Transportation	n/a
Virginia Department of Transportation	Depends on the type of structure given consideration to its span length, beam/girder types, bridge geometry (straight, high skewed, curved) simple or multi-span, and its behavior to type of substructure (i.e. boxed pier caps, prestressed/ post-tensioned)
Washington State Department of Transportation	No, we review and approve all demolition plans, including overhangs, rail and damaged girder replacements
Wisconsin DOT	n/a

Responding Agency	Response
WYDOT	No, case by case depending on structure type, what it crosses and for deck removal/replacement

15. Do you have requirements as to the qualifications of those who prepare the erection plan? If so, please provide.

Responding Agency	Y/N	Response
Arizona Department of Transportation	Yes	PE civil or structural
Arkansas Department of Transportation	No	
CA Department of Transportation	Yes	An engineer who is registered as a civil engineer in the State.
CDOT, Staff Bridge Branch	Yes	See link on last page
Delaware DOT	Yes	Must be stamped by profession engineer registered in the state of Delaware
Georgia Department of Transportation	No	
Illinois Department of Transportation	Yes	In Illinois, all require a Structural Engineer's seal and for certain bridges we require IDOT Prequalified Engineers (see link in Question 8 above).
Iowa Department of Transportation	Yes	Licensed Engineer
Kansas Department of Transportation	Yes	Contractor's Engineer
Maryland SHA, Office of Structures	Yes	When calculations and other information are deemed necessary to backup erection plans, they shall be signed and sealed by a Md. PE.
MassDOT	Yes	The method and all submissions shall be prepared under the supervision of a professional engineer, registered in Massachusetts, who is familiar with these Specifications, AASHTO, the work, and experienced in this technical field.
Michigan Department of Transportation	No	
Minnesota Department of Transportation	Yes	Preparation of the erection plan needs to be done by a registered Professional Engineer in the state of Minnesota.
Missouri Department of Transportation	Yes	We require the falsework to be approved by a Registered Professional Engineer. Any erection plans needed are provided by the fabricator. Fabricators provide the shop drawings, which are reviewed by MoDOT.

Responding Agency	Y/N	Response
Nevada Dept of Transportation	No	
New Jersey Department of Transportation	Yes	P.E. Required
New Mexico Department of Transportation	Yes	Section 1 of ftp://ftp.mdt.mt.gov/research/LIBRARY/NSBASBEGS-1-OL-STEEL_BRIDGE-AASHTO.PDF . We also require the erector to have 5 years' experience and have completed construction of a minimum 2 bridges with high skew or curved steel within the last 5 years.
North Carolina Department of Transportation	Yes	Professional Engineer licensed by the State of North Carolina.
North Dakota Department of Transportation	No	
NYSDOT	Yes	
Ohio Department of Transportation	Yes	Plans shall be prepared, signed, sealed, and dated by an Ohio registered Professional Engineer and shall be checked, signed, sealed and dated by a second Ohio registered Professional Engineer.
Oklahoma Department of Transportation (ODOT)	No	
Oregon Department of Transportation	Yes	The drawings and specifications shall be provided by a registered professional engineer licensed in the State of Oregon.
Pennsylvania Department of Transportation	Yes	The plans must be signed by a PE registered in Pennsylvania.
SDDOT	Yes	P.E.
State of Florida Department of Transportation	Yes	Must be prepared by a Professional Engineer Licensed in Florida
Texas Department of Transportation	Yes	Must be a P.E.
Utah Department of Transportation	Yes	PE or SE is required
Vermont Agency of Transportation	Yes	PE.
Virginia Department of Transportation	Yes	Given structure type, we may/will enforce Engineer in performing the task per VDOT Road and Bridge Specifications

Responding Agency	Y/N	Response
Washington State Department of Transportation	Yes	Licensed Professional Engineer in the State of Washington
Wisconsin DOT	No	
WYDOT	Yes	WY PE with experience in the type of work

16. Do you have requirements as to the qualifications of those who prepare the demolition plan? If so, please provide.

Responding Agency	Y/N	Response
Arizona Department of Transportation	No	
Arkansas Department of Transportation	No	
CA Department of Transportation	Yes	An engineer who is registered as a civil engineer in the State.
CDOT, Staff Bridge Branch	Yes	
Delaware DOT	Yes	Must be stamped by profession engineer registered in the state of Delaware
Georgia Department of Transportation	No	
Illinois Department of Transportation	Yes	In Illinois, we require a Structural Engineer's seal.
Iowa Department of Transportation	Yes	Licensed Engineer
Kansas Department of Transportation	Yes	Contractor's Engineer
Maryland SHA, Office of Structures	Yes	When calculations and other information are deemed necessary to backup erection plans, they shall be signed and sealed by a Md. PE.
MassDOT	Yes	Same as for 15.
Michigan Department of Transportation		
Minnesota Department of Transportation	No	
Missouri Department of Transportation	No	
Nevada Dept of Transportation	No	
New Jersey Department of Transportation	Yes	P.E. Required

Responding Agency	Y/N	Response
New Mexico Department of Transportation	Yes	Must be a plan stamped by a Professional Engineer in NM. The P.E. must have a minimum 5 years' experience in this type of work and have done a minimum of 2 projects within the last 5 years.
North Carolina Department of Transportation	Yes	Professional Engineer licensed by the State of North Carolina. This requirement is only for activities that have the capacity to affect travel ways- vehicular or navigable
North Dakota Department of Transportation	No	
NYSDOT	Yes	See Comment
Ohio Department of Transportation	Yes	Plans shall be prepared, signed, sealed, and dated by an Ohio registered Professional Engineer and shall be checked, signed, sealed and dated by a second Ohio registered Professional Engineer.
Oklahoma Department of Transportation (ODOT)	No	
Oregon Department of Transportation	Yes	Depend on size, type and location of the bridge requirement may vary however an engineer registered in the State of Oregon needs to sign and stamp the demolition drawings and specifications.
Pennsylvania Department of Transportation	Yes	The plans must be signed by a PE registered in Pennsylvania.
SDDOT	No	
State of Florida Department of Transportation	Yes	Qualifications are only required for highly sophisticated demolition projects (particularly if explosive demolition is permitted) and then FDOT requires a Professional Engineer Licensed in Florida to develop the plan and this is determined on a project by project basis.
Texas Department of Transportation	Yes	Must be a P.E.
Utah Department of Transportation	Yes	PE or SE is required
Vermont Agency of Transportation	No	
Virginia Department of Transportation	Yes	Given structure type, we may/will enforce Engineer in performing the task per VDOT Road and Bridge Specifications
Washington State Department of Transportation	Yes	Licensed Professional Engineer in the State of Washington
Wisconsin DOT	No	

Responding Agency	Y/N	Response
WYDOT	Yes	WY PE

17. Do you have requirements as to the level/degree of engineering analysis that is performed by the erector? If yes, please provide a copy or link.

Responding Agency	Y/N	Response
Arizona Department of Transportation	No	
Arkansas Department of Transportation	No	
CA Department of Transportation	No	
CDOT, Staff Bridge Branch	Yes	To some degree, it is based on the requirement of the specification for Girder Erection, See link on last page
Delaware DOT	No	
Georgia Department of Transportation	No	
Illinois Department of Transportation	No	
Iowa Department of Transportation	No	
Kansas Department of Transportation	Yes	Special Provision 07-07004-R03 http://www.ksdot.org/burconsmain/specprov/2007/pdf/07-07004-r03.pdf
Maryland SHA, Office of Structures	No	
MassDOT	Yes	See link for 8
Michigan Department of Transportation	No	
Minnesota Department of Transportation	No	
Missouri Department of Transportation	No	Registered Professional Engineer
Nevada Dept of Transportation	No	
New Jersey Department of Transportation	Yes	P.E. Required
New Mexico Department of Transportation	Yes	Section 2 of ftp://ftp.mdt.mt.gov/research/LIBRARY/NSBASBEGS-1-OL-STEEL_BRIDGE-AASHTO.PDF .
North Carolina Department of Transportation	No	

Responding Agency	Y/N	Response
North Dakota Department of Transportation	No	
NYSDOT	No	
Ohio Department of Transportation	Yes	See the response to Question #8 above.
Oklahoma Department of Transportation (ODOT)	No	
Oregon Department of Transportation	No	
Pennsylvania Department of Transportation	No	
SDDOT	No	
State of Florida Department of Transportation	Yes	For beam and girder temporary bracing, the erector must comply with the AASHTO Guide Design Specifications for Bridge Temporary Works and Construction Handbook for Bridge Temporary Works.
Texas Department of Transportation	No	Not at this time but we are working on a list of checks to be performed for a future specs
Utah Department of Transportation	Yes	http://www.udot.utah.gov/main/uconowner.gf?n=7602520459537371
Vermont Agency of Transportation	No	
Virginia Department of Transportation	No	But, in certain projects due to structure erection complexities, the contract plans will show and determine the necessary requirements as needed
Washington State Department of Transportation	Yes	Same as question 8
Wisconsin DOT	No	
WYDOT	No	

18. Who reviews erector supplied erection plans?

Responding Agency	Response
Arizona Department of Transportation	Resident Engineer
Arkansas Department of Transportation	Bridge Construction Engineer
CA Department of Transportation	Resident Engineer with the assistance of Maintenance Engineer if crane loads the existing bridge.
CDOT, Staff Bridge Branch	Participants of the Pre Girder Erection meeting review the plan and comment.
Delaware DOT	Design engineer of record
Georgia Department of Transportation	The Design Group responsible for the bridge plans.
Illinois Department of Transportation	Our Bridge design and Construction Review Unit.
Iowa Department of Transportation	Design Engineer
Kansas Department of Transportation	This depends on the category of the erection.
Maryland SHA, Office of Structures	A Transportation Engineer with a bachelor's degree in engineering from an accredited college and a minimum of two years experience in professional engineering.
MassDOT	The designer of record of the bridge project on behalf of the DOT
Michigan Department of Transportation	The design engineer of record.
Minnesota Department of Transportation	The design engineer (engineer of record) and fabrication engineer (shop plan reviewer).
Missouri Department of Transportation	The fabricator or the contractor is responsible for erection plans.
Nevada Dept of Transportation	Department personnel, or a design consultant hired for construction support
New Jersey Department of Transportation	Design Consultants or in-house design.
New Mexico Department of Transportation	NMDOT Bridge Bureau P. E. or a Consultant P.E. hired by NMDOT.

Responding Agency	Response
North Carolina Department of Transportation	NCDOT Structures Management Unit has a designated Working Drawing Review & Approval Section. For the railway projects the General Engineering Consultant for the railway.
North Dakota Department of Transportation	
NYSDOT	Main Office Metals Engineering Unit (steel structures) or Region Bridge Design (concrete structures)
Ohio Department of Transportation	For projects with railroad involvement, the plans require railroad approval which is typically performed by an independent consultant retained by the railroad. For projects without railroad involvement, the plans require design and checking by two different Ohio Registered Professional Engineers. Department review and approval is not required.
Oklahoma Department of Transportation (ODOT)	ODOT Bridge Division
Oregon Department of Transportation	The EOR of the designed bridge.
Pennsylvania Department of Transportation	For most bridges, the reviews are conducted by the DOT construction personnel. For larger projects, consultants are utilized to review erection procedures.
SDDOT	Bridge Construction Engineer
State of Florida Department of Transportation	Usually the construction management staff (FDOT employees or Construction Engineering and Inspection Consultants hired by FDOT) and for some situations the Engineer of Record, FDOT State Construction Office, FDOT State Structures Design Office or FDOT District Structures Design Office.
Texas Department of Transportation	Bridge Division, Bridge Construction & Maintenance Branch (90%)
Utah Department of Transportation	Engineer of Record
Vermont Agency of Transportation	VTrans construction personnel typically with additional review by Structures Design as requested.
Virginia Department of Transportation	VDOT Engineer or Agents of VDOT
Washington State Department of Transportation	The Bridge Construction Support Team, within the Bridge & Structures Offices, composed of experienced Professional Engineers familiar with all types of temporary construction submittals.

Responding Agency	Response
Wisconsin DOT	Staff in our fabrication unit sometimes review erection plans but they do not approve them.
WYDOT	Design Engineer

APPENDIX D

RECOMMENDED ENGINEERING CRITERIA

The plans and procedures for bridge construction shall be in conformance with applicable provisions of the AASHTO LRFD Bridge Construction Specifications, the supplementary criteria presented herein, and any other additional requirements provided by the Owner. Further discussion of these engineering criteria is contained in the referenced sections of the Manual that are indicated in parenthesis.

The term Engineer as used herein refers to the Contractor's Engineer charged with the development of the Contractor's construction plan and/or the Engineer charged with performing a detailed investigation of the anticipated erection sequence for the bridge superstructure.

D-1. MEMBER AND COMPONENT EVALUATION

D-1.1 GENERAL

Girder-bridge superstructures shall be constructed in such a way that strength and stability is maintained at all intermediate stages until completion. All members shall be lifted, supported, connected, and braced in such a way that no limit states are violated at any time and damage such as yielding, buckling and/or concrete cracking is avoided. Stability shall include local, member, system and rigid body (rollover) stability.

The calculations supporting the plans and procedures shall be in accordance with the AASHTO LRFD Bridge Design Specifications (referred to herein as the AASHTO LRFD BDS), AASHTO LRFD Bridge Construction Specifications, AASHTO Guide Specifications for Temporary Works, and the following supplementary provisions. Nominal yielding or reliance on post-buckling resistance shall not be permitted for main steel load-carrying members for all construction conditions, except for potential local yielding of the web in hybrid sections.

Sections in the Manual that provide additional information and commentary related to the provisions in this Criteria are referenced throughout.

As a minimum, bridge members and partially completed structures shall be evaluated at all of the following stages of construction (ref. Manual 7.2.2):

1. Girder lifting
2. Placement of the initial girder and any associated temporary bracing used to hold the girder in place
3. First pair of girders set with permanent bracing installed; and subsequent stages of girder erection which the Engineer deems critical
4. All girders and bracing installed prior to deck placement

The effects of wind on the behavior shall be considered during all of the above stages.

Stability during the following stages also shall be evaluated where the Contractor's construction plan related to these stages differs from what is shown in the Contract Documents:

- Deck placement with bracing fully installed between all girders
- Application of the deck overhang bracket loads to the fascia girders during the deck placement

Wind loads may be neglected in evaluating the fully erected system of girders for concrete loads due to deck placement if Contractor is precluded from work when winds in excess of 20 mph are forecast.

D-1.2 ANALYSIS

Analysis methods used (ref. Manual 7.2) shall be sufficiently refined to accurately evaluate the applicable force effects and limit states for each stage of girder erection and deck placement. For lifting/setting of a single girder, a line girder analysis may be satisfactory. Refined grid or 3D analysis methods shall be utilized for the evaluation of multi-girder systems, unless it is determined that girder system interaction effects can be neglected.

A global stability analysis shall be conducted to verify adequate stability when any procedures are being utilized for which the stability condition is not known, by either engineering judgment or documented experience on bridges of similar span, slenderness, lateral stiffness and bracing.

In cases where the force effects approach the elastic critical buckling limit, the effects of a potential second-order amplification of the response shall be considered. This may be addressed by a second-order analysis, or by supplemental bracing measures.

The boundary conditions assumed in the analysis model shall be representative of those specified in the erection plans, and provided in the field. The boundary conditions shall recognize the absence of any vertical restraint in the investigation of uplift scenarios. Girder twist shall be restrained at all support locations at all stages. This may induce fit-up forces for some detailing provisions.

D-2. LOAD COMBINATIONS AND LOAD FACTORS

The totaled factored force effect (ref. Manual 7.3) at each stage of construction shall be taken as:

$$Q = \sum Y_i Q_i \quad (D-2)$$

where:

Y_i = Load factors specified in Table D-1

Q_i = Force effects from the construction design loads specified in Section D-3

Table D-1 Load Combinations and Load Factors

	DC	C_{DL}	C_{LL} ^(d)	C_w
Strength I	1.25	1.50	1.50	—
Strength III	1.25	1.25	—	1.0 ^(b)
Strength VI ^(a)	1.40	1.40	1.40	—
Service	1.00	1.00	1.00	0.7 ^(b)
Uplift ^(c) /Overturning	0.90/1.35	0.90/1.35	—	1.0 ^(b)

- a) Steel structures for only the case of placing the deck on the fully erected steel. Use Strength I or III for intermediate steel conditions, as applicable.
- b) The specified load factor of 1.0 for wind load force effects for the strength load combinations is based on basic wind speed maps wherein an appropriate safety margin is built into the maps. Wind loads may be computed using a wind velocity based on the Wind Velocity Modification Factor specified in Table D-3.2 for the construction phase duration under investigation. Note that the wind load factor is less than 1.0 for the service load combination.
- c) Where a construction design load produces uplift/overturning at the location being investigated, the maximum specified load factor shall be applied to the load force effect and where the load resists uplift/overturning, the force effect shall be multiplied by the minimum specified load factor.
- d) An appropriate dynamic load allowance shall be considered for C_{LL} where applicable.

Resistance factors, ϕ , for the strength limit state shall be taken in accordance with the AASHTO LRFD BDS, Articles 6.5.4.2 and 5.5.4.2, as applicable. Additional resistance factors applied herein include:

ϕ_{b} , resistance factor for lateral torsional buckling = 0.90

ϕ_{br} , resistance factor for steel girder bracing stiffness = 0.75

ϕ_{bk} , resistance factor for girder system buckling = 0.90

Safety margins on lifting accessories, jacks, and other manufactured items shall be in accordance with manufacturer's recommendations.

For evaluation of ancillary items, lifting devices, and other conditions for which LRFD specifications do not exist, the resistance factors used shall be documented and submitted to the Owner for approval.

D-3. LOADS

D-3.1 PERMANENT DEAD LOADS (C_D)

Permanent Dead Loads (ref. Manual 7.4.2) include the weight of all permanent in-place components of the partially completed structure as computed in accordance with the AASHTO LRFD BDS design specifications, including the weight of the deck concrete during placement and any stay-in-place forms. In the absence of more precise information, material unit weights used in the computation of permanent dead loads may be taken from Table 3.5.1-1 of the AASHTO LRFD BDS. The weight of stiffeners, splices, studs, bolts, paint and other miscellaneous items may be taken as an equivalent uniformly distributed weight along the girder, or accounted for using an increased material density. In lieu of a project-specific value, the following may be assumed for the weight of stay-in-place formwork.

Stay-in-place corrugated metal formwork = 20 psf
Stay-in-place concrete plank formwork = as calculated

D-3.2 CONSTRUCTION DEAD LOADS (C_{DL})

Construction Dead Loads (ref. Manual 7.4.3) include the weight of removable temporary construction supported by the partially completed structure.

Removable Formwork = 10 psf
Cantilever Formwork = as calculated
Protective Shielding = as calculated
Other = as calculated

D-3.3 CONSTRUCTION LIVE LOADS (C_{LL})

Construction live loads (ref. Manual 7.4.3.1) include the weight of workers, miscellaneous tools and supplies, materials and equipment that are only on the bridge during construction. Suggested construction live loads that might be considered include the following:

Workers and light tools ^a = 20 psf
Overhang live load ^b = 75 plf

Materials = actual weight
 Additional uniform live load (when motorized
 buggies are used to place concrete)^a = 25 psf

^a = applied to concrete deck area

^b = applied to outside edge of each concrete deck overhang

Screed machine = per manufacturer's data
 Other equipment = per manufacturer's data

Dynamic Load Allowance

Moving and stationary equipment
 = minimum 10% of equipment weight for moving equipment
 (except screed machine)
 = minimum 10% of equipment operating load for stationary
 equipment

Crane lift as a part of member removal (demolition)
 = minimum 20% of the calculated member weight

Accidental release for segmental concrete girders shall be considered as specified in AASHTO LRFD BDS Article 5.14.2.3.2.

Incidental loads = 5 psf lateral load (minimum) over the vertical face of the girder applied at the centroid of the loaded area. Load factor for C_w shall apply.

D-3.4 WIND LOADS (C_w)

The construction horizontal wind load (ref. Manual 7.4.4), C_w , shall be applied at the centroid of the exposed projected area of the windward girder/truss and shall be computed as follows:

$$C_w = q_z G C_f A_f \quad (D-3.4a)$$

where:

q_z = velocity pressure at height z above grade (Equation D-3.4b) (psf)

G = gust effect factor, use 0.85

C_f = wind net force coefficient (Tables D-3.3 or D-3.4, as applicable)

A_f = exposed projected area of girder or truss (ft²)

The effects of superelevation and horizontal curvature shall be considered in determining the exposed projected area.

Velocity pressure, q_z , evaluated at height z shall be taken as:

$$q_z = 0.00256 K_z K_{zt} K_d (V_m V)^2 \quad (\text{D-3.4b})$$

Note: $V_m V = 20$ mph minimum

where:

K_z = wind velocity pressure exposure coefficient (Table D-3.1)

K_{zt} = topographic factor, use 1.0

K_d = wind directionality factor, use 0.85

V = basic wind speed (mph)

V_m = wind velocity modification factor (Table D-3.2)

z = height of the top of the bridge deck above grade/water (ft)

Height above ground level, z Exposure Category

Table D-3.1 Velocity Pressure Exposure Coefficient, K_z

Height above ground level, z		Exposure Category		
Ft	(m)	B	C	D
0-15	(0-4.6)	0.57	0.85	1.03
20	(6.1)	0.62	0.90	1.08
25	(7.6)	0.66	0.94	1.12
30	(9.1)	0.70	0.98	1.16
40	(12.2)	0.76	1.04	1.22
50	(15.2)	0.81	1.09	1.27
60	(18)	0.85	1.13	1.31
70	(21.3)	0.89	1.17	1.34
80	(24.4)	0.93	1.21	1.38
90	(27.4)	0.96	1.24	1.40
100	(30.5)	0.99	1.26	1.43
120	(36.6)	1.04	1.31	1.48
140	(42.7)	1.09	1.36	1.52
160	(48.8)	1.13	1.39	1.55
180	(54.9)	1.17	1.43	1.58
200	(61.0)	1.20	1.46	1.61
250	(76.2)	1.28	1.53	1.68
300	(91.4)	1.35	1.59	1.73
350	(106.7)	1.41	1.64	1.78
400	(121.9)	1.47	1.69	1.82
450	(137.2)	1.52	1.73	1.86
500	(152.4)	1.56	1.77	1.89

*For exposure Category descriptions, see Reference Manual 7.4.4)

Table D-3.2 Wind Velocity Modification Factors, V_m

Construction Duration	Velocity Modification Factor
0 – 6 weeks	0.65
6 weeks – 1 year	0.75
1 year – 2 years	0.80
2 years – 5 years	0.85

Note: wind load may be neglected during girder lifting and during deck casting if Contractor is precluded from work when winds in excess of 20 mph are forecast.

Table D-3.3 Wind Net Force Coefficient, C_f , (For Girder Bridges During Construction)

Component Type	Construction Condition	Force Coefficient
I-Shaped Girder Superstructure	Deck forms not in place	2.2*
	Deck forms in place	1.1
U-Shaped and Box-Girder Superstructure	Deck forms not in place	1.5
	Deck forms in place	1.1
Flat Slab or Segmental Box-Girder Superstructure	Any	1.1

*When s/d is greater than 2.0, $C_f = 2(1 + 0.05 s/d) \leq 4$

Where

s = girder spacing

d = girder height

Table D-3.4 Wind Net Force Coefficient, C_f (For Truss Bridges During Construction)

ϵ	C_f
<0.1	2
0.1 to 0.29	1.8
0.29 to 0.7	1.7

Where

$\epsilon = \frac{\text{Solid Area of Truss Members}}{\text{Gross Area of Truss}}$

D-3.5 SEISMIC LOADS

Seismic Loads (ref. Manual 7.4.5) shall not be considered unless required by the bridge Owner. If required, the loads shall comply with the Owner's requirements.

D-4. GIRDER LIFTING

D-4.1 GENERAL

Lifting procedures (ref. Manual 7.5) shall be developed to ensure that the girder has adequate strength and buckling resistance during lifting operations. Girder weight shall be based on shop drawing weight or computed from member dimensions accounting for fabrication tolerances in finished dimensions. Steel girders should be lifted near their quarter points whenever possible. Where a spreader beam is employed, the line of support, or line running through the girder lifting points, should pass through the center of gravity of the member, and the lifting reactions at each pick point should be equal. Cross-frames attached to the girders prior to lifting, and any other variations to the girder configuration shall be considered in determining the center of gravity.

The girder bending moments shall be determined treating the girder as simply supported at the lift points. Lift points shall not be assumed to provide any lateral-torsional restraint.

D-4.2 STEEL GIRDERS

Straight steel I-girders, or horizontally curved steel I-girders (ref. Manual 7.5.2) where the central angle subtended by the girder length is less than 3 degrees, that are doubly symmetric and lifted at two points, shall satisfy the following:

$$M_u < \phi_b M_{cr} = \phi_b C_{bL} \frac{\pi}{L} \sqrt{EI_y GJ + E^2 I_y C_w \left(\frac{\pi^2}{L^2} \right)} \quad (\text{D-4.2a})$$

in which:

C_{bL} = Moment gradient magnifier during lifting

$$C_{bL} = 2.0 \text{ for } \frac{L_{Lift}}{L} \leq 0.225$$

$$C_{bL} = 6.0 \text{ for } 0.225 < \frac{L_{Lift}}{L} < 0.3$$

$$C_{bL} = 4.0 \text{ for } \frac{L_{Lift}}{L} \geq 0.3$$

where:

M_u = Factored maximum moment from static analysis (kip-in)

M_{cr} = Nominal lateral-torsional buckling resistance (kip-in.)

ϕ_b = Resistance factor for lateral torsional buckling = 0.9

L = Unbraced length = (total length of girder segment) (in)

I_y = Moment of inertia of the girder about the vertical axis in the plane of the web (in⁴)

L_{Lift} = Average length from the lift points to the ends of the girder (in)

G = Shear modulus, 11,200 (ksi)

J = Torsional constant (in⁴)

C_w = Warping constant (in⁶)

For singly symmetric sections the flexural resistance based on lateral torsional buckling shall be computed in accordance with AASHTO LRFD BDS Appendix D-6.4.2, where C_{bL} as computed above shall replace the C_b term.

D-4.3 CONCRETE GIRDERS

Concrete girders shall be stable and not crack during lifting. The roll stability (ref. Manual 7.5) of concrete I-girders shall satisfy Equation D-4.3a and the cracking shall satisfy Equation D-4.3b.

$$\text{Rollover: } 1.5 \leq \left[\frac{y_r \theta'_{\max}}{\bar{z}'_o \theta'_{\max} + e_i} \right] \quad (\text{D-4.3a})$$

$$\text{Cracking: } 1.0 \leq 1 \div \left[\frac{\bar{z}'_o}{y_r} + \frac{\theta_i}{\theta_{\max}} \right] \quad (\text{D-4.3b})$$

in which:

$$\theta'_{\max} = \sqrt{\frac{e_i}{2.5 \bar{z}'_o}} \quad (\text{D-4.3c})$$

$$\theta_{\max} = \frac{M_{lat}}{M_g} \quad (\text{D-4.3d})$$

$$\theta_i = \frac{e_i}{y_r} \quad (\text{D-4.3e})$$

where:

θ_i = initial roll angle of rigid beam (rad)

e_i = the initial lateral eccentricity of the center of gravity with respect to the roll axis (rad).

y_r = the height of the roll axis above the center of gravity of the beam (in).

\bar{z}_o = the theoretical lateral deflection of the center of gravity of the beam, computed with the full weight applied as a lateral load, measured to the center of gravity for the deflected arc of the beam (in).

θ_{\max} = tilt angle at which cracking begins, based on tension at the top corner equal to the modulus of rupture (rad)

θ'_{\max} = tilt angle at rollover (rad)

M_g = service level strong-axis moment in girder due to selfweight (kip-in)

M_{lat} = service level weak-axis moment that would cause cracking in top flange of girder (kip-in)

The assumed initial eccentricity, e_i , shall be taken as a minimum of ¼ inch plus 1/8 inch for each 10 feet of beam length or fraction thereof.

For a beam with overall length, l , and equal overhangs beyond the lifting points of length, a , at each end:

$$\bar{z}_o = \frac{w}{12EI_y l} \left[0.1(l_1)^5 - a^2(l_1)^3 + 3a^4(l_1) + 1.2(a^5) \right] \quad (\text{D-4.3f})$$

where:

$l_1 = l - 2a$ (in)

I_y = moment of inertia of beam about weak axis (in⁴)

w = girder weight per unit length (kip/in)

\bar{z}'_o = lateral deflection of girder center of gravity including rotation effects (in) = $\bar{z}_o (1 + 2.5 \theta'_{\max})$

D-5. GIRDER STABILITY

D-5.1 GENERAL

The Contractor shall ensure that girders are stable throughout the erection process. Prior to release of a girder from its lifting line, adequate bracing shall be installed, as necessary, to provide girder stability. Bracing shall be designed for vertical and lateral load effects and girder stability forces. Bracing shall be evaluated for both strength and

stiffness as specified herein. Girder twist shall be restrained at all support locations at all stages.

Steel girders shall be evaluated for local, member, and global stability as specified herein and the bracing details shall be defined at each construction stage under investigation. The stage of completeness of all bolted connections shall be considered when evaluating the strength and stability of the steel during erection.

The Contractor shall ensure that sufficient internal cross-frames or diaphragms are provided to control cross-sectional distortion throughout construction. Top lateral bracing of tub girders should be installed in the girders prior to shipping and erection of the field pieces.

Steel I-girder compression flanges and top flanges of tub girders subject to compression shall be evaluated for flange local buckling and for lateral-torsional buckling in-between brace points according to Eq. 6.10.3.2.1-2 of the AASHTO LRFD BDS.

Yielding of I-girder compression and tension flanges and top flanges of tub girders subject to compression and tension shall be evaluated according to Eqs. 6.10.3.2.1-1 and 6.10.3.2.2-1, respectively, of the AASHTO LRFD BDS, as applicable. Tub-girder bottom flanges subject to compression shall be evaluated for flange local buckling according to Eq. 6.11.3.2-1 of the AASHTO LRFD BDS.

Yielding of tub-girder bottom flanges subject to tension shall be evaluated according to Eq. 6.11.3.2-3 of the AASHTO LRFD BDS. Web bend-buckling shall be evaluated for I- and tub girders according to Eqs. 6.10.3.2.1-3 and 6.11.3.2-2, respectively, of the AASHTO LRFD BDS, as applicable.

D-5.2 STEEL GIRDER BRACING

Bracing members for steel girders (ref. Manual 5.3) shall be evaluated for the applicable provisions of Articles 6.8 and 6.9 of the AASHTO LRFD BDS. In addition to resisting all applied force effects, bracing members and systems for steel I-girders shall also satisfy the applicable stiffness and strength requirements specified below for a discrete torsional nodal, relative lateral or nodal bracing system.

For Torsional Nodal Bracing Provided by Cross-frames or Diaphragms:

$$M_{br} = F_{br} h_b > \frac{0.005 L_b L M_u^2}{n E I_{eff} C_b^2 h_o} \quad (D-5.2a)$$

$$\beta_T = \frac{\bar{\beta}_T L}{n} > \frac{2.4LM_u^2}{\phi_{br} n E I_{eff} C_b^2} \quad (D-5.2b)$$

For Relative Lateral Bracing Provided by a Lateral Truss on the Compression Flange:

$$P_{br} \geq 0.008 \left(\frac{M_u C_d}{h_o} \right) \quad (D-5.2c)$$

$$\beta_{br} \geq \frac{1}{\phi_{br}} \left(\frac{4M_u C_d}{L_b h_o} \right) \quad (D-5.2d)$$

For Nodal Lateral Bracing Provided by Laterally Bracing the Girders from a Rigid Structure such as an Adjacent Bridge:

$$P_{br} \geq 0.2 \left(\frac{M_u C_d}{h_o} \right) \quad (D-5.2e)$$

$$\beta_{br} \geq \frac{1}{\phi_{br}} \left(\frac{10M_u C_d}{L_b h_o} \right) \quad (D-5.2f)$$

in which:

$$\frac{1}{\beta_T} = \frac{1}{\beta_b} + \frac{1}{\beta_{sec}} + \frac{1}{\beta_g} \quad (D-5.2g)$$

$$\beta_{sec} = 3.3 \frac{E}{h_o} \left[\frac{(1.5h_o)t^3}{12} + \frac{t_s b_s^3}{12} \right] \quad (D-5.2h)$$

$$\beta_g = \frac{24(n_g - 1)^2}{n_g} \frac{S^2 E I_x}{L^3} \quad (D-5.2i)$$

$$I_{eff} = I_{yc} + (t/c) I_{yt} \quad (D-5.2j)$$

where:

β_T = torsional stiffness of the bracing system (k-in/rad)

β_b = stiffness of the cross-frame or diaphragm (k-in/rad)(ref. Manual Fig. 5-6 and 5-7)

β_{sec} = web distortional stiffness (k-in/rad)

β_g = in-plane girder system stiffness (k-in/rad)

β_T = continuous bracing stiffness (k-in/rad-in)

β_{br} = the required lateral bracing stiffness (k/in)

E = modulus of elasticity (29000 ksi for steel)

t_w = web thickness (in)

b_s = width of transverse web stiffener plates (in), (=total combined width for double-sided stiffeners and width of single plate for single sided stiffeners, ref. Manual Fig 5-8)

n_g = number of girders across the width of the bridge

S = girder spacing (in)

I_x = moment of inertia about the strong axis (in⁴)

L = span of the girders (in)

M_{br} = In-plane flexural resistance of cross-frame or diaphragm (k-in)

M_u = maximum factored girder moment within the unbraced length (k-in)

L = span length (in)

L_b = unbraced length (in)

h_o = distance between girder flange centroids (in)

h_b = vertical distance between cross-frame bracing chords (or work points) (in)

n = number of cross-frames or diaphragms in the span under consideration

I_{eff} = effective moment of inertia (in⁴)

I_{yc} and I_{yt} = moment of inertia of the compression and tension flange, respectively, of the steel section about a vertical axis through the plane of the web (in⁴)

c, t = distance from the centroid of the steel-girder cross-section to the centroid of the compression and tension flanges, respectively (in)

C_b = moment magnification factor (use $C_b = 1.0$ or AISC Chapter F to calculate alternate values)

F_{br} = required force couple in cross-frame or diaphragm (kip)

P_{br} = the required lateral bracing strength (kip)

For the lateral braces closest to the inflection point for a girder in reverse curvature:

$C_d = 2.0$

Otherwise:

$C_d = 1.0$

Φ_{br} resistance factor for steel girder bracing = 0.75

For all girder geometries except cantilevers, lateral braces shall be positioned as close to the compression flange as possible. For cantilever sections, lateral braces shall be positioned as close to the tension flange as possible (considering gravity loading, this will be the top flange).

D-5.3 CONCRETE GIRDER BRACING

Concrete girders (ref. Manual 7.7) shall be braced to satisfy the following (after setting the girder on the bearing pads):

$$\text{Rollover: } 1.5 \leq \frac{r(\theta'_{\max} - \alpha)}{\bar{z}'_{\max} \theta'_{\max} + e_i + y \theta'_{\max}} \quad (\text{D-5.3a})$$

$$\text{Cracking: } 1.0 \leq \frac{r(\theta_{\max} - \alpha)}{\bar{z}_o \theta_{\max} + e_i + y \theta_{\max}} \quad (\text{D-5.3b})$$

in which:

$$\bar{z}_o = \frac{1}{120} \frac{wL^4}{E_c I_y} \quad (\text{D-5.3c})$$

$$\theta_{\max} = \frac{M_{lat}}{M_g} \quad (\text{D-5.3d})$$

$$r = \frac{K_\theta}{W} \quad (\text{D-5.3e})$$

$$\theta'_{\max} = \frac{(z_{\max} - h_r \alpha)}{r} + \alpha \quad (\text{D-5.3f})$$

$$\bar{z}'_{\max} = (1 + 2.5\theta'_{\max}) \quad (\text{D-5.3g})$$

where:

e_i = initial lateral eccentricity of center of gravity with respect to roll axis (in) (min. of 1 inch + 1/8 inch for each 10 feet of girder length or fraction thereof)

h_r = distance from bottom of girder to roll axis (use one half of bearing pad thickness)(in)

\bar{z}_o = the theoretical lateral deflection of the center of gravity of the beam, computed with the full weight applied as a lateral load, measured to the center of gravity for the deflected arc of the beam (in)

θ_{max} = tilt angle at which cracking begins, based on tension at the top corner equal to the modulus of rupture (rad)

M_{lat} = service level weak-axis moment that would cause cracking in top flange of girder (k-in)

M_g = service level strong-axis moment in girder due to and selfweight (k-in)

w = girder weight per unit length (k/in)

E_c = modulus of elasticity of concrete (ksi)

I_y = moment of inertia of beam about weak axis (in⁴)

L = girder span (in)

Z_{max} = maximum resisting moment arm (typically half of bottom flange width) (in)

K_θ = sum of rotational spring constants of supports (k-in/rad)

W = total weight of beam (k)

y = height of center of gravity of beam above roll axis (in)

α = tilt angle of support (rad)

θ'_{max} = maximum roll angle at failure (rad)

r = radius of stability (in)

\bar{z}'_0 = lateral deflection including rotation effects (in)

In evaluating girder rollover, effects of camber, bearing pad skew, and cross slope shall be considered. For wind loading effects, the initial eccentricity, e_i , shall be increased by a value equal to the lateral deflection at midspan due to wind on the uncracked section, plus an eccentricity equal to the wind overturning moment divided by the girder selfweight.

D-5.4 STEEL GIRDERS GLOBAL STABILITY

Noncomposite steel I-girder systems consisting of two to four girders interconnected by cross-frames or diaphragms such that each of the girders interacts in developing the global lateral-torsional buckling resistance of the unit as a structural system shall also be evaluated to ensure that elastic lateral-torsional buckling of the system does not occur (ref. Manual 5.4). For elastic system buckling, singly and doubly symmetric girder systems shall satisfy the following:

$$M_u \leq \frac{\phi_{bk} M_{gs}}{n} = \phi_{bk} \left(\frac{\pi^2 SE}{L^2} \sqrt{I_y I_x} \right) / n \quad (D-5.4a)$$

where:

n = number of girders in the system

M_u = maximum factored applied moment in a single girder (k-in)

M_{gs} = nominal buckling resistance of the single girder system, (k-in)

S = girder spacing (in)

L = span length (in)

I_y, I_x = moments of inertia of a single girder about its weak and strong axis, respectively. (in⁴)

Φ_{bk} = resistance factor for system buckling = 0.9

E = modulus of elasticity of steel

For singly-symmetric girders, I_y shall be replaced with I_{eff} as follows:

$$I_{eff} = I_{yc} + \frac{t}{c} I_{yt} \quad (D-5.4b)$$

where:

I_{yc} and I_{yt} = moment of inertia of the compression and tension flange, respectively, of the steel section about a vertical axis through the plane of the web (in)⁴

c, t = distances from the centroid of the steel-girder cross-section to the centroid of the compression and tension flange, respectively (in)

For three-girder systems:

$$I_{yc} = 1.5 I_{yc} \quad (D-5.4c)$$

$$S = 2S$$

For four-girder systems:

$$I_{yc} = 2I_{yc} \quad (\text{D-5.4d})$$

$$S = 3S$$

Lateral load effects when combined with the system global buckling resistance calculated in D-5.4a through D-5.4d shall satisfy the AASHTO LRFD BDS Eq. A6.1.1-1 where the global buckling capacity, $\phi_{bk}M_{gs}/n$ from Equation D-5.4a, replaces the term $\phi_f M_{nc}$ on the right hand side of the equation.

The Yura equation: $M_{gs} = \frac{\pi^2 SE}{L^2} \sqrt{I_{eff} I_x}$ has recently been incorporated into AASHTO LRFD as Eq. 6.10.3.4.2-1 for checking system buckling during deck pour. The approach presented above is applicable for intermediate steel checks prior to the deck pour ($M_u \leq \phi M_{gs} / n$), but the AASHTO approach omits the ϕ factor and limits the total sum of the factored positive girder moments to 50% M_{gs} during the deck pour (Strength VI load combination). Should the sum of the moments exceed 50%, the design can add flange level lateral bracing, revise the girder spans/sizes to increase system stiffness, or evaluate the amplified girder second-order displacements and verify that they are within owner tolerances. Note that amplification can also occur under steel-only dead load as the buckling limit is approached, but the recommended system buckling ϕ factor and Strength I/III load factors should provide an adequate level of safety for most narrow systems subject to buckling in the steel-only condition.

D-6. CONCENTRATED LOADS EFFECTS

At bearing locations and at other locations on steel girders (ref. manual 7.8) subjected to concentrated loads, where the loads are not transmitted through a deck or deck system, webs without bearing stiffeners shall be investigated for the limit states of local web yielding, and local web crippling according to the provisions of Appendix D6.5 – *Concentrated Loads Applied to Webs without Bearing Stiffeners* of the AASHTO LRFD BDS. Web sidesway buckling, if applicable, shall be investigated according to AISC Specifications J10.4.

The local effect of lifting clamp loads on girder flanges shall be investigated. In lieu of a more refined local analysis, the local transverse flange bending stress may be computed as follows:

$$\frac{R_c k}{(b_f + C_L)(t_f)^2 / 6} \leq 0.75 F_{yf} \quad (\text{D-6})$$

where:

R_c = service level concentrated force at each flange edge (kip)

F_{yf} = specified minimum flange yield stress (ksi)

b_f = flange width (in)

t_f = flange thickness (in)

C_L = length of clamp along flange (in)

k = distance from outer face of flange to web toe of fillet (in)

D-7. DEFLECTION CONTROL

Girder deflections and rotations (ref. Manual 7.9) shall be controlled such that members fit-up can be achieved at each erection stage, and the finished bridge geometry conforms with the design plans. Deflections shall be investigated for service loads. Deflection and rotation limits for erection shall be established in consultation with the erector. Where no job specific criteria are provided, a maximum girder lateral deflection limit of the span divided by 150, and maximum girder torsional misalignments (twist) at field splice locations of 1.5 degrees shall be used.

D-8. STEEL CONNECTIONS

Connections between permanent and temporary components (ref. Manual 7.10.1) required as part of the erection operations shall be evaluated in accordance with the applicable provisions of Article 6.13 of the AASHTO LRFD BDS. Any proposed changes to the Owner's standard specified bolting procedures shall be supported by engineering calculations and be submitted to the Owner for acceptance. Existing connections and splices shall be evaluated for each stage of construction under consideration according to the same provisions, as applicable. Reaming of bolt holes during erection shall be permitted only with the approval of the Engineer.

D-9. CONCRETE CONNECTIONS

Anchorage and other connections (ref. Manual 7.10.2) to concrete structures or components required as part of the erection operations shall be designed in accordance with Appendix D of ACI 318, Building Code Requirements for Structural Concrete. Existing anchorages and connections shall be evaluated for each stage of construction under consideration according to the same provisions, as applicable.

D-10. TEMPORARY SUPPORTS (FALSEWORK)

Temporary supports and their associated components (ref. Manual 7.11) shall be designed to carry vertical and lateral loads due to self-weight and wind and any loads

that are applied to the temporary supports as the erection progresses. The design of temporary towers and falsework shall conform to the AASHTO Guide Specifications for Temporary Works. The effects of any longitudinal jacking during the erection shall also be investigated. The elevation of the temporary supports shall be such as to support the girders at their cambered no-load elevation. The use of temporary supports shall not result in any overstressing of the girders. Jacks used in conjunction with the temporary supports shall have a stroke adequate to permit full unloading. Unloading of temporary supports should be performed such that all temporary supports at each cross-section are unloaded uniformly. The deflections of the erected girders at the temporary supports when they are removed shall be evaluated, and stability of the girders shall be ensured prior to removal of the temporary supports. Where appropriate, holding cranes may be substituted for temporary supports.

D-11. BEARINGS

Computed bearing rotations (ref. Manual 7.12) during each stage of construction under investigation shall not exceed the rotational capacity of the bearing. Bearings shall be installed such that, after dead load has been applied, sufficient rotation capacity shall be available to accommodate rotations due to environmental loads and live loads. Expansion bearings shall be installed so that they will be in the center of the permitted travel at an ambient temperature of 60°F, unless otherwise specified by the Owner. Temporary blocking or restraints shall be provided as may be required to control bearing movement and assure stability at all construction stages. Note that for skewed supports out-of-plane rotations need to be considered in addition to in-plane rotations.

D-12. DECK

D-12.1 FORMS

Plywood, permanent metal forms or concrete panels may be used as deck forms, as approved by the Owner (ref. Manual 7.13). Proprietary forms shall be placed in accordance with the manufacturer's specifications incorporating any modifications to those specifications approved by the Engineer. Formwork shall be supported by the superstructure.

D-12.2 OVERHANGS

Overhang forms (ref. Manual 5.6) shall be removed after the deck has cured. Wherever practical, overhang brackets should bear near the bottom flange of the girder and be attached to the top flange of the girder. If overhangs bear against the girder web, particularly the compression zone of the web, the Engineer shall ensure that precautions have been taken to prevent permanent deformation of the web and excessive deflection of the wet slab and forms.

If the loads or their application are to be different than those provided for in the contract documents, an additional analysis shall be made by the Engineer. Loads applied on the overhang brackets shall be considered in determining and evaluating the lateral force on the top flange and the associated lateral flange bending stresses, cross-frame forces

and web and top flange deformations. The effects of the forces from deck overhang brackets acting on the fascia girders of steel-girder bridges shall be evaluated as specified in Article 6.10.3.4 of the AASHTO LFRD BDS.

D-12.3 DECK PLACEMENT

Concrete placements (ref. Manual 5.4) shall either be made in the sequence specified in the contract documents, or based on a sequence developed entirely by the Contractor, in which case the Engineer shall evaluate the effects of the desired placement sequence according to the criteria specified in Article 6.10.3.4 of the AASHTO LFRD BDS.

The duration of each placement shall be specified in the construction plan. The time between placements shall be such that the concrete in prior pours had reached an age or strength specified in the construction plan. Placements that include both negative and positive dead load moment regions should be placed such that the positive moment region is poured first. Any accelerating or retarding agents to be used in the concrete mix shall be specified.

GLOSSARY

Accepted Method of Analysis – a method of analysis that requires no further verification and that has become a regular part of structural engineering practice

Anchorage Zone – the portion of the structure in which the prestressing force is transferred from the anchorage device onto the local zone of the concrete, and then distributed more widely into the general zone of the structure

At Transfer – immediately after the transfer of prestressing force to the concrete

Axial – in line with the longitudinal axis of a member

Bascule bridge – a bridge over a waterway with one or two leaves which rotate from a horizontal to a near vertical position, providing unlimited overhead clearance

Base plate – steel plate, whether cast, rolled or forged, connected to a column, bearing, or other member to transmit and distribute its load to the superstructure

Beam – a structural member whose primary function is to transmit loads to the support primarily through flexure and shear. Generally this term is used when the component is made of rolled shapes.

Beam Column – a structural member whose primary function is to resist both axial loads and bending moments

Bearing – A structural device that transmits loads while facilitating translation and/or rotation

Bearing Capacity – the load per unit area which a structural material, rock or soil can safely carry

Bearing Failure – crushing of material under extreme compressive load

Bearing Plate – a steel plate, which transfers loads from the superstructure to the substructure.

Bend-Buckling Resistance – the maximum load that can be carried by a web plate without experiencing theoretical elastic local buckling due to bending

Bending Moment – the internal force within a beam resulting from transverse loading.

Biaxial Bending – simultaneous bending of a member or component about two perpendicular axes.

Bifurcation – the phenomenon whereby an ideally straight or flat member or component under compression may either assume a deflected position or may remain undeflected, or an ideally straight member under flexure may either deflect and twist out-of-plane or remain in its in-plane deflected position

Box Flange – a flange that is connected to two webs. The flange may be a flat unstiffened plate, a stiffened plate or a flat plate with reinforced concrete attached to the plate with shear connectors.

Box Girder – a hollow rectangular or trapezoidal shaped girder, a primary member along the longitudinal axis of the bridge, which provides good torsional rigidity

Bracing Member – a member intended to brace a main member or part thereof against lateral movement

Bracket – a projecting support fixed upon two intersecting members to strengthen and provide rigidity to the connection

Buckling Load – the load at which an ideally straight member or component under compression assumes a deflected position

Built-Up Member – a column or beam composed of plates and angles and other structural shapes united by bolting, riveting or welding to enhance section properties

Bulb T-Girder – a t-shaped concrete girder with a bulb shape at the bottom of the girder cross-section

Cable-Stayed Bridge – a bridge in which the superstructure is directly supported by cables, or stays, passing over or attached to towers located at the main piers.

Camber – the slightly arched or convex curvature provided in beams to compensate for dead load deflection

Cantilever – a structural member that has a free end projecting beyond a support; length of span overhanging the support

Cantilever Span – a superstructure span composed of two cantilever arms, or a suspended span supported by one or two cantilever arms

Cast-In-Place Concrete – concrete placed in its final location in the structure while still in a plastic state

Center of Gravity – the point at which the entire mass of a body acts; the balancing point of an object

Chord – a generally horizontal member of a truss or cross-frame

Closed-Box Section – a cross-section composed of two vertical or inclined webs which has at least one completely enclosed cell – a closed-section member is effective in resisting applied torsion by developing shear flow in the webs and flanges

Column – a general term applying to a vertical member resisting compressive stresses and having, in general, a considerable length in comparison with its transverse dimensions

Compact Flange – for a composite section in negative flexure or a noncomposite section, a discretely braced compression flange with a slenderness at or below which the flange can sustain sufficient strains such that the maximum potential flexural resistance is achieved prior to flange local buckling having a statistically significant influence on the response, provided that sufficient lateral bracing requirements are satisfied to develop the maximum potential flexural resistance

Compact Unbraced Length – for a composite section in negative flexure or a noncomposite section, the limiting unbraced length of a discretely braced compression flange at or below which the maximum potential flexural resistance can be achieved prior to lateral torsional buckling having a statistically significant influence on the response, provided that sufficient flange slenderness requirements are satisfied to develop the maximum potential flexural resistance

Complex Bridge – movable, suspension, cable stayed, and other bridges with unusual characteristics

Component – a constituent part of a structure

Composite Action – the contribution of a concrete deck to the moment resisting capacity of the superstructure beam when the superstructure beams are not the same material as the deck.

Composite Beam – a steel beam connected to a deck so that they respond to force effects as a unit

Composite Construction – concrete components or concrete and steel components interconnected to respond to force effects as a unit

Composite Girder – a steel flexural member connected to a concrete slab so that the steel element and the concrete slab, or the longitudinal reinforcement within the slab, respond to force effects as a unit

Compression – a type of stress involving pressing together; tends to shorten a member; opposite of tension

Compression Failure - buckling, crushing, or collapse cause by compression stress

Compression Flange – the part of a beam that is compressed due to a bending moment.

Concentrated Load – a force applied over a small contact area; also known as a point load

Connection Angle – a piece of angle serving to connect two elements of a member or two members of a structure; also known as a clip angle

Construction Joint – a pair of adjacent surfaces in reinforced concrete where two pours meet; reinforcement steel extends through this joint

Continuous Beam – a general term applied to a beam that spans uninterrupted over one or more intermediate supports

Continuous Bridge – a bridge designed to extend without joints over one or more interior supports

Continuous Span – spans designed to extend without joints over one or more intermediate supports

Continuously Braced Flange – a flange encased in concrete or anchored by shear connectors for which flange lateral bending effects need not be considered. A continuously braced flange in compression is also assumed not to be subject to local or lateral torsional buckling.

Cracked Section – a composite section in which the concrete is assumed to carry no tensile strength

Creep – time-dependent deformation of concrete under permanent load

Cross-Frame – a transverse truss framework connecting adjacent longitudinal flexural components or inside a tub section or closed box used to transfer and distribute vertical and lateral loads and provide stability to the compression flanges. Sometimes synonymous with the term diaphragm.

Cross-Section - the shape of an object cut transversely to its length

Cross-Section Distortion – change in shape of the cross-section profile due to torsional loading

Cross-Sectional Area - the area of a cross-section

Curved Girder – an I-, closed-box or tub girder that is curved in a horizontal plane

Deck – a component, with or without wearing surface, directly supporting wheel loads

Deformation – a change in structural geometry due to force effects, including axial displacement, shear displacement and rotations

Degree-of-Freedom – one of a number of translations or rotations required to define the movement of a node. The displaced shape of components and/or the entire structure may be defined by a number of degrees-of-freedom

Design Load - the force for which a structure is designed; the most severe combination of loads

Diagonal - a sloping structural member of a truss or bracing system

Diagonal Stay - a cable support in a suspension bridge extending diagonally from the tower to the roadway to add stiffness to the structure and diminish the deformations and undulations resulting from traffic service

Diagonal Tension - the tensile force due to horizontal and vertical shear in a beam

Diaphragm - a vertically oriented solid transverse member connecting adjacent longitudinal flexural components or inside a closed box or tub section to transfer and distribute vertical and lateral loads and to provide stability to the compression flanges

Disc Bearing – A bearing that accommodates rotation by deformation of a single elastomeric disc molded from a urethane compound. It may be movable, guided, unguided, or fixed. Movement is accommodated by sliding of polished stainless steel or PTFE

Differential Deflection - Non-uniform deflection of adjacent girders; i.e., when one girder deflects a greater amount than an adjacent girder

Differential Settlement - uneven settlement of individual or independent elements of a substructure; tilting in the longitudinal or transverse direction due to deformation or loss of foundation material

Discretely Braced Flange – a flange supported at discrete intervals by bracing sufficient to restrain lateral deflection of the flange and twisting of the entire cross-section at the brace points

Drift Bolt - a short length of metal bar used to connect and hold in position wooden members placed in contact; similar to a dowel

Drift Pin - tapered steel rod used by ironworkers to align bolt holes

Ductile - capable of being molded or shaped without breaking; plastic

Ductile Fracture - a fracture characterized by plastic deformation

Effective Depth – the depth of a component effective in resisting flexural or shear forces

Effective Length – the equivalent length KL used in compression formulas and determined by a bifurcation analysis.

Effective Prestress – the stress or force remaining in the prestressing steel after all losses have occurred

Effective Width – the reduced width of a plate or concrete slab which, with an assumed uniform stress distribution, produces the same effect on the behavior of a structural member as the actual plate width with its nonuniform stress distribution

Eigenvalue – a value used to represent the factor that is applied to the reference load to determine the critical buckling load

Elastic - capable of sustaining deformation without permanent loss of shape

Elastic Analysis – determination of load effects on members and connections based on the assumption that the material stress-strain response is linear and the material deformation disappears on removal of the force that produced it

Elastic Deformation - non-permanent deformation; when the stress is removed, the material returns to its original shape

Elastic Limit – the point at which the structural member will begin to permanently deform under load

Elasticity - the property whereby a material changes its shape under the action of loads but recovers its original shape when the loads are removed

Elongation - the elastic or plastic extension of a member

Embedment Length – the length of reinforcement or anchor provided beyond a critical section over which transfer of force between concrete and reinforcement may occur

End Post - the end compression member of a truss, either vertical or inclined in position and extending from top chord to bottom chord

End Span - a span adjacent to an abutment

Engineer of Record – registered licensed professional responsible for the design of the structure

Equilibrium – a state where the sum of forces and moments about any point in space is 0.0

Equivalent Uniform Load - a load having a constant intensity per unit of its length producing an effect equal to that of a live load consisting of vehicle axle or wheel concentrations spaced at varying distances

Failure - a condition at which a structure reaches a limit state such as cracking or deflection where it is no longer able to perform its usual function; collapse; fracture

Falsework - a temporary wooden or metal framework built to support the weight of a structure during the period of its construction and until it becomes self-supporting

Fascia - an outside, covering member designed on the basis of architectural effect rather than strength and rigidity, although its function may involve both

Fascia Girder - an exposed outermost girder of a span sometimes treated architecturally or otherwise to provide an attractive appearance

Filler - a piece used primarily to fill a space beneath a batten, splice plate, gusset, connection angle, stiffener or other element; also known as filler plate

Finite Element Method – a method of analysis in which a structure is discretized into elements connected at nodes, the shape of the element displacement field is assumed, partial or complete compatibility is maintained among the element interfaces, and nodal displacements are determined by using energy variational principles or equilibrium methods

First-Order Analysis – analysis in which equilibrium conditions are formulated on the undeformed structure; that is, the effect of deflections is not considered in writing equations of equilibrium

Fixed Bearing – a bearing that prevents differential longitudinal translation of abutting structural elements. It may or may not provide for differential lateral translation or rotation

Flange Lateral Bending – bending of a flange about an axis perpendicular to the flange plane due to lateral loads applied to the flange and/or non-uniform torsion in the member.

Flange Lateral Bending Stress – the normal stress caused by flange lateral bending

Flexural Buckling – a buckling mode in which a compression member deflects laterally without twist or change in cross-sectional shape

Flexural-Torsional Buckling – a buckling mode in which a compression member bends and twists simultaneously without a change in cross-sectional shape

Floorbeam – a primary horizontal member located transversely to the general bridge alignment

Floor System - the complete framework of members supporting the bridge deck and the traffic loading

Forms - the molds that hold concrete in place while it is hardening; also known as form work

Fracture Critical Member (FCM) - a steel member in tension, or with a tension element, whose failure would probably cause a portion of or the entire bridge to collapse

Frame - a structure which transmits bending moments from the horizontal beam member through rigid joints to vertical or inclined supporting members

Framing - the arrangement and connection of the component members of a bridge superstructure

Free End - movement is not restrained

Girder Radius – the radius of the circumferential centerline of a segment of a curved girder

Global Analysis – analysis of a structure as a whole

Global Stability – the stability of an entire structural system in terms of resistance to buckling (as opposed to the stability of an individual girder or other element within a structural system in terms of resistance to local buckling of that girder or element)

Grillage Analogy Method – a method of analysis in which all or part of the superstructure is discretized into orthotropic components that represent the characteristics of the structure

Gusset Plate – plate material used to interconnect vertical, diagonal and horizontal truss members at a panel point

Hanger - a tension member serving to suspend an attached member; allows for expansion between a cantilevered and suspended span

Haunch - an increase in the depth of a member usually at points of support; the outside areas of a pipe between the spring line and the bottom of the pipe

Haunched Girder - a horizontal beam whose cross-sectional depth varies along its length

H-Beam - a rolled steel member having an H-shaped cross-section (flange width equals beam depth) commonly used for piling; also H-pile

Hinge - a point in a structure at which a member is free to rotate

Hinged Joint – a joint constructed with a pin, cylinder segment, spherical segment or other device permitting rotational movement

Holding Crane – a crane used to provide a temporary vertical support to a girder during erection

Horizontal Alignment - a roadway's centerline or baseline alignment in the horizontal plane

Horizontal Curve - a roadway baseline or centerline alignment defined by a radius in the horizontal plane

I-Beam - a structural member with a cross-sectional shape similar to the capital letter "I"

Inelastic – any structural behavior in which the ratio of stress and strain is not constant, and part of the deformation remains after load removal

Inelastic Redistribution – the redistribution of internal force effects in a component or structure caused by inelastic deformations at one or more sections

Instability – a condition reached in the loading of a component or structure in which continued deformation results in a decrease of load-resisting capacity

Integral Bridge - a bridge without deck joints

Interior Girder - any girder between exterior or fascia girders

Jacking - the lifting of elements using a type of jack (e.g., hydraulic), sometimes acts as a temporary support system

Jacking Force – the force exerted by the device that introduces tension into the tendons

Knee Brace - a short member engaging at its ends two other members that are joined to form a right angle or a near-right angle to strengthen and stiffen the connecting joint

Large Deflection Theory – any method of analysis in which the effects of deformation upon force effects is taken into account

Lateral Bending Stress – the normal stress caused by flange lateral bending

Lateral Bracing – a truss placed in a horizontal plane between two I-girders or two flanges of a tub girder to maintain cross-sectional geometry and provide additional stiffness and stability to the bridge system

Lateral Torsional Buckling – buckling of a component subject to compression involving lateral deflection and twist

Leaf - the movable portion of a bascule bridge that forms the span of the structure

Lightweight Concrete – concrete containing lightweight aggregate and having an air-dry unit weight not exceeding 0.120 kcf, as determined by ASTM C567. Lightweight concrete without natural sand is termed “all-lightweight concrete” and lightweight concrete in which all of the fine aggregate consists of normal weight sand is termed “sand-lightweight concrete.”

Linear Response – structural behavior in which deflections are directly proportional to loads

Load Buckling – the buckling of a plate element in compression

Load Factor Design - a design method used by AASHTO, based on limit states of material and arbitrarily increased loads

Load And Resistance Factor Design (LRFD) - design method used by AASHTO, based on limit states of material with increased loads and reduced member capacity based on statistical probabilities

Local Buckling - localized buckling of a component subject to compression

Loss of Prestress - loss of prestressing force due to a variety of factors, including shrinkage and creep of the concrete, creep of the prestressing tendons, and loss of bond

Member - an individual angle, beam, plate, or built component piece intended ultimately to *become* an integral part of an assembled frame or structure

Metal Rocker or Roller Bearing - a bearing that carries vertical load by direct contact between two metal surfaces and that accommodates movement by rocking or rolling of one surface with respect to the other

Moment of Inertia - the sum of each area in a section times the distance between the centroid of that area and the axis under consideration squared

Movable Bridge - a bridge having one or more spans capable of being raised, turned, lifted, or slid from its normal service location to provide a clear navigation passage

Movable Span - a general term applied to a superstructure span designed to be swung, lifted or otherwise moved longitudinally, horizontally or vertically, usually to provide increased navigational clearance

Multirotational Bearing - a bearing consisting of a rotational element of the pot type, disc type, or spherical type when used as a fixed bearing and that may, in addition, have sliding surfaces to accommodate translation when used as an expansion bearing. Translation may be constrained to a specified direction by guide bars

Negative Moment – moment producing tension at the top of a flexural element

Noncompact Flange – for a composite section in negative flexure or a noncomposite section, a discretely braced compression flange with a slenderness at or below the limit at which localized yielding within the member cross-section associated with a hybrid web, residual stresses and/or cross-section monosymmetry has a statistically significant effect on the nominal flexural resistance

Noncompact Section – a composite section in positive flexure for which the nominal resistance is not permitted to exceed the moment at first yield

Noncompact Unbraced Length – for a composite section in negative flexure or a noncomposite section, the limiting unbraced length of a discretely braced compression flange at or below the limit at which the onset of yielding in either flange of the cross-section with consideration of compression-flange residual stress effects has a statistically significant effect on the nominal flexural resistance.

Nonlinear Response – structural behavior in which the deflections are not directly proportional to the loads due to stresses in the inelastic range, or deflections causing significant changes in force effects, or by a combination thereof

Nonuniform Torsion – an internal resisting torsion in thin-walled sections, also known as warping torsion, producing shear stress and normal stresses, and under which cross-sections do not remain plane. Members resist the externally applied torsion by warping torsion and St. Venant torsion. Each of these components of internal resisting torsions varies along the member length, although the externally applied concentrated torque may be uniform along the member between two adjacent points of torsional restraint. Warping torsion is dominant over St. Venant torsion in members having open cross-sections, whereas St. Venant torsion is dominant over warping torsion in members having closed cross-sections.

Normal Weight Concrete – concrete having a weight between 0.135 and 0.155 kcf.

Pier - a substructure unit that supports the spans of a multi-span superstructure at an intermediate location between its abutments

Pier Cap - the topmost horizontal portion of a pier that distributes loads from the superstructure to the vertical pier elements

Pile - a shaft-like linear member which carries loads to underlying rock or soil strata

Pile Bent - a row of driven or placed piles extending above the ground surface supporting a pile cap

Pile Cap - a slab or beam which acts to secure the piles in position laterally and provides a bridge seat to receive and distribute superstructure loads

Pin - a cylindrical bar used to connect elements of a structure

Pin-Connected Truss - a general term applied to a truss of any type having its chord and web members connected at each panel point by a single pin

Pin and Hanger - a hinged connection detail designed to allow for expansion and rotation between a cantilevered and suspended span at a point between supports.

Pintle - a relatively small steel pin engaging the rocker of an expansion bearing, in a sole plate or masonry plate, thereby preventing sliding of the rocker

Plate Girder - a large I-shaped beam composed of a solid web plate with flange plates attached to the web plate by flange angles or fillet welds

Point of Contraflexure – the point where the sense of the flexural moment changes; synonymous with point of inflection

Positive Moment – moment producing tension at the bottom of a flexural element

Post-Tensioning – a method of prestressing in which the tendons are tensioned after the concrete has reached a predetermined strength

Post-Tensioning Duct – a form device used to provide a path for post-tensioning tendons or bars in hardened concrete

Pot Bearing - a bearing type that allows for multi-dimensional rotation by using a piston supported on an elastomer contained on a cylinder, the pot, or spherical bearing element

Precast Concrete - concrete members that are cast and cured before being placed into their final positions on a construction site

Prestressed Concrete – concrete components in which stresses and deformations are introduced by application of prestressing force

Prestressing - applying forces to a structure to deform it in such a way that it will withstand its working loads more effectively

Pretensioning - a method of prestressing concrete in which the strands are stressed before the concrete is placed; strands are released after the concrete has hardened, inducing internal compression into the concrete

Primary Member - a member designed to resist flexure and distribute primary live loads and dead loads

Professional Engineer (PE) - an individual, who has fulfilled education and experience requirements and passed rigorous exams that, under State licensure laws, permits them to offer engineering services directly to the public. Engineering licensure laws vary from State to State, but, in general, to become a PE an individual must be a graduate of an engineering program accredited by the Accreditation Board for Engineering and Technology, pass the Fundamentals of Engineering exam, gain four years of experience working under a PE, and pass the Principles of Practice of Engineering exam

Redistribution of Moments – a process that results from formation of inelastic deformations in continuous structures

Refined Methods of Analysis – methods of structural analysis that consider the entire superstructure as an integral unit and provide the required deflections and actions.

Reinforced Concrete - concrete with steel reinforcing bars embedded in it to supply increased tensile strength and durability

Reinforcing Bar - a steel bar, plain or with a deformed surface, which bonds to the concrete and supplies tensile strength to the concrete

Residual Stresses – stresses locked into the cross-section typically caused during manufacturing or fabrication

Rocker Bearing - a bridge support that accommodates expansion and contraction of the superstructure through a tilting action

Rolled Shape - forms of rolled steel having I, H, C, Z or other cross-sectional shapes

Roller Bearing - a single roller or a group of rollers so installed as to permit longitudinal movement of a structure

Safety Factor - the difference between the ultimate strength of a member and the maximum load it is expected to carry

Sag - to sink or bend downward due to weight or pressure

Seat - a base on which an object or member is placed

Seat Angle - a piece of angle attached to the side of a member to provide support for a connecting member either temporarily during its erection or permanently; also known as a shelf angle

Second-Order Analysis – analysis in which equilibrium conditions are formulated on the deformed structure; that is, in which the deflected position of the structure is used in writing the equations of equilibrium

Secondary Member - a member that does not carry calculated loads; bracing members

Settlement - the movement of substructure elements due to changes in the soil properties

Shear - the load acting across a beam near its support

Shear Connectors - devices that extend from the top flange of a beam and are embedded in the above concrete slab, forcing the beam and the concrete to act as a single unit

Shear Stress - the shear force per unit of cross-sectional area

Shop Drawings - detailed drawings developed from the more general design drawings used in the manufacture or fabrication of bridge components

Skew Angle - the angle produced when the longitudinal members of a bridge are not perpendicular to the substructure; the skew angle is the acute angle between the alignment of the bridge and a line perpendicular to the centerline of the substructure units

Slab - a wide beam, usually of reinforced concrete, which supports load by flexure

Slab Bridge - a bridge having a superstructure composed of a reinforced concrete slab constructed either as a single unit or as a series of narrow slabs placed parallel with the roadway alignment and spanning the space between the supporting substructure units

Slenderness Ratio – the ratio of the effective length of a member to the radius of gyration of member cross-section, both with the respect to the same axis of bending, or the full or partial width or depth of a component divided by its thickness

Small Deflection Theory – a basis for methods of analysis where the effects of deformation upon force effects in the structure is neglected

Sole Plate - a plate attached to the bottom flange of a beam that distributes the reaction of the bearing to the beam

Span - the distance between the supports of a beam; the distance between the faces of the substructure elements; the complete superstructure of a single span bridge or a corresponding integral unit of a multiple span structure

Specifications - a detailed description of requirements, materials, tolerances, etc., for construction which are not shown on the drawings; also known as specs

Splice – a group of bolted connections, or a welded connection, sufficient to transfer the moment, shear, axial force, or torque between two structural elements joined at their ends to form a single, longer element

Spliced Precast Girder – a type of superstructure in which precast concrete beam-type elements are joined longitudinally, typically using post-tensioning, to form the completed girder. The bridge cross-section is typically a conventional structure consisting of multiple precast girders. This type of construction is not considered to be segmental construction.

St. Venant Torsion – that portion of the internal resisting torsion in a member producing only pure shear stresses on a cross-section; also referred to as pure torsion or uniform torsion

Stability Limit States – the limit states that often control the design of members subject to compression in steel structures. These limit states include local buckling, lateral torsional buckling and global buckling

Staged Construction - construction performed in phases, usually to permit the flow of traffic

Stay-In-Place Formwork – permanent metal or precast concrete forms that remain in place after construction is finished

Steel-Reinforced Elastomeric Bearing – A bearing made from alternate laminates of steel and elastomer bonded together during vulcanization. Vertical loads are carried by compression of the elastomer. Movements parallel to the reinforcing layers and rotations are accommodated by deformation of the elastomer

Stiffener – a member, usually an angle or plate, attached to a plate or web of a beam or girder to distribute load, to transfer shear, or to prevent buckling of the member to which it is attached

Stiffening Girder - a girder incorporated in a suspension bridge to distribute the traffic loads uniformly among the suspenders and reduce local deflections

Stiffening Truss - a truss used to increase the stability of a girder during lifting

Stirrup - U-shaped bar used as a connection device in timber and metal bridges; U-shaped bar placed in concrete to resist diagonal tension (shear) stresses

Strain - the change in length of a body produced by the application of external forces, measured in units of length; this is the proportional relation of the amount of change in length divided by the original length

Stress - the force acting across a unit area in a solid material

Stress Concentration - local increases in stress caused by a sudden change of cross-section in a member

State Transportation Department - that department, commission, board, or official of any State charged by its laws with the responsibility for highway construction

Stringer - a longitudinal beam spanning between transverse floorbeams and supporting a bridge deck

Strongback – a structural element, usually located above the supported member, that provides supplemental support to the structure

Structural Member - an individual piece, such as a beam or strut, which is an integral part of a structure

Structural Stability - the ability of a structure to maintain its normal configuration, not collapse or tip in any way, under existing and expected loads

Strut - a member acting to resist axial compressive stress; usually a secondary member

Substructure - the abutments and piers built to support the span of a bridge superstructure

Superelevation - the difference in elevation between the inside and outside edges of a roadway in a horizontal curve; required to counteract the effects of centrifugal force

Superimposed Dead Load - dead load that is applied to a compositely designed bridge after the concrete deck has cured; for example, the weight of parapets or railings placed after the concrete deck has cured

Superstructure - the entire portion of a bridge structure that primarily receives and supports traffic loads and in turn transfers these loads to the bridge substructure

Suspended Span - a simple span supported from the free ends of cantilevers

Suspension Bridge - a bridge in which the floor system is supported by catenary cables that are supported upon towers and are anchored at their extreme ends

Sway Bracing – transverse vertical bracing between truss members

Sweep – an out of plane imperfection from a member's perfect geometry

Tee Beam - a rolled steel section shaped like a T; reinforced concrete beam shaped like the letter T

Temperature Steel - reinforcement in a concrete member to prevent cracks due to stresses caused by temperature changes

Tendon - a prestressing cable, strand, or bar

Tensile Force - a force caused by pulling at the ends of a member

Tensile Strength - the maximum tensile stress at which a material fails

Tension - stress that tends to pull apart material

Thermal Movement - contraction and expansion of a structure due to a change in temperature

Tie - a member carrying tension

Tie Plate - relatively short, flat member carrying tension forces across a transverse member; for example, the plate connecting a floor beam cantilever to the main floor beam on the opposite side of a longitudinal girder

Tie Rod - a rod-like member in a frame functioning to transmit tensile stress; also known as tie bar

Tied Arch – an arch in which the horizontal thrust of the arch rib is resisted by a horizontal tie

Torque - the angular force causing rotation

Torsion - twisting about the longitudinal axis of a member

Torsional Buckling – a buckling mode in which a compression member twists about its shear center

Torsional Rigidity - a beam's capacity to resist a twisting force along the longitudinal axis

Torsional Shear Stress – shear stress induced by St. Venant torsion

Transverse Bracing - the bracing assemblage engaging the columns of bents and towers in planes transverse to the bridge alignment that resists the transverse forces tending to produce lateral movement and deformation of the columns

True Arch – an arch in which the horizontal component of the force in the arch rib is resisted by an external force supplied by its foundation

Truss – a system of members comprising a series of triangles that distribute the load by creating tension and compression in the members rather than bending

Tub Section – an open-topped section which is composed of a bottom flange, two inclined or vertical webs and top flanges

Tubular Sections - structural steel tubes, rectangular, square or circular; also known as hollow sections

Turnbuckle - a long, cylindrical, internally threaded nut with opposite hand threads at either end used to connect the elements of adjustable rod and bar members

U-Bolt - a bar bent in the shape of the letter "U" and fitted with threads and nuts at its ends

Ultimate Strength - the highest stress that a material can withstand before breaking

Unbraced Length – distance between brace points resisting the mode of buckling or distortion under consideration; generally, the distance between panel points or brace locations

Uncracked Section – a section in which the concrete is assumed to be fully effective in tension and compression

Uniform Load - a load of constant magnitude along the length of a member

Uplift - a negative reaction or a force tending to lift a beam, truss, pile, or any other bridge element upwards off of a bearing or support

Upper Chord - the top longitudinal member of a truss

Vertical - describes the axis of a bridge perpendicular to the underpass surface

Vertical Alignment - a roadway's centerline or baseline alignment in the vertical plane

Warping – distortion of the cross-section from torsion

Warping Stress – normal stress induced in the cross-section by warping torsion and/or by distortion of the cross-section

Warping Torsion – that portion of the total resistance to torsion in a member producing shear and normal stresses that is provided by resistance to out-of-plane warping of the cross-section

Web Crippling – the local failure of a web plate in the immediate vicinity of a concentrated load or bearing reaction due to the transverse compression introduced by this load

Web Members - the intermediate members of a truss, not including the end posts, usually vertical or inclined

Web Plate - the plate forming the web element of a plate girder, built-up beam or column

Wide Flange - a rolled I-shaped member having flange plates of rectangular cross-section, differentiated from an S-beam (American Standard) in that the flanges are not tapered

Working Stress - the unit stress in a member under service or design load

Working Stress Design - a method of design using the yield stress of a material and a factor of safety that determine the maximum allowable stresses

Yield Strength – the stress at which a material exhibits a specified limiting deviation from the proportionality of stress to strain

Yield Stress - the stress at which noticeable, suddenly increased deformation occurs under slowly increasing load