Spreadsheet Title:	I-Girder Section Analysis, AASHTO LRFD 3rd Ed. (2004), Art. 6.10; ver. 0.122 ((c) DesignSpreadsheets.com 2005-2008)
Last Revision Date:	8/27/2005

All blue fields with double frames are used for input.

by DesignSpreadsheets.com
www.DesignSpreadsheets.com
email: info@designspreadsheets.com

Project	I-Girder Section Analysis and Design Spreadsheet Demo for DesignSpreadsheets.com Website
Job No.	xy
Subject	Spreadsheet Demo
Sheet No.	xy
Made By	admin
Date Made	4/12/2006
Checked By	admin
Date Checked	4/12/2006

Comments

Spreadsheet Instructions

Checks for marking relevant/nonrelevant calculations in the left column:

1. Positive / Negative Moment Region
2. Composite / Noncomposite Section
3. Compact / Noncompact Section (Local)

Note:

Factor 1.5 used for construction loads (Strength IV load combination - high dead load to live load ratio).

Color Coding:

	[LIGHT BROWN shading]	headings
	[CYAN shading]	user's input
	[STRIPED CYAN shading]	user's input that typically depends on other input (typically, these cells do not need to be changed)
	[YELLOW shading]	important conclusions, values, etc.
||		'||' with green shading in very left column - relevant calculations for current input
x		red 'x' in very left column - nonrelevant calculations for current input
.		'.' in very left column - calculations not considered in the current version of the spreadsheet but referenced for completeness

[RED bold font]	failed code check

Disclaimer

This spreadsheet was tested for reliability and accuracy by DesignSpreadsheets.com. However, it is entirely responsibility of the spreadsheet user to verify the applicability and accuracy of any results obtained from this spreadsheet. DesignSpreadsheets.com is not liable for any damages, direct or indirect, resulting from using this spreadsheet.

Spreadsheet Revision History

4/11/2006 [ver. 0.121]	First BridgeArt.net version.
4/14/2006 [ver. 0.122]	Minor cosmetic revisions.

Registration

	Unregistered Demo Version
Registration Key	???

Active Record Name

Girder 4

Database of All Records

1	Girder 1
2	Girder 2 - Neg. Moment Region
3	Girder 3 - Pos. Moment Region
4	Girder 4
5	Girder 5
6	Girder 6
7	Girder 7
8	Girder 8
9	Girder 9
10	
11	
12	
13	
14	
15	
16	
17	
18	
19	
20	
21	
22	
23	
24	
25	
26	
27	
28	
29	
30	
31	
32	
33	
34	

Select database record you want to upload and hit "Upload From Database" button.

Save To Database

Save current input data to the database. The input data will be associated with active record name.

Upload From Database

Upload input data for active (selected) record.

Upload input data for the previous or next record in the database.

Add new names for records into the blue shaded fields.

Hide NonRelevant Rows

Hide non-relevant rows for current input (use e.g prior to printing).

Show All Rows

Show all rows

Project: ***I-Girder Section Analysis and Design Spreadsheet Demo for DesignSpreadsheets.com Website***	Made By: *admin* Date: *4/12/2006*	Job No: *xy*
Subject: *Spreadsheet Demo*	Checked By: *admin* Date: *4/12/2006*	Sheet No: *xy*

Girder 4

INPUT

Girder

b_{bf}	18	*in*
b_{tf}	18	*in*
t_{bf}	1.875	*in*
t_{tf}	1.75	*in*
h_w	51	*in*
t_w	0.625	*in*

Deck

b_d	75.468	*in*
t_d	8	*in*
A_{s1}	1.98	*in^2*
A_{s2}	1.36	*in^2*
d_1	2	*in*
d_2	2	*in*

Hauch

b_h	0	*in*
t_h	0	*in*

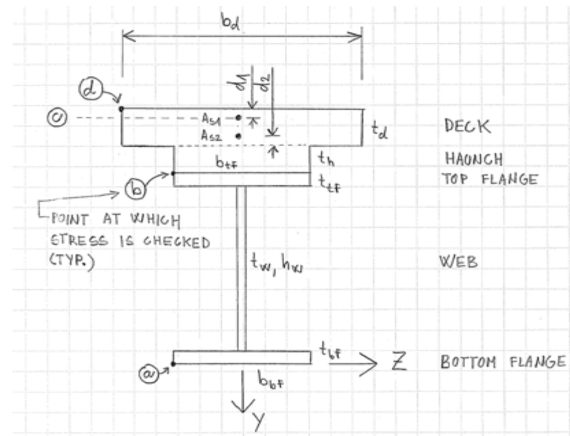
Modular ratio

n	7.9	*[-]*

OUTPUT - Sectional Properties

(section transformed into steel)

	Girder Only	Composite (3n)	Composite (n)	Composite (rebar)
Girder A [in^2]	97.13	97.13	97.13	97.13
Girder y_{cg} [in]	26.742	26.742	26.742	26.742
Girder I_z [in^4]	52389.9	52389.9	52389.9	52389.9
Haunch A [in^2]	-	0.00	0.00	-
Haunch y_{cg} [in]	-	54.625	54.625	-
Haunch I_z [in^4]	-	0.0	0.0	-
Deck A [in^2]	-	25.47	76.42	-
Deck y_{cg} [in]	-	58.625	58.625	-
Deck I_z [in^4]	-	135.9	407.6	-
Rebar A	-	-	-	3.34
Rebar y_{cg}	-	-	-	58.996
Rebar I_z	-	-	-	12.9
Total A	97.13	122.60	173.55	100.47
Total y_{cg}	26.742	33.367	40.782	27.815
Total Iz	52389.9	73040.1	96273.3	55762.0
y_topdeck(d) [in]	-	29.258	21.843	-
y_topbar(c) [in]	-	27.258	19.843	32.810
y_topgrd(b) [in]	27.883	21.258	13.843	26.810
y_botgrd(a) [in]	26.742	33.367	40.782	27.815
S_topdeck(d) [in^3]	-	2496.4	4407.5	-
S_topbar(c) [in^3]	-	2679.6	4851.8	1699.5
S_topgrd(b) [in^3]	1878.9	3435.9	6954.7	2079.9
S_botgrd(a) [in^3]	1959.1	2189.0	2360.7	2004.8



Section Properties about Weak Axis

I_y =	1762.8	*in^4*
$S_{y[TOP\ FLANGE]}$ =	195.9	*in^3*
$S_{y[BOT\ FLANGE]}$ =	195.9	*in^3*

Neutral Axis Check

OK - neutral axis is within girder

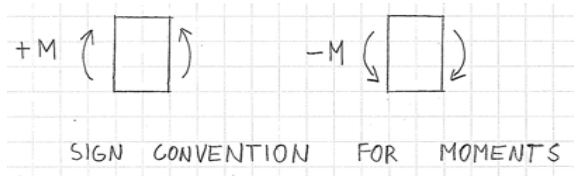
Comment

I-Girder Section Analysis, AASHTO LRFD 3rd Ed. (2004), Art. 6.10; ver. 0.122 ((c) DesignSpreadsheets.com 2005-2008)

Project:	***I-Girder Section Analysis and Design Spreadsheet Demo for DesignSpreadsheets.com Website***	Made By: *admin* Date: *4/12/2006*	Job No: *xy*
Subject:	*Spreadsheet Demo*	Checked By: *admin* Date: *4/12/2006*	Sheet No: *xy*

Note: Stress sign convention for this sheet - compressive stresses are reported as negative, tensile stresses as positive (sign convention for moments - see sketch below).

Girder 4



INPUT - Moments

DC1	-2388	*kip-ft*
DC2	-230	*kip-ft*
DW	-308	*kip-ft*
LL+I	-1552	*kip-ft*
LL+I fat range	-370	*kip-ft*
LL+I permit	-1665	*kip-ft*
SE	0	*kip-ft*

OUTPUT - Stresses

Positive Moment Region (stress at the top of deck is reported as stress in concrete: $f_c=f_s/n$ or $f_c=f_s/(3n)$)

		Load Acting on	Grd Only	Composite (3n)			Composite (n)							
x		Load Type	DC1	DC2	DW	SE	LL+I	LL+I fat	LL+I p	Service II	Strength I	Strength II	Fatigue	Governing
x	STRESS [ksi]	@ topdeck(d)	0	0.0	0.1	0.0	0.5	0.1	0.6	0.8	1.1	0.9	0.1	0.1
x		@ topbar(c)	0	1.0	1.4	0.0	3.8	0.9	4.1	7.4	10.1	8.9	0.7	0.7
x		@ topgrd(b)	15.3	0.8	1.1	0.0	2.7	0.6	2.9	20.6	26.4	25.6	0.5	0.5
x		@ botgrd(a)	-14.6	-1.3	-1.7	0.0	-7.9	-1.9	-8.5	-27.8	-36.2	-33.8	-1.4	-1.4

Negative Moment Region (stress at the top of deck is reported as stress in concrete using short-term composite section: $f_c=f_s/n$)

		Load Acting on	Grd Only	Composite (rebar)										
||		Load Type	DC1	DC2	DW	SE	LL+I	LL+I fat	LL+I p	Service II	Strength I	Strength II	Fatigue	Governing
||	STRESS [ksi]	@ topdeck(d)	0	0.1	0.1	0.0	0.5	0.1	0.6	0.9	1.2	1.0	0.1	1.2
||		@ topbar(c)	0	1.6	2.2	0.0	11.0	2.6	11.8	18.0	24.5	21.2	2.0	24.5
||		@ topgrd(b)	15.3	1.3	1.8	0.0	9.0	2.1	9.6	30.0	39.1	36.4	1.6	39.1
||		@ botgrd(a)	-14.6	-1.4	-1.8	0.0	-9.3	-2.2	-10.0	-29.9	-39.0	-36.2	-1.7	-39.0
									Moment [kip-ft] →	-4943.6	-6450.5	-5982.3	-277.5	

Project: ***I-Girder Section Analysis and Design Spreadsheet Demo for DesignSpreadsheets.com Website***	Made By: *admin* Date: *4/12/2006*	Job No: *xy*
Subject: *Spreadsheet Demo*	Checked By: *admin* Date: *4/12/2006*	Sheet No: *xy*

Note: sign convention for this sheet - all moments and stresses are entered and reported as absolute values.

Variable/Formula	*Value*	*Units*	*Comment*	*AASHTO Page*

|| *indicates that corresponding checks are applicable for section under consideration (based on user's input)*

BASIC INPUTS

Girder 4

||				● general	
||		NEG		positive moment region (POS), negative moment region (NEG)	
||		COMPOSITE		composite section (COMPOSITE), noncomposite section (NONCOMPOSITE)	
||					
||				● geometry	
||	t_w =	0.625	*in*	web thickness	
||	b_{fc} =	18.000	*in*	full width of compression flange	
||	t_{fc} =	1.875	*in*	thickness of compression flange	
||	D =	51.000	*in*	web depth	
||					
||				● material properties	
||	E =	29,000	*ksi*	steel Young's modulus	
||	F_{yf} =	50.0	*ksi*	specified minimum yield strenth of flange	
||	F_{yw} =	50.0	*ksi*	specified mininun yield stress of web	
||	$F_{y,reinf}$ =	60.0	*ksi*	specified minimum yield strength of reinforcement	
||	F_{yc} =	50.0	*ksi*	specified minimum yield strength of compression flange	
||	F_{yt} =	50.0	*ksi*	specified minimum yield strength of tension flange	
||	F_{yr} =	35.0	ksi	compression flange stress at the onset of nominal yielding	6-108
||	note: F_{yr} = min (0.7F_{yc}, F_{yw}), Fyr ≥ 0.5F_{yc}				
||					
||				● effect of applied loads	
||					
||				● load factors [AASHTO 6.5.4.2]	6-27
||	Φ_f =	1.00	*[-]*	resistance factor for flexure	
||	Φ_v =	1.00	*[-]*	resistance factor for shear	
||					
||				● slenderness ratios for local buckling resistance [AASHTO 6.10.8.2.2]	6-107
||	**$\lambda_f = b_{fc} / (2t_{fc})$ =**	**4.800**	***[-]***	slenderness ratio for compression flange	
||	λ_{pf} = 0.38 sqrt (E/F_{yc}) =	9.152	*[-]*	limiting slenderness ratio for a compact flange	
||	λ_{rf} = 0.56 sqrt (E/F_{yc}) =	13.968	*[-]*	limiting slenderness ratio for a noncompact flange	
||					
||				● reduction factors	
||	**R_h =**	**1.000**	***[-]***	○ hybrid factor to account for reduced contribution of web to nominal flexural resistance at first yield in flange element; use 1.0 for girders with same steel strength for flange and web	6-80
.	NOT CONSIDERED:				
.	$R_h = [12+\beta(3\rho-\rho^3)] / [12+2\beta]$ =				
.	$\beta = 2D_nt_w / A_{fn}$ =				
.	ρ = min (F_{yw}/f_n, 1.0) =				
||	**R_b =**	**1.000**	***[-]***	○ web load-shedding factor; accounts for increase in compression flange stress due to web local buckling	6-81
||	$R_b = 1 - [a_{wc}/(1200+300a_{wc})] [2D_c/t_w-\lambda_{rw}]$ =	1.000	*[-]*		
||	λ_{rw} = 5.7 sqrt(E/F_{yc}) =	137.3	*[-]*	limiting slenderness ratio for noncompact web	
||	$a_{wc} = 2D_ct_w/b_{fc}t_{fc}$ =	0.921	*[-]*		

Project: *I-Girder Section Analysis and Design Spreadsheet Demo for DesignSpreadsheets.com Website*	Made By: *admin*	Date: *4/12/2006*	Job No: *xy*
Subject: *Spreadsheet Demo*	Checked By: *admin*	Date: *4/12/2006*	Sheet No: *xy*

Note: sign convention for this sheet - all moments and stresses are entered and reported as absolute values.

Variable/Formula	Value	Units	Comment	AASHTO Page

|| *indicates that corresponding checks are applicable for section under consideration (based on user's input)*

||					
||				● Nominal Shear Resistance of Unstiffened Webs [AASHTO 6.10.9.2]	6-115
||	**$V_n = V_{cr} = CV_p$ =**	**632**	***kip***	V_n = nominal shear resistance, V_{cr} = shear-buckling resistance	
||	$V_p = 0.58F_{yw}Dt_w$ =	924	*kip*	plastic shear force	
||	C =	0.684	*[-]*	ratio of the shear buckling resistance to the shear yield strength	
||	if $D/t_w \le 1.12$ sqrt (Ek/F_{yw}) then C =	1.000	*[-]*		
||	if 1.12 sqrt $(Ek/F_{yw}) < D/t_w \le 1.40$ sqrt (Ek/F_{yw}) then C =				
||	= 1.12 / (D/t_w) sqrt (Ek/F_{yw}) =	0.739	*[-]*		
||	if $D/t_w > 1.40$ sqrt (Ek/F_{yw}) then C =				
||	= 1.57 / $(D/t_w)^2$ (Ek/F_{yw}) =	0.684	*[-]*		
||	note: D/t_w =	81.6	*[-]*		
||	note: 1.12 sqrt (Ek/F_{yw}) =	60.3	*[-]*		
||	note: 1.40 sqrt (Ek/F_{yw}) =	75.4	*[-]*		
||	k =	5.0	*[-]*	shear buckling coefficient (k=5 for unstiffened webs)	
.				○ Nominal Resistance of Stiffened Webs - Interior Panels [AASHTO 6.10.9.3.2]	6-116
.	NOT CONSIDERED			not considered, because current version of the spreadsheet handles only unstiffened webs	
.					
.				○ Nominal Resistance of Stiffened Webs - End Panels [AASHTO 6.10.9.3.3]	6-117
.	NOT CONSIDERED			not considered, because current version of the spreadsheet handles only unstiffened webs	

CONSTRUCTIBILITY

6-86

				● Basic Inputs	
||	$f_{bu[C]}$ =	21.9	*ksi*	stress in compression flange due to DC1 (no lateral bending), factored by 1.5 (Strength IV load combination)	
||	$f_{bu[T]}$ =	22.9	*ksi*	stress in tension flange due to DC2 (no lateral bending), factored by 1.5 (Strength IV load combination)	
||	$f_{l[C]}$ =	10.0	*ksi*	stress in compression flange due to lateral bending (wind, from exterior girder bracket during construction, etc.)	
||	$f_{l[T]}$ =	10.0	*ksi*	stress in tension flange due to lateral bending	
||	V_u =	96	*kips*	factored shear in web, factored by 1.5 (Strength IV load combination)	
||					
||	D_c =	24.867	*in*	depth of web in compression in the elastic range	6-69
||	$r_t = b_{fc}$ / sqrt{ 12 $[1+(D_ct_w)/(3b_{fc}t_{fc})]$ } =	4.838	*in*	effective radius of gyration for lateral torsional buckling	6-109
||	C_b =	1.000	*[-]*	moment gradient modifier (conservatively, use C_b = 1.0)	6-108
||					
||				● unbraced lengths for lateral torsional buckling resistance [AASHTO 6.10.8.2.3]	6-108
||	**L_b =**	**264.0**	***in***	unbraced length	
||	$L_p = 1.0\ r_t$ sqrt(E/F_{yc}) =	116.5	*in*	limiting unbraced length 1 (for compact)	
||	$L_r = \pi\ r_t$ sqrt (E/F_{yr}) =	437.5	*in*	limiting unbraced length 2 (for noncompact)	
||					
||				● Compression Flange Flexural Resistance [AASHTO 6.10.8.2]	6-106

Note: sign convention for this sheet - all moments and stresses are entered and reported as absolute values.

|| indicates that corresponding checks are applicable for section under consideration (based on user's input)

	Variable/Formula	*Value*	*Units*	*Comment*	*AASHTO Page*
||	**$F_{nc} = \min (F_{nc[1]}, F_{nc[2]}) =$**	**43.1**	***ksi***	○ nominal flexural resistance of flange taken as smaller local buckling resistance and lateral torsional buckling resistance [AASHTO 6.10.8.2.1]	
||	$F_{nc[1]} =$	50.0	*ksi*	○ local buckling resistance of the compression flange [AASHTO 6.10.8.2.2]	
||	if $\lambda_f \le \lambda_{pf}$ then $F_{nc[1]} = R_bR_hF_{yc} =$	50.0	*ksi*		
||	if $\lambda_f > \lambda_{pf}$ then $F_{nc[1]} =$				
||	$= \{1-[1-F_{yr}/(R_hF_{yc})][(\lambda_f-\lambda_{pf})/(\lambda_{rf}-\lambda_{pf})]\}\ R_bR_hF_{yc} =$	63.6	*ksi*		
||	$F_{nc[2]} =$	43.1	*ksi*	○ lateral torsional buckling resistance of compression flange [AASHTO 6.10.8.2.3]	
||	if $L_b \le L_p$ then $F_{nc[2]} = R_bR_hF_{yc} =$	50.0	*ksi*		
||	if $L_p < L_b \le L_r$ then $F_{nc[2]} =$				
||	$= C_b\ \{1-[1-F_{yr}/(R_hF_{yc})][(L_b-L_p)/(L_r-L_p)]\}\ R_bR_hF_{yc} =$	43.1	ksi	note: $F_{nc[2]} \le R_bR_hF_{yc}$	
||	if $L_b > L_r$ then $F_{nc[2]} = F_{cr} =$	96.1	*ksi*	note: $F_{nc[2]} \le R_bR_hF_{yc}$	
||	$F_{cr} = C_bR_b\pi^2E / (L_b/r_t)^2 =$	96.1	*ksi*		
||					
||	**$F_{crw} = \min (F_{crw[1]}, F_{crw[2]}, F_{crw[3]}) =$**	**50.0**	***ksi***	○ nominal bend buckling resistance for webs without longitudinal stiffeners [AASHTO 6.10.1.9.1]	6-67
||	$F_{crw[1]} = 0.9Ek / (D/t_w)^2 =$	148.4	*ksi*		
||	$k = 9 / (D_c/D)^2 =$	37.9	*[-]*	bend buckling coefficient	
||	$F_{crw[2]} = R_hF_{yc} =$	50.0	*ksi*		
||	$F_{crw[3]} = F_{yw}/0.7 =$	71.4	*ksi*		
||					
||				● Flexure - Discretely Braced Flanges in Compression [AASHTO 6.10.3.2.1]	6-87
||	**$f_{bu} + f_l \le \Phi_fR_hF_{yc}$**	**OK**			
||	$f_{bu} + f_l =$	31.9	*ksi*		
||	$\Phi_fR_hF_{yc} =$	50.0	*ksi*		
||					
||	**$f_{bu} + (1/3)f_l \le \Phi_fF_{nc}$**	**OK**			
||	$f_{bu} + (1/3)f_l =$	25.3	*ksi*		
||	$\Phi_fF_{nc} =$	43.1	*ksi*		
||					
||	**$f_{bu} \le \Phi_fF_{crw}$**	**OK**			
||	$f_{bu} =$	21.9	*ksi*		
||	$\Phi_fF_{crw} =$	50.0	*ksi*		
||					
||				● Flexure - Discretely Braced Flanges in Tension [AASHTO 6.10.3.2.2]	6-89
||	**$f_{bu} + f_l \le \Phi_fR_hF_{yt}$**	**OK**			
||	$f_{bu} + f_l =$	32.9	*ksi*		
||	$\Phi_fR_hF_{yt} =$	50.0	*ksi*		
.				● Flexure - Continously Braced Flanges in C or T; assumption that continously braced flange is not subject to local or lateral torsional buckling	
.	**$f_{bu} \le \Phi_fR_hF_{yf}$**				
.	NOT CONSIDERED			not considered, because no flanges are continously braced during constructruction	
||				● Shear	6-90
||	**$V_u \le \Phi_vV_{cr}$**	**OK**			
||	$V_u =$	96	*kips*	factored shear in the web	
||	$V_{cr} =$	632	*kips*	shear buckling resistance	
||				● Stress in Concrete Deck [AASHTO 6.10.3.2.4]	6-89

Project: *I-Girder Section Analysis and Design Spreadsheet Demo for DesignSpreadsheets.com Website*	Made By: *admin* Date: *4/12/2006*	Job No: *xy*
Subject: *Spreadsheet Demo*	Checked By: *admin* Date: *4/12/2006*	Sheet No: *xy*

Note: sign convention for this sheet - all moments and stresses are entered and reported as absolute values.

Variable/Formula	*Value*	*Units*	*Comment*	*AASHTO Page*
|| *indicates that corresponding checks are applicable for section under consideration (based on user's input)*				
|| Shall 1% longitudinal reinforcement be provided?	YES		If the actual tensile stress in concrete deck exceeds Φf_r, to control cracking, the area of longitudinal reinforcement shall be at least 1% of the concrete deck area. [AASHTO 6.10.1.7]	6-75
|| σ_c = (1/n) (M/S) =	1.2	*ksi*	actual tensile stress in concrete deck	
|| M =	42,984	*kip-in*	moment due to construction loads, DC1 , factored by 1.5 (Strength IV load combination - high DL to LL ratio) [imported from Stresses tab] [AASHTO 6.10.1.7]	6-75
|| note: M =	3,582	*kip-ft*		
|| S =	4407.5	*in^3*	section modulus for top of deck using n = 1 (section transformed in steel) [imported from Stresses tab]	
|| Φf_r =	0.0	*ksi*	factored concrete tensile resistance	
|| Φ =		*[-]*	resistance factor for concrete in tension	5-53
|| f_r = 0.24 sqrt (f_c') =	0.00	*ksi*	modulus of rupture for concrete deck [AASHTO 5.4.2.6]	5-16
|| f_c' =		*ksi*	specified compressive strength of concrete	

SERVICE LIMIT STATE

[AASHTO 6.10.4] 6-93

||			● Basic Inputs	
|| **$f_{f[TOP]}$ =**	30.0	*ksi*	stress in top flange for Service II load combination	
|| **$f_{f[BOT]}$ =**	29.9	*ksi*	stress in bottom flange for Service II load combination	
|| **f_l =**	10.0	*ksi*		
||				
||			● Flexural Checks	6-93
|| **$f_f \le 0.95R_hF_{yf}$**	**OK**		○ top steel flange of composite sections	
|| f_f =	30.0	*ksi*		
|| $0.95R_hF_{yf}$ =	47.5	*ksi*		
||				
|| **$f_f + f_l/2 \le 0.95R_hF_{yf}$**	**OK**		○ bottom steel flange of composite sections	
|| $f_f + f_l/2$ =	34.9	*ksi*		
|| $0.95R_hF_{yf}$ =	47.5	*ksi*		
. **$f_f + f_l/2 \le 0.80R_hF_{yf}$**			○ both steel flanges of noncomposite sections	
. NOT CONSIDERED			noncomposite sections are not considered in this version of the spreadsheet	
.				
|| **$f_c \le F_{crw}$**	**OK**		○ this check applies to all sections except for composite sections in positive flexure with web proportions such that $D/t_w \le 150$ [AASHTO Eq. 6.10.4.2.2-4]	6-94
|| $f_c = f_{f[TOP]}$ =	30.0	*ksi*	compression flange stress due to Service II loads	
|| F_{crw} =	50.0	*ksi*	nominal bend buckling resistance for webs [see "Strength Limit State" section below for the calculation of F_{crw}] [AASHTO 6.10.1.9.1]	6-77
			● Stress in Concrete Deck [AASHTO 6.10.3.2.4]	6-89
|| Shall 1% longitudinal reinforcement be provided?	YES		If the actual tensile stress in concrete deck exceeds Φf_r, to control cracking, the area of longitudinal reinforcement shall be at least 1% of the concrete deck area. [AASHTO 6.10.1.7]	6-75
|| σ_c = (1/n) (M/S) =	1.70	*ksi*	actual tensile stress in concrete deck	
|| M =	59,323	*kip-in*	moment due to Service II load combination [imported from Stresses tab] [AASHTO 6.10.1.7]	6-75

Project: *I-Girder Section Analysis and Design Spreadsheet Demo for DesignSpreadsheets.com Website*	Made By: *admin*	Date: *4/12/2006*	Job No: *xy*
Subject: *Spreadsheet Demo*	Checked By: *admin*	Date: *4/12/2006*	Sheet No: *xy*

Note: sign convention for this sheet - all moments and stresses are entered and reported as absolute values.

Variable/Formula	*Value*	*Units*	*Comment*	*AASHTO Page*

|| *indicates that corresponding checks are applicable for section under consideration (based on user's input)*

|| S =	4407.5	*in^3*	section modulus for top of deck using n = 1; section transformed in steel (same as for constructibility check above)	
|| Φf_r =	0.0	*ksi*	factored concrete tensile resistance (same as for constructibility check above)	

FATIGUE AND FRACTURE LIMIT STATES

[AASHTO 6.10.5] 6-95

|| fatigue detail description =	tension flange			
|| fatigue detail category =	C			
|| **$\gamma(\Delta f) \le (\Delta F)_n$**	**OK**			6-29
|| $\gamma(\Delta f)$ =	1.6	*ksi*	live load stress range due to passage of fatigue load multiplied by load factor γ = 0.75	
|| $(\Delta F)_{n\,=}$ (A/N)^(1/3) =	5.0	*ksi*	nominal fatigue resistance [AASHTO 6.6.1.2.5]	6-40
|| A =	44.00	*10^8 ksi^3*	constant from [AASHTO Tab. 6.6.1.2.5-1]	6-42
|| N = (365)(75)n$(ADTT)_{SL}$ =	232,687,500			
|| n =	1		number of stress range cycles per truck passage taken from [AASHTO Tab. 6.6.1.2.5-2]	
|| $(ADTT)_{SL}$ = (p)(ADTT) =	8,500		single lane ADTT (number of trucks per day in one direction averaged over the design life) [AASHTO 3.6.1.4]	3-24
|| p =	0.85		reduction factor for number of trucks for multiple lanes taken from [AASHTO Tab. 3.6.1.4.2-1]	
|| ADTT = (ftt) (ADT) =	10,000		number of trucks per day in one direction averaged over the design life	
|| ftt =	0.25		fraction of trucks in traffic	
|| ADT = (nl) $(ADT)_{SL}$ =	40,000		average daily traffic per whole bridge	
|| $(ADT)_{SL}$ =	20,000		average daily traffic per single lane (20,000 is considered maximum)	
|| nl =	2		number of lanes	
|| note: $(\Delta F)n \ge (1/2)(\Delta F)_{TH}$ =	5.0	*ksi*		
|| $(\Delta F)_{TH}$ =	10.00	*10^8 ksi^3*	constant amplitude fatigue threshold taken from [AASHTO Tab. 6.6.1.2.5-3]	
.			● special requirements for webs [AASHTO 6.10.5.3]	6-95
. **$V_u \le V_{cr}$**				
. NOT CONDERED			not considered, because this requirement does not need to be checked for unstiffened webs	

STRENGTH LIMIT STATE

6-96

||			● Basic Inputs	
|| $f_{bu[C]}$ =	39.0	*ksi*	stress in compression flange (no lateral bending)	
|| $f_{bu[T]}$ =	39.1	*ksi*	stress in tension flange (no lateral bending)	
|| $f_{l[C]}$ =	10.0	*ksi*	stress in compression flange due to lateral bending	
|| $f_{l[T]}$ =	10.0	*ksi*	stress in tension flange due to lateral bending	
|| M_u =	6,451	*kip-ft*	bending moment about major axis	
|| note: M_u =	77,406	*kip-in*		
|| V_u =	389	*kips*	factored shear in web	
||				
|| D_c =	25.427	*in*	depth of web in compression in the elastic range	6-69
|| D_{cp} =	26.906	*in*	depth of web in compression at the plastic moment	6-68

Project: *I-Girder Section Analysis and Design Spreadsheet Demo for DesignSpreadsheets.com Website*	Made By: *admin* Date: *4/12/2006*	Job No: *xy*
Subject: *Spreadsheet Demo*	Checked By: *admin* Date: *4/12/2006*	Sheet No: *xy*

Note: sign convention for this sheet - all moments and stresses are entered and reported as absolute values.

	Variable/Formula	Value	Units	Comment	AASHTO Page
	|| indicates that corresponding checks are applicable for section under consideration (based on user's input)				
||	$r_t = b_{fc}$ / sqrt{ 12 [1+(D_ct_w)/($3b_{fc}t_{fc}$)] } =	4.831	*in*	effective radius of gyration for lateral torsional buckling	6-109
||	C_b =	1.000	*[-]*	moment gradient modifier (conservatively, use C_b = 1.0)	6-108
||					
||				● unbraced lengths for lateral torsional buckling resistance [AASHTO 6.10.8.2.3]	6-108
||	**L_b =**	**264.0**	***in***	unbraced length	
||	L_p = 1.0 r_t sqrt (E/F_{yc}) =	116.3	*in*	limiting unbraced length 1 (for compact)	
||	L_r = π r_t sqrt (E/F_{yr}) =	436.9	*in*	limiting unbraced length 2 (for noncompact)	
||					
||				● Compression Flange Flexural Resistance [AASHTO 6.10.8.2]	6-106
||	**F_{nc} = min ($F_{nc[1]}$, $F_{nc[2]}$) =**	43.1	***ksi***	○ nominal flexural resistance of flange taken as smaller local buckling resistance and lateral torsional buckling resistance [AASHTO 6.10.8.2.1]	
||	$F_{nc[1]}$ =	50.0	*ksi*	○ local buckling resistance of the compression flange [AASHTO 6.10.8.2.2], same as for constructibility - see calculations above	
||	$F_{nc[2]}$ =	43.1	*ksi*	○ lateral torsional buckling resistance of compression flange [AASHTO 6.10.8.2.3]	
||	if $L_b \le L_p$ then $F_{nc[2]} = R_bR_hF_{yc}$ =	50.0	*ksi*		
||	if $L_p < L_b \le L_r$ then $F_{nc[2]}$ =				
||	= C_b {1-[1-F_{yr}/(R_hF_{yc})] [(L_b-L_p)/(L_r-L_p)]} $R_bR_hF_{yc}$ =	43.1	ksi		
||	if $L_b > L_r$ then $F_{nc[2]} = F_{cr} = C_bR_b\pi^2E / (L_b/r_t)^2$ =	95.8	*ksi*		
||	note: $F_{nc[2]} \le R_bR_hF_{yc}$				
||					
||	**F_{crw} = min ($F_{crw[1]}$, $F_{crw[2]}$, $F_{crw[3]}$) =**	**50.0**	***ksi***	○ nominal bend buckling resistance for webs without longitudinal stiffeners [AASHTO 6.10.1.9.1]	6-67
||	$F_{crw[1]}$ = 0.9Ek / $(D/t_w)^2$ =	141.9	*ksi*		
||	k = 9 / $(D_c/D)^2$ =	36.2	*[-]*	bend buckling coefficient	
||	$F_{crw[2]} = R_hF_{yc}$ =	50.0	*ksi*		
||	$F_{crw[3]} = F_{yw}/0.7$ =	71.4	*ksi*		
x	**Composite Section in Positive Flexure**				
x					
x				○ Compact Section Criteria	6-98
x		COMPACT		Compact/Noncompact (To qualify as compact, section must meet all the criteria listed below).	
x	Is $F_{yf} \le$ 70ksi satisfied?	YES			
x	Is $D/t_w \le$ 150 satisfied?	YES			
x	D/t_w =	81.6	*[-]*		
x	Is $2D_{cp}/t_w \le$ 3.76 sqrt(E/F_{yc}) satisfied?	YES			
x	$2D_{cp}/t_w$ =	0.0	*[-]*		
x	3.76 sqrt(E/F_{yc}) =	90.6	*[-]*		
x				○ Flexural Resistance for Compact Section [AASHTO 6.10.7.1]	6-101
x	**$M_u + 1/3f_lS_{xt} \le \Phi_fM_n$**	**OK**			
x	$M_u + 1/3f_lS_{xt}$ =	84,738	*kip-in*		
x	Φ_fM_n =	136,224	*kip-in*		
x	note: M_n =	11,352	*kip-ft*		
x					
x	$S_{xt} = M_{yt}/F_{yt}$ =	2199.7	*in^3*	elastic section modulus about the major axis of the section to the tension flange	
x	M_{yt} =	109,985	*kip-in*	yield moment with respect to tension flange	6-252
x	M_n =	136,224	*kip-in*	nominal flexural resistance of the section	

Project: *I-Girder Section Analysis and Design Spreadsheet Demo for DesignSpreadsheets.com Website*	Made By: *admin* Date: *4/12/2006*	Job No: *xy*
Subject: *Spreadsheet Demo*	Checked By: *admin* Date: *4/12/2006*	Sheet No: *xy*

Note: sign convention for this sheet - all moments and stresses are entered and reported as absolute values.

|| indicates that corresponding checks are applicable for section under consideration (based on user's input)

	Variable/Formula	*Value*	*Units*	*Comment*	*AASHTO Page*
x	note: M_n =	11,352	*kip-ft*		
x	if $D_p \le 0.1\ D_t$ then $M_n = M_p$ =	141,433	*kip-in*		
x	if $D_p > 0.1\ D_t$ then $M_n = M_p\ (1.07-0.7D_p/D_t)$ =	136,224	*kip-in*		
x	D_t =	62.625	*in*	total depth of composite section [imported from Mp tab]	
x	D_p =	9.558	*in*	distance from top of concrete deck to the neutral axis of composite section at plastic moment [imported from Mp tab]	
x	M_p =	141,433	*kip-in*	plastic moment of composite section [imported from Mp tab]	
x	note: M_p =	11,786	*kip-ft*		
x	note: $M_n \le 1.3R_hM_y$ =	142,981	*kip-in*		
x	M_y =	109,985	*kip-in*	yield moment [import from My tab]	6-102
	note: M_y =	9,165	*kip-ft*		
x				○ Flexural Resistance for Noncompact Section [AASHTO 6.10.7.2]	
x				- Compression Flange Check	
x	**$f_{bu} \le \Phi_fF_{nc}$**	**OK**			
x	f_{bu} =	39.0	*ksi*		
x	Φ_fF_{nc} =	50.0	*ksi*		
x	$F_{nc} = R_bR_hF_{yc}$ =	50.0	*ksi*	nominal flexural resistance of compression flange	
x					
x				- Tension Flange Check	
x	**$f_{bu} + 1/3f_l \le \Phi_fF_{nt}$**	**OK**			
x	$f_{bu} + 1/3f_l$ =	42.4	*ksi*		
x	Φ_fF_{nt} =	50.0	*ksi*		
x	$F_{nt} = R_hF_{yt}$ =	50.0	*ksi*	nominal flexural resistance of tension flange	
x					
x				○ Ductility Requirement (For Both Compact and Noncompact Sections)	6-105
x	**$D_p \le 0.42\ D_t$**	**OK**			
x	D_p =	9.558	*in*		
x	$0.42\ D_t$ =	26.303	*in*		
||	**Composite Sections in Negative Flexure and Noncomposite Sections**				
||					
||				○ Compact Section Criteria	
||		COMPACT		Compact/Noncompact (To qualify as compact, section must meet all the criteria listed below).	
||	Is $F_{yf} \le$ 70ksi satisfied?	YES			
||	Is $2D_c/t_w \le 5.7\ \mathrm{sqrt}(E/F_{yc})$ satisfied?	YES			
||	$2D_c/t_w$ =	81.4	*[-]*		
||	$5.7\ \mathrm{sqrt}(E/F_{yc})$ =	137.3	*[-]*		
||					
||				○ Flexural Resistance - Discretely Braced Flanges in Compression [AASHTO 6.10.8.1.1]	
||	**$f_{bu} + (1/3)f_l \le \Phi_fF_{nc}$**	**OK**			
||	$f_{bu} + (1/3)f_l$ =	42.4	*ksi*		
||	Φ_fF_{nc} =	43.1	*ksi*		
.				○ Flexural Resistance - Discretely Braced Flanges in Tension	
.	**$f_{bu} + (1/3)f_l \le \Phi_fF_{nt}$**				
.	NOT CONSIDERED			not considered, because the flanges in tension are continously braced by deck in the negative moment region for strength check	

Project: *I-Girder Section Analysis and Design Spreadsheet Demo for DesignSpreadsheets.com Website*	Made By: *admin* Date: *4/12/2006*	Job No: *xy*
Subject: *Spreadsheet Demo*	Checked By: *admin* Date: *4/12/2006*	Sheet No: *xy*

Note: sign convention for this sheet - all moments and stresses are entered and reported as absolute values.

Variable/Formula	*Value*	*Units*	*Comment*	*AASHTO Page*

|| *indicates that corresponding checks are applicable for section under consideration (based on user's input)*

||				○ Flexural Resistance - Continously Braced Flanges in T or C	
||	**$f_{bu} \le \Phi_f R_h F_{yf}$**	**OK**			
||	f_{bu} =	39.0	ksi		
||	$\Phi_f R_h F_{yf}$ =	50.0	ksi		
||	**Shear Resistance [AASHTO 6.10.9]**				6-114
||	**$V_u \le \Phi_v V_n$**	**OK**			
||	V_u =	389	*kip*		
||	$\Phi_v V_n$ =	632	*kip*		

Project: *I-Girder Section Analysis and Design Spreadsheet Demo for DesignSpreadsheets.com Website*	Made By: *admin* Date: *4/12/2006*	Job No: *xy*
Subject: *Spreadsheet Demo*	Checked By: *admin* Date: *4/12/2006*	Sheet No: *xy*

Note: sign convention for this sheet - all moments are reported as absolute values;
negative "P" forces are compressive, positive "P" forces are tensile.

|| *indicates that corresponding checks are applicable for section under consideration (based on user's input)*

DETERMINATION OF PLASTIC MOMENT Mp [AASHTO D6.1, p. 6-250]

Girder 4

Input Taken from Other Tabs

||	b_{bf} =	18	*in*	○ girder dimensions
||	b_{tf} =	18	*in*	
||	t_{bf} =	1.875	*in*	
||	t_{tf} =	1.75	*in*	
||	h_w =	51	*in*	
||	t_w =	0.625	*in*	
||	b_d =	75.468	*in*	○ deck dimensions
||	t_d =	8	*in*	
||	t_h =	0	*in*	○ haunch dimensions

Additional Input

||	F_{yf} =	50.0	*ksi*	specified minimum yield stress of flange
||	F_{yw} =	50.0	*ksi*	specified minimum yield stress of web
||	$F_{y,reinf}$ =	60.0	*ksi*	specified minimum yield stress of reinforcement
||	f_c' =	4.0	*ksi*	minimum specified 28-day compressive strength of concrete
||	β_1 =	1.000	*[-]*	use β_1 = 1 to consider whole concrete block in compression

Outputs - Positive Moment Region

		top coord [in]	bot coord [in]	Height [in]	Force [kips]	Arm to PNA [in]	Moment @ PNA [kip-in]	
x	Ps compression	0.000	8.000	8.000	-2052.7	-5.558	11408.1	concrete
x	Pc compression	8.000	9.558	1.558	-1401.8	-0.779	1091.6	top flange
x	Pc tension	9.558	9.750	0.192	173.2	0.096	16.7	
x	Pw compression	9.750	9.750	0.000	0.0	0.192	0.0	web
x	Pw tension	9.750	60.750	51.000	1593.8	25.692	40947.4	
x	Pt tension	60.750	62.625	1.875	1687.5	52.130	87969.4	bottom flange
x	Total				0.0		141433.1	

x	y = D_p =	9.558	*in*	distance of PNA from the top of section
x	M_p =	141,433	*kip-in*	plastic moment
x	note: M_p =	11,786	*kip-ft*	plastic moment
x	D_{cp} =	0.000	*in*	depth of web in compression at plastic moment
x	D_t =	62.625	*in*	total depth of composite section

Outputs - Negative Moment Region

		bot coord [in]	top coord [in]	Height [in]	Force [kips]	Arm to PNA [in]	Moment @ PNA [kip-in]	
||	Pr	56.625	56.625	-	200.4	27.844	5579.9	reinforcement
||	Pt tension	52.875	54.625	1.750	1575.0	24.969	39325.5	top flange
||	Pt compression	52.875	52.875	0.000	0.0	24.094	0.0	
||	Pw tension	28.781	52.875	24.094	752.9	12.047	9070.3	web
||	Pw compression	1.875	28.781	26.906	-840.8	-13.453	11311.8	
||	Pc compression	0.000	1.875	1.875	-1687.5	-27.844	46986.6	bottom flange

Project: ***I-Girder Section Analysis and Design Spreadsheet Demo for DesignSpreadsheets.com Website***	Made By: *admin* Date: *4/12/2006*	Job No: *xy*
Subject: *Spreadsheet Demo*	Checked By: *admin* Date: *4/12/2006*	Sheet No: *xy*

Note: sign convention for this sheet - all moments are reported as absolute values;
negative "P" forces are compressive, positive "P" forces are tensile.

|| *indicates that corresponding checks are applicable for section under consideration (based on user's input)*

||	Total		0.0	112274.1
||				
||	$y = D_p =$	28.781 *in*	distance of PNA from the bottom of section	
||	$M_p =$	112,274 *kip-in*	plastic moment	
||	note: $M_p =$	9,356 *kip-ft*	plastic moment	
||	$D_{cp} =$	26.906 *in*	depth of web in compression at plastic moment	
||	$D_t =$	56.625 *in*	total depth of composite section	

Note: sign convention for this sheet - all moments and stresses are entered and reported as absolute values.

|| *indicates that corresponding checks are applicable for section under consideration (based on user's input)*

DETERMINATION OF YIELD MOMENT My (for strength check) [AASHTO D6.2, p. 6-252]

Girder 4

||	**Input**			
||				● general
||	M_{D1} =	35,820	*kip-in*	factored moment applied to noncomposite section (1.25 DC1)
||	M_{D2} =	8,994	*kip-in*	factored moment applied to longterm composite section (1.25 DC2 + 1.5 DW)
||	note: M_{D1} =	2985.0	*kip-ft*	
||	note: M_{D2} =	749.5	*kip-ft*	
||	f_c =	39.0	*ksi*	sum of compression flange stresses [import from Stresses tab, for Service II load group]
||	f_t =	39.1	*ksi*	sum of tension flange stresses [import from Stresses tab, for Service I| load group]
||	F_{yf} =	50.0	*ksi*	specified minimum yield strength of flange
||	$F_{y,reinf}$ =	60.0	*ksi*	specified minimum yield strength of reinforcement

x	**Composite Section in Positive Flexure**			
x				
x	**$M_Y = \min(M_{Y[T]}, M_{Y[C]})$ =**	**109,985**	***kip-in***	
x	note: M_Y =	9,165	*kip-ft*	
x				
x				● determine moment to cause yielding in tension (bottom) flange
x	$M_{Y[T]} = M_{D1} + M_{D2} + M_{AD[T]}$ =	109,985	*kip-in*	
x	$M_{AD[T]} = (F_{yf} - M_{D1}/S_{NC[T]} - M_{D2}/S_{LT[T]})\ S_{ST[T]}$ =	65,171	*kip-in*	additional moment applied to short term composite section to cause nominal yielding in tension flange
x	$S_{NC[T]}$ =	1,959.1	*in^3*	noncomposite section modulus
x	$S_{LT[T]}$ =	2,189.0	*in^3*	short-term composite section modulus
x	$S_{ST[T]}$ =	2,360.7	*in^3*	long-term composite section modulus
x				
x				● determine moment to cause yielding in compression (top) flange
x	$M_{Y[C]} = M_{D1} + M_{D2} + M_{AD[C]}$ =	241,760	*kip-in*	
x	$M_{AD[C]} = (F_{yf} - M_{D1}/S_{NC[C]} - M_{D2}/S_{LT[C]})\ S_{ST[C]}$ =	196,946	*kip-in*	
x	$S_{NC[C]}$ =	1,878.9	*in^3*	
x	$S_{LT[C]}$ =	3,435.9	*in^3*	
x	$S_{ST[C]}$ =	6,954.7	*in^3*	
x				
x	$D_c = \|f_c\| / (\|f_c\|+f_t)\ d - t_{fc}$ =	25.552	*in*	depth of web in compression in the elastic range
x	d =	54.625	*in*	depth of steel section
x	t_{fc} =	1.750	*in*	thickness of top flange

||	**Composite Section in Negative Flexure**			
||				
||	**$M_Y = \min(M_{Y[T1]}, M_{Y[T2]}, M_{Y[C]})$ =**	75,145	***kip-in***	
||	note: M_Y =	6,262	*kip-ft*	
||				
||				● determine moment to cause yielding in tension (top) flange
||	$M_{Y[T1]} = M_{D1} + M_{D2} + M_{AD[T1]}$ =	100,163	*kip-in*	

Project: ***I-Girder Section Analysis and Design Spreadsheet Demo for DesignSpreadsheets.com Website***	Made By: *admin* Date: *4/12/2006*	Job No: *xy*
Subject: *Spreadsheet Demo*	Checked By: *admin* Date: *4/12/2006*	Sheet No: *xy*

Note: sign convention for this sheet - all moments and stresses are entered and reported as absolute values.

|| indicates that corresponding checks are applicable for section under consideration (based on user's input)

|| $M_{AD[T1]} = (F_{yf} - M_{D1}/S_{NC[T1]} - M_{D2}/_{COMP[T1]})\ S_{COMP[T1]} =$ 55,349 *kip-in* — additional moment applied to short term composite section to cause nominal yielding in tension flange

|| $S_{NC[T1]} =$ 1,878.9 in^3 — noncomposite section modulus

|| $S_{COMP[T1]} = S_{LT[T1]} = S_{ST[T1]} =$ 2,079.9 in^3 — composite section modulus (concrete deck not effective)

||

|| ● determine moment to cause yielding in reinforcement

|| $M_{Y[T2]} = M_{D2} + M_{AD[T2]} =$ 75,145 *kip-in*

|| $M_{AD[T2]} = (F_{y,reinf} - M_{D1}/S_{NC[T2]})\ S_{COMP[T2]} =$ 66,151 *kip-in*

|| $S_{COMP[T2]} = S_{LT[T2]} = S_{ST[T2]} =$ 1,699.5 in^3

||

|| ● determine moment to cause yielding in compression (bottom) flange

|| $M_{Y[C]} = M_{D1} + M_{D2} + M_{AD[C]} =$ 99,403 *kip-in*

|| $M_{AD[C]} = (F_{yf} - M_{D1}/S_{NC[C]} - M_{D2}/S_{COMP[C]})\ S_{COM[C]} =$ 54,589 *kip-in*

|| $S_{NC[C]} =$ 1,959.1 in^3

|| $S_{COMP[C]} = S_{LT[C]} = S_{ST[C]} =$ 2,004.8 in^3

||

|| $D_c = |f_c| / (|f_c|+f_t)\ d - t_{fc} =$ 25.427 *in* — depth of web in compression in the elastic range

|| $d =$ 54.625 *in* — depth of steel section

|| $t_{bc} =$ 1.875 *in* — thickness of bottom flange

DETERMINATION OF YIELD MOMENT My (for constructibility check)

[AASHTO D6.2, p. 6-252]

|| Input

||

|| $f_c =$ 14.6 *ksi* — sum of compression flange stresses [import from Stresses tab, for DC1]

|| $f_t =$ 15.3 *ksi* — sum of tension flange stresses [import from Stresses tab, for DC1]

|| $F_{yf} =$ 50.0 *ksi* — specified minimum yield strength of flange

x Noncomposite Section in Positive Flexure

x

x $M_Y = \min(S_{NC[T]}F_{yf},\ S_{NC[C]}F_{yf}) =$ 93,947 *kip-in*

x note: $M_Y =$ 7,829 *kip-ft*

x $S_{NC[T]} =$ 1,959.1 in^3 — section modulus for tension flange

x $S_{NC[C]} =$ 1,878.9 in^3 — section modulus for compression flange

x

x

x $D_c = |f_c| / (|f_c|+f_t)\ d - t_{fc} =$ 24.992 *in* — depth of web in compression in the elastic range

x $d =$ 54.625 *in* — depth of steel section

x $t_{fc} =$ 1.750 *in* — thickness of top flange

|| Noncomposite Section in Negative Flexure

||

|| $M_Y = \min(S_{NC[T]}F_{yf},\ S_{NC[C]}F_{yf}) =$ 93,947 *kip-in*

|| note: $M_Y =$ 7,829 *kip-ft*

|| $S_{NC[T]} =$ 1,878.9 in^3 — section modulus for tension flange

|| $S_{NC[C]} =$ 1,959.1 in^3 — section modulus for compression flange

||

|| $D_c = |f_c| / (|f_c|+f_t)\ d - t_{fc} =$ 24.867 *in* — depth of web in compression in the elastic range

Project: ***I-Girder Section Analysis and Design Spreadsheet Demo for DesignSpreadsheets.com Website***	Made By: *admin* Date: *4/12/2006*	Job No: *xy*
Subject: *Spreadsheet Demo*	Checked By: *admin* Date: *4/12/2006*	Sheet No: *xy*

Note: sign convention for this sheet - all moments and stresses are entered and reported as absolute values.

|| *indicates that corresponding checks are applicable for section under consideration (based on user's input)*

||	d =	54.625 *in*	depth of steel section
||	t_{bc} =	1.875 *in*	thickness of bottom flange