research report

Direct Strength Method for Steel Deck

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for the Design of Cold-Formed
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American Iron and Steel Institute

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PREFACE

The American Iron and Steel Institute (AISI) Standards Council selected this project as one of four winning research proposals for its 2014 Small Project Fellowship Program. Project selections were based on several factors, including the potential for long-term impact on the industry; steel industry engagement and co-funding; and results for the AISI standards development committee, the student, and the academic institution.

The objective of this project was to determine and compare the behavior and usable strength of existing floor and roof deck sections with both the Direct Strength Method (DSM) and Effective Width Method (EWM). It is anticipated that the results of this study will guide future research and development efforts.

DIRECT STRENGTH METHOD FOR STEEL DECK

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LIST OF SYMBOLS AND DEFINITIONS

Symbol	Definition
A_g	Gross area of element including stiffeners
b	Flange width
b_e	Effective element width
b_o	Total flat width of stiffened element
b_p	Largest sub-element flat width
f	Stress
F_{cr}	Plate elastic buckling stress
F_y	Yield Stress
h	Width of elements adjoining stiffened element (depth of web)
I_G	Moment of inertia of gross section
I_{sp}	Moment of inertia of stiffener about centerline of flat portion of element
I'_x	Moment of inertia about element's own axis
k	Plate buckling coefficient
k_d	Plate buckling coefficient for distortional buckling
k_{loc}	Plate buckling coefficient for local sub-element buckling
L	Element length
M_{crd}	Critical elastic distortional buckling moment
M_{cre}	Critical elastic lateral-torsional buckling moment
M_{crl}	Critical elastic local buckling moment
M_n	Nominal flexural strength
M_{nd}	Nominal flexural strength for distortional buckling

Mn_{DSM}	Nominal flexural strength calculated using direct strength method
M_{ne}	Nominal flexural strength for lateral-torsional buckling
Mn_{EWM}	Nominal flexural strength calculated using effective width method
M_{nl}	Nominal flexural strength for local buckling
M_y	Yield Moment (S_gF_y)
n	Number of stiffeners in element
R	Modification factor for distortional plate buckling coefficient
S_e	Elastic section modulus of effective section
S_g, S_{xx}	Elastic section modulus of gross section
t	Thickness
w	Actual element width
$\bar{y}$	Distance from neutral axis to extreme fiber of section
β	Coefficient
γ	Coefficient
δ	Coefficient
Θ	Web angle from horizontal
Θ_{Stiff}	Stiffener angle from horizontal
λ, λ_l	Slenderness factors
ρ	Reduction factor

ABSTRACT

With the proposed reorganization of the AISI S100 Standard, the Direct Strength Method (DSM) will take a position of equal footing with the Equivalent Width Method (EWM) for calculating strength. The majority of previous DSM studies focus on C and Z profiles while little study of panel sections, especially steel deck sections, has been performed. A study was undertaken to determine and compare the behavior and usable strength of existing floor and roof deck sections with both DSM and EWM. The Cornell University – Finite Strip Method (CUFSM) was used for the elastic buckling analysis, taking into account the wide, continuous nature of installed deck sections. Flexural capacity was analyzed for positive and negative flexure to account for gravity loading as well as uplift of the steel deck sections. We have included graphical representations of the relationships for DSM strength to the EWM strength ratio vs. material width to thickness ratio. While we are not exactly sure what the relationships mean yet, DSM strength seems to suffer vs. EWM strength for sections with relatively wide and thin compression flanges or in other words, large b/t ratios.

CHAPTER 1: INTRODUCTION

1.0 Acknowledgements

The presented research has been performed with the financial support of the American Iron and Steel Institute and the Steel Deck Institute.

1.1 Research Goals

As the Direct Strength Method (DSM) will be taking equal footing as the Effective Width Method (EWM) in the proposed reorganization of the AISI S100, we set following goals: Firstly, we aimed to analyze a variety of existing floor and roof deck sections to observe the behavior and compare the usable flexural strengths using both DSM and EWM. DSM has mostly been previously applied to C and Z profiles so it was necessary to develop a finite strip method (FSM) model that would accurately model and account for multi-web deck sections installed in an adjacent fashion. Once we developed a FSM model that would accurately represent installed floor and roof deck, we studied potential enhancements to existing deck sections that would take advantage of DSM (i.e. DSM predicts higher flexural strength than EWM).

1.2 Direct Strength Method

"A new design method: Direct Strength, has been created that aims to alleviate the current complexity, ease calculation, provide a more robust and flexible design procedure, and integrate with available, established, numerical methods" (DSM Design Guide *Preface*).

The Direct Strength Method (DSM) is one method of analyzing cold-formed steel (wide, light gauge) members. In DSM, the elastic buckling capacity is determined over the entire cross section rather than neglecting less "effective" portions of the cross section.

In order to apply DSM, the elastic local, distortional, and global buckling capacities are first computed. Graphical representations of local, distortional, and global buckling are illustrated below in Figures 1, 2, and 3 respectively. The lateral-torsional buckling, local buckling, and distortional buckling flexural strengths are calculated to observe the governing buckling mode per DSM 1.2.2.1, 1.2.2.2, and 1.2.2.3. In this study, we used the Cornell University Finite Strip Method to find the elastic local, distortional, and global buckling capacities.

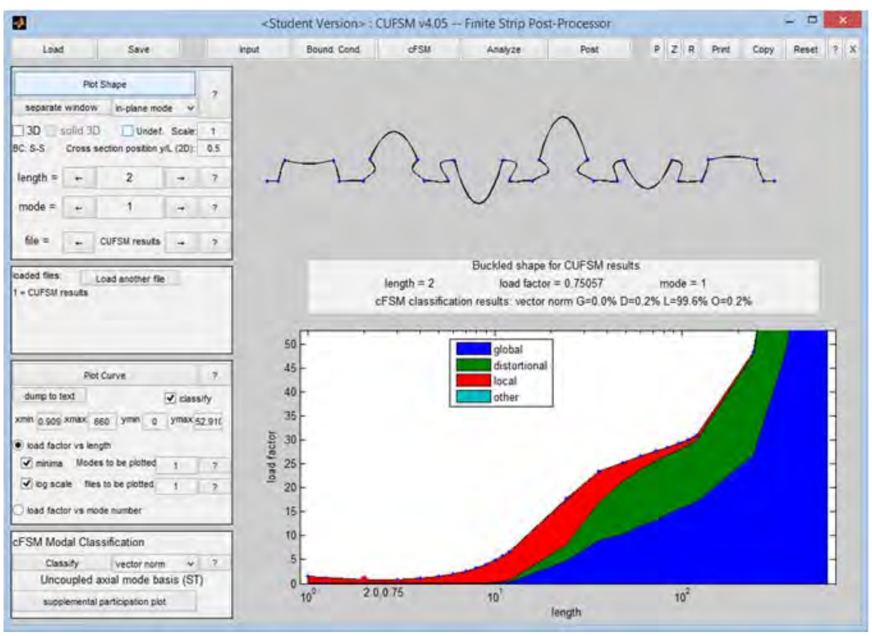


Figure 1 - 1.5B 22GA Deck 33 KSI Local Buckling (CUFSM Output)

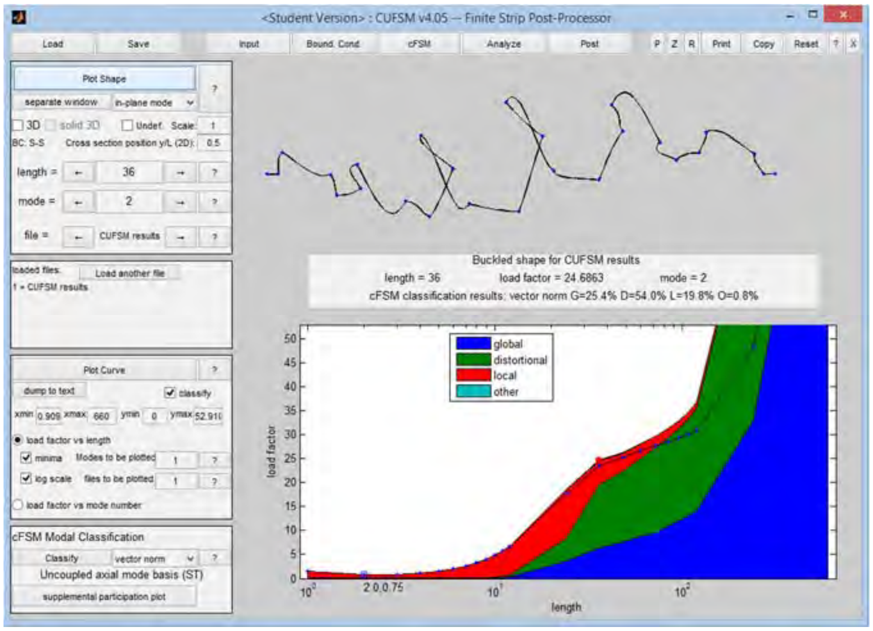


Figure 2 - 1.5B 22GA Deck 33 KSI Distortional Buckling (CUFSM Output)

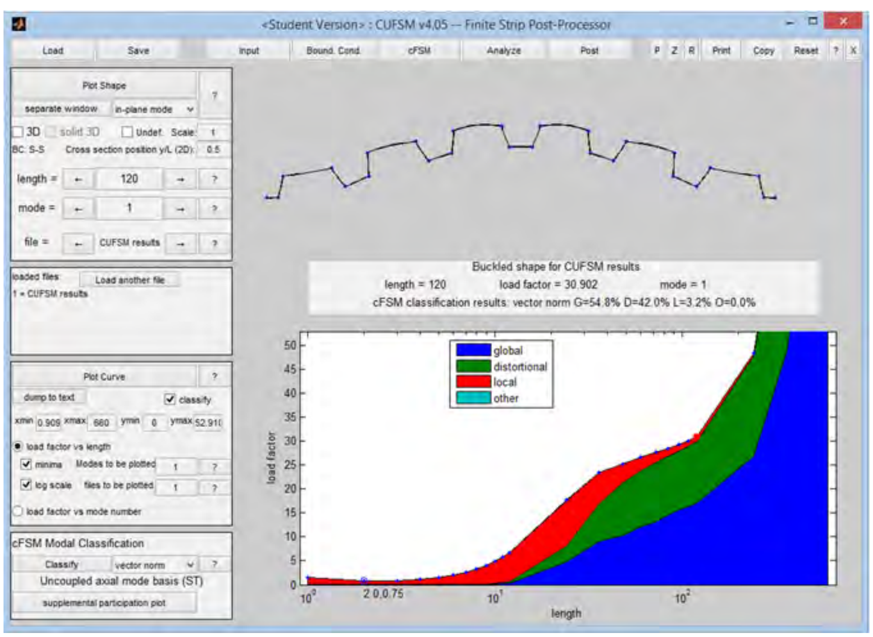


Figure 3 - 1.5B 22GA Deck 33 KSI Global Buckling (CUFSM Output)

1.3 Effective Width Method

The Effective Width Method (EWM) is another method for analyzing cold-formed steel members. In the EWM, an effective width of compression elements is computed and used as the lightly stressed areas, near the center of an element, are neglected. The regions near junctions or stiffeners are considered to be fully effective, as these areas are most effective in resisting the applied stress. Figure 4 shows the actual compression element and the effective width, b, of the element when subjected to compressive stress.

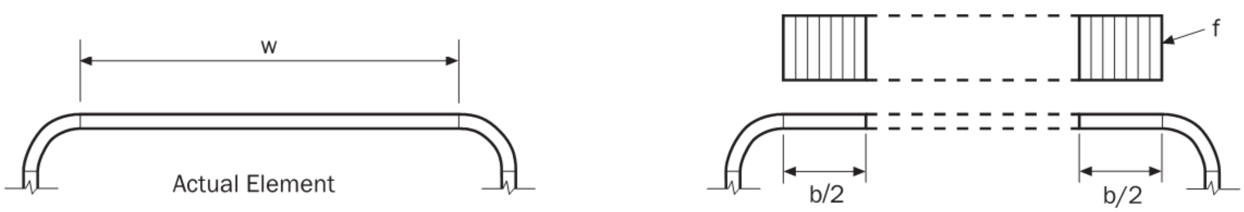


Figure 4 – Flange under Compressive Stress | Effective Element Width, b

The same stress concentrations can be seen for a web element experiencing a stress gradient in Figure 5.

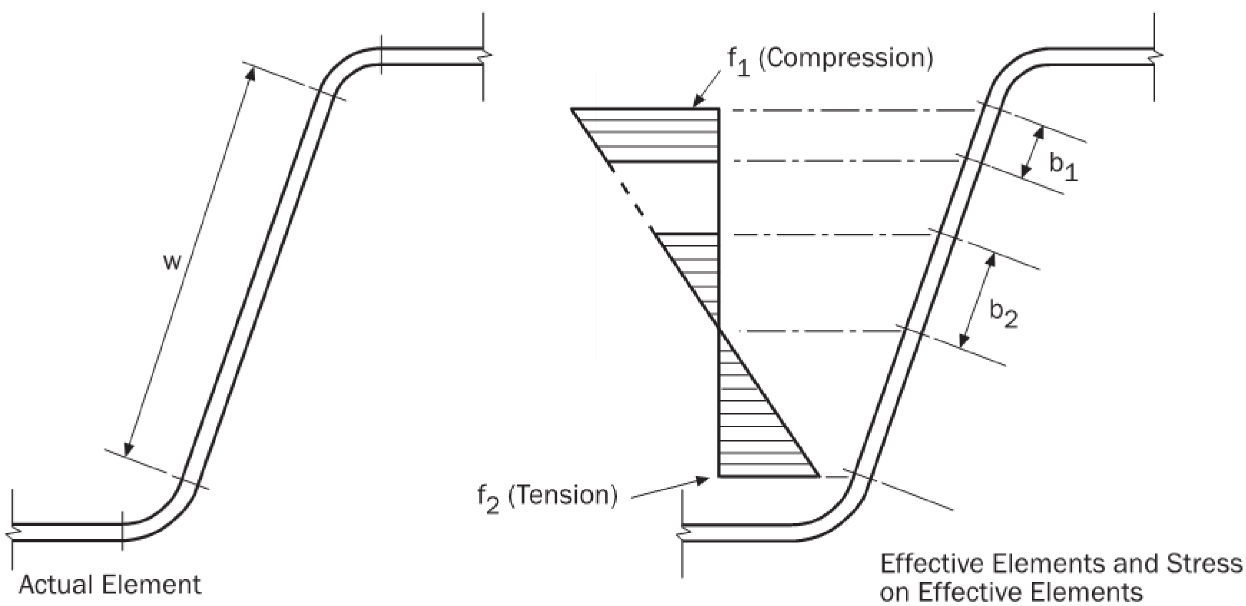


Figure 5 - Web under Stress Gradient

Once the effective width of a compression element is calculated, the effective section properties, center of gravity, and moment of inertia can be found by applying the parallel axis theorem in a tabular format as shown in Table 1.

Table 1 - Parallel Axis Theorem Applied to Obtain Effective Section Properties

Element	Quantity	ΣL	y from top fiber	ΣLy (in.^{2})	ΣLy^2 (in.^{3})	ΣI'$_x$ (in.^{3})
Lip	2	0.660	0.998	0.659	0.657	--
Bottom Corner	18	2.083	0.870	1.812	1.577	0.000
Web	18	22.386	0.508	11.370	5.775	1.513
Top Corner	18	2.083	0.146	0.303	0.044	0.000
Top Flange	9	7.071	0.018	0.127	0.002	--
Bottom Flange	8	6.286	0.998	6.272	6.259	--
	Σ	40.568		20.543	14.314	1.513

Solved $\bar{y}$ =	$\Sigma Ly/\Sigma L$ =	0.506	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max($\bar{y}$, h - $\bar{y}$) =	0.509	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2 \Sigma L]t$ =	0.194	in.^{4}
S_e =	$I_x/\bar{y}$ =	0.381	in.^{3}

As the effective width of an element is dependent on the location of the neutral axis and the neutral axis is dependent on the effective width of an element, this becomes an iterative process involving a guess as to where the neutral axis actually lies. Often, an initial guess of the gross cross-sectional neutral axis is used. After the first iteration, the solved location of the neutral axis can be used as the new guess value until the guess location and the solved location are in agreement.

1.4 Cornell University Finite Strip Method

The Cornell University Finite Strip Method (CUFSM) is a tool that provides cross-section elastic buckling solutions. This powerful program allows the user to define a cross-section based on nodal coordinates, member end designations, fixities, etc. CUFSM allows the user to apply axial and flexure stress and observe the elastic buckling solutions over a variety of user-defined unbraced lengths.

The analysis procedure is "specialized to apply to plate deformations beyond conventional beam theory. The semi-analytical finite strip method is a variant of the more common finite element method. A thin-walled cross-section is discretized into a series of longitudinal strips, or elements. Based on these strips elastic and geometric stiffness matrices can be formulated" (Ben Schafer).

1.5 Deck Sections

This study observes the comparison and behavior of DSM and EWM for both stiffened and unstiffened deck sections. The unstiffened deck sections are 1F and 1.5B. The stiffened deck sections are 1.5B, 2C, and 3C. The stiffened 1.5B Deck section is a non-standard shape. As a point of reference, we added the 2C compression flange stiffener to the compression flange of the 1.5B Deck section and performed the analysis to observe the benefits. The 1.5B and 2C Deck both include flange stiffeners 0.37 inches deep and 1.25 inches wide. The 3C Deck includes flange stiffeners 0.37 inches deep and 1 inch wide. Each deck section was checked for positive and negative flexure. Deck sections symmetric about the axis they bend in were analyzed for flexure in one direction. Each deck section was checked for yield stresses of 33, 40, 50, and 60 KSI at various gage thicknesses shown in Table 2. No cold working or cold forming was done to strengthen the deck sections.

Table 2 - Range of Yield Stresses and Thicknesses for Deck Sections

Deck Type	Yield Stress (KSI)	Thickness (GA)
1F	33, 40, 50, 60	26, 24, 22, 20
1.5B	33, 40, 50, 60	24, 22, 20, 18, 16
2C	33, 40, 50, 60	22, 20, 18, 16
3C	33, 40, 50, 60	22, 20, 18, 16

CHAPTER 2: PROCESS OF MODELING AND ANALYSIS

2.0 DSM Analysis Procedure

For DSM analysis, we developed a preprocessor to process input files for the elastic buckling analysis done with CUFSM. We then applied the CUFSM output (load factors) to the DSM equations to predict strength.

2.1 DSM Preprocessor

In order to run CUFSM to obtain the elastic buckling solutions, the user must define the cross-section's parameters. CUFSM takes in information such as the material properties, nodes, elements, and boundary conditions. As it can be very tedious to calculate nodal locations, assign member end designations, and enter other parameters manually, a preprocessor was created to expedite the process.

A preprocessor processes its input data to produce output that is used as input for another program. In this case, a MATLAB code was written to preprocess the information required to run CUFSM. This eased the process of segmenting and refining members to obtain more accurate results (i.e. the curved corners at joints could be segmented into many line elements that adequately represent a curve).

The preprocessor used in this study produced the input data for the Nodes, Members, and Lengths input areas for CUFSM. Once the information was entered, program files for each deck section and each thickness were retained for later accessibility for analyzing the deck sections at a variety of thicknesses and yield stresses.

2.2 DSM Deck Model

With Dr. Ben Schafer's advice, we ran two sets of models for each deck section: Curved Corner models and Straight Corner models. Curved corners were added at each point an element would change direction (i.e. the corners where the web and flange meet as well as where the flange and stiffener meet). Although the curved corner models provided more representative elastic buckling solutions, straight corner models, where no curvature appears at the element junctions, were modeled to accurately capture the buckling classification. The straight corner models were not used to evaluate strength because the models would have overly penalized the DSM by misrepresenting the actual flat length of the compression flange. The end nodal locations of the deck profile were restrained to account for adjacent deck sections and represent the wide and continuous nature of installed floor and roof deck.

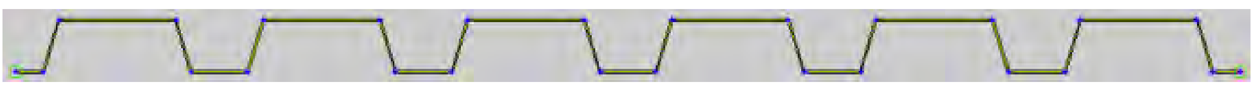


Figure 6 - Straight Corner Model | Buckling Modes

Figure 7 - Curved Corner Model | Elastic Strength

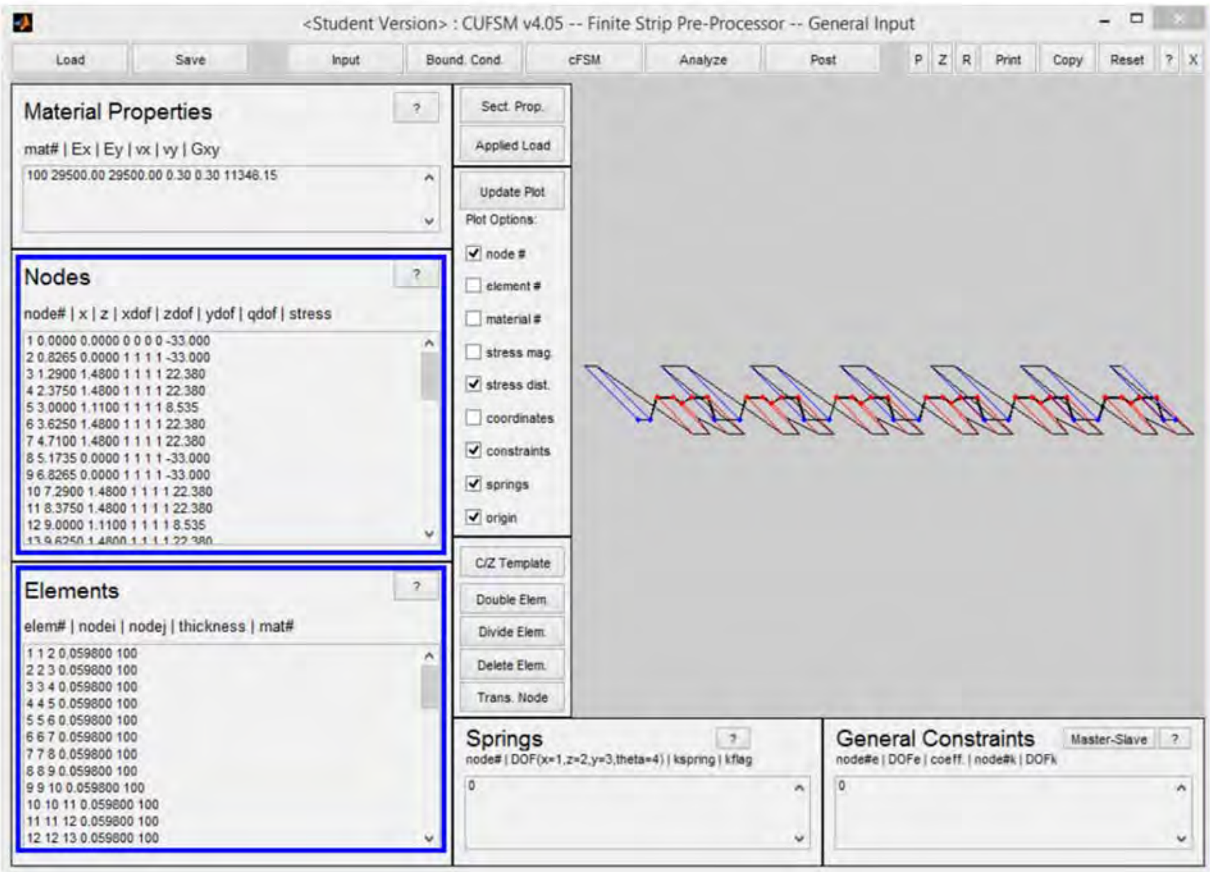


Figure 8 - CUFSM General Input

2.3 DSM Deck Analysis

The deck profile models were analyzed at stresses of 33, 40, 50, and 60 KSI for positive flexure and likewise at stresses of -33, -40, -50, and -60 KSI for negative flexure for a variety of unbraced lengths ranging from 1 inch to 50 feet. The CUFSM output supplies the load factors (nominal buckling moment to yield moment) which are used as input for the strength prediction for the deck profile, Mn_{DSM}.

2.4 EWM Deck Analysis

As stated above, for EWM, an effective width of compression elements is computed and used as the lightly stressed areas, near the center of an element, are neglected. For each deck section, the parallel axis theorem was used in a tabular format to provide the effective section properties to obtain the effective nominal flexural strength using EWM, Mn_{EWM}. The deck sections bend about their neutral axis for positive and negative flexure. The compression elements of the cross-section consist of the compression flange as well as a portion of the web element. The junctions are considered to be fully effective. For each deck section at each variety of thickness and stress, the webs were found to be fully effective. Only the compression flange then needed to be computed for its effective width before iterating to convergence to obtain the nominal flexural capacity of the effective section, Mn_{EWM}.

CHAPTER 3: ANALYSIS | 1F DECK | ± BENDING

3.0 Executive Summary

The Direct Strength Method predicted higher strengths for all of the 1F Deck sections analyzed for positive and negative flexure in this study, 33-40KSI and 26-20GA. DSM is able to take advantage the short, flat compression flange. The nominal moment capacity ratio (Mn_{DSM}/Mn_{EWM}) ranged between 1.060 and 1.126.

3.1 Mn_{DSM} / Mn_{EWM} vs. Thickness Plot

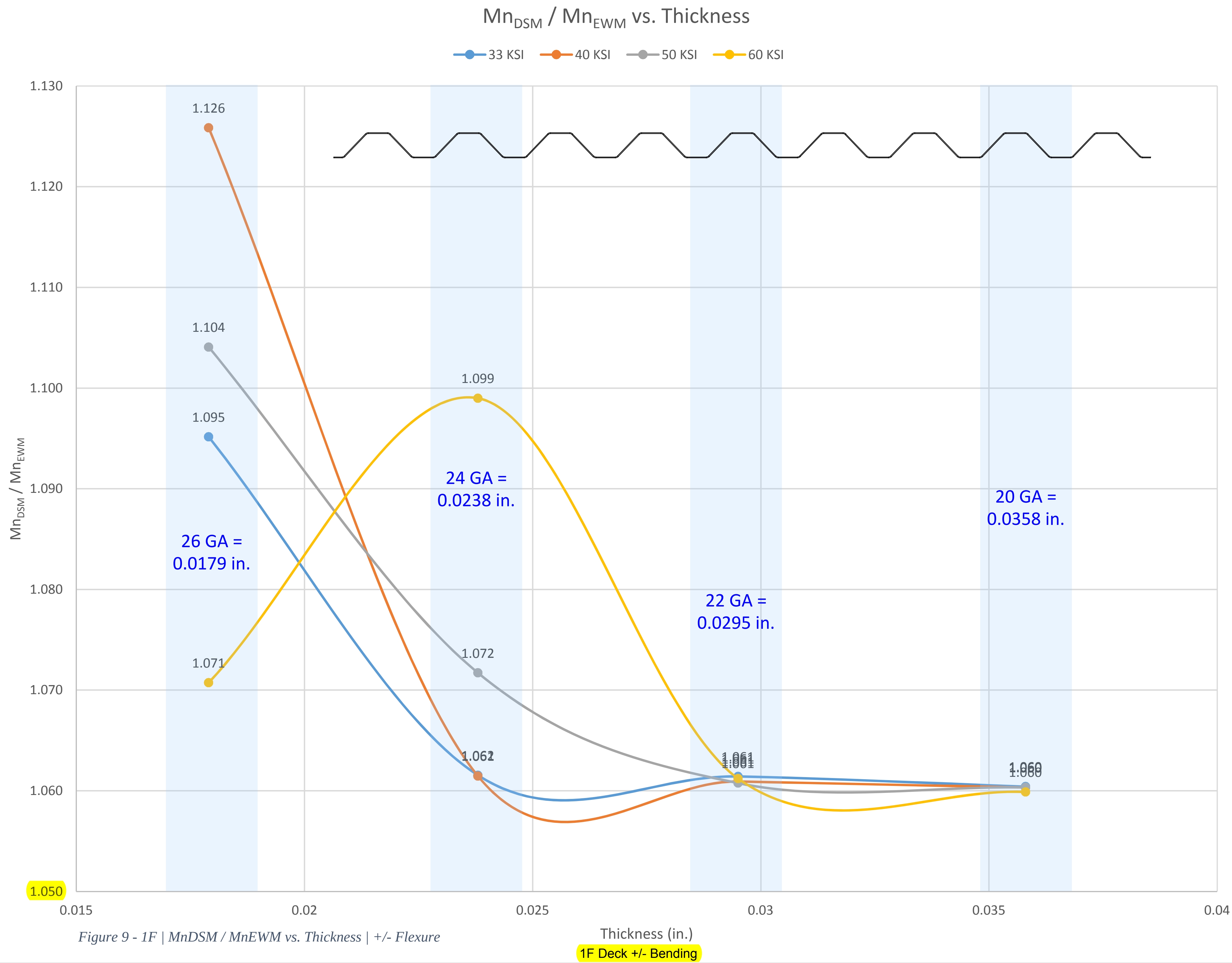


Figure 9 - 1F | MnDSM / MnEWM vs. Thickness | +/- Flexure

3.2 Analysis Results Summary

Table 3 - 1F | Analysis Results Summary | +/- Flexure

1F DECK - 33 KSI				
Gage	**26**	**24**	**22**	**20**
Thickness	0.0179	0.0238	0.0295	0.0358
Curve Radius	0.1340	0.1369	0.1398	0.1429
I_G (CUFSM)	0.103	0.137	0.170	0.206
y-bar (CUFSM)	0.500	0.503	0.506	0.509
Sxx	0.206	0.272	0.335	0.404
My	6.79	8.97	11.06	13.34
Mn_{DSM}	6.79	8.97	11.06	13.34
Mn_{EWM}	6.2	8.45	10.42	12.58
% ERROR	8.689%	5.797%	5.787%	5.697%

1F DECK - 33 KSI				
Thickness		Mn_{DSM}	Mn_{EWM}	Mn_{DSM} / Mn_{EWM}
26	0.0179	6.79	6.20	1.095
24	0.0238	8.97	8.45	1.062
22	0.0295	11.06	10.42	1.061
20	0.0358	13.34	12.58	1.060

1F DECK - 40 KSI				
Gage	**26**	**24**	**22**	**20**
Thickness	0.0179	0.0238	0.0295	0.0358
Curve Radius	0.1340	0.1369	0.1398	0.1429
I_G (CUFSM)	0.103	0.137	0.170	0.206
y-bar (CUFSM)	0.500	0.503	0.506	0.509
Sxx	0.206	0.272	0.335	0.404
My	8.23	10.88	13.41	16.17
Mn_{DSM}	8.23	10.88	13.41	16.17
Mn_{EWM}	7.31	10.25	12.64	15.25
% ERROR	11.179%	5.790%	5.742%	5.690%

1F DECK - 40 KSI				
Thickness		Mn_{DSM}	Mn_{EWM}	Mn_{DSM} / Mn_{EWM}
26	0.0179	8.23	7.31	1.126
24	0.0238	10.88	10.25	1.061
22	0.0295	13.41	12.64	1.061
20	0.0358	16.17	15.25	1.060

1F DECK - 50 KSI				
Gage	**26**	**24**	**22**	**20**
Thickness	0.0179	0.0238	0.0295	0.0358
Curve Radius	0.1340	0.1369	0.1398	0.1429
I_G (CUFSM)	0.103	0.137	0.170	0.206
y-bar (CUFSM)	0.500	0.503	0.506	0.509
Sxx	0.206	0.272	0.335	0.404
My	10.29	13.60	16.76	20.21
Mn_{DSM}	9.76	13.60	16.76	20.21
Mn_{EWM}	8.84	12.69	15.8	19.06
% ERROR	9.426%	6.691%	5.728%	5.690%

1F DECK - 50 KSI				
Thickness		Mn_{DSM}	Mn_{EWM}	Mn_{DSM} / Mn_{EWM}
26	0.0179	9.76	8.84	1.104
24	0.0238	13.60	12.69	1.072
22	0.0295	16.76	15.80	1.061
20	0.0358	20.21	19.06	1.060

1F DECK - 60 KSI				
Gage	**26**	**24**	**22**	**20**
Thickness	0.0179	0.0238	0.0295	0.0358
Curve Radius	0.1340	0.1369	0.1398	0.1429
I_G (CUFSM)	0.103	0.137	0.170	0.206
y-bar (CUFSM)	0.500	0.503	0.506	0.509
Sxx	0.206	0.272	0.335	0.404
My	12.35	16.32	20.11	24.25
Mn_{DSM}	11.05	16.32	20.11	24.25
Mn_{EWM}	10.32	14.85	18.95	22.88
% ERROR	6.606%	9.007%	5.768%	5.649%

1F DECK - 60 KSI				
Thickness		Mn_{DSM}	Mn_{EWM}	Mn_{DSM} / Mn_{EWM}
26	0.0179	11.05	10.32	1.071
24	0.0238	16.32	14.85	1.099
22	0.0295	20.11	18.95	1.061
20	0.0358	24.25	22.88	1.060

3.3 Direct Strength Method Calculations

Date:	11/26/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:								
	Deck:	1F						
	Gage:	20	GA					
	Strength:	33	KSI					
	M_y =	13.34	kip-in				Length:	
local	$M_{cr\ell}/M_y$ =	8.29880	$M_{cr\ell}$ =	110.70599	kip-in		1	in
dist.	M_{crd}/M_y =	5.00000	M_{crd} =	66.7	kip-in		-	in
global	M_{cre}/M_y =	5.00000	M_{cre} =	66.7	kip-in		-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **13.34** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ $\qquad (Eq.\ 1.2.2\text{-}7)$

λ_ℓ = 0.35 (local-global slenderness)

$M_{n\ell}$ = **13.34** kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ $\qquad (Eq.\ 1.2.2\text{-}10)$

λ_d = 0.45 (distortional slenderness)

M_{nd} = **13.34** kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 13.34 kip-in (local-global controls)

Date:	11/26/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1F					
	Gage:	20 GA					
	Strength:	40 KSI					
	M_y =	16.17	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	6.84650	$M_{cr\ell}$ =	110.70791	kip-in	1	in
dist.	M_{crd}/M_y =	5.00000	M_{crd} =	80.85	kip-in	-	in
global	M_{cre}/M_y =	5.00000	M_{cre} =	80.85	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1-\frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **16.17** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1-0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.38 (local-global slenderness)

$M_{n\ell}$ = **16.17** kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1-0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.45 (distortional slenderness)

M_{nd} = **16.17** kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 16.17 kip-in (local-global controls)

Date:	11/26/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1F						
	Gage:	20	GA					
	Strength:	50	KSI					
	M_y =	20.21	kip-in				Length:	
local	$M_{cr\ell}/M_y$ =	5.47720	$M_{cr\ell}$ =	110.69421	kip-in		1	in
dist.	M_{crd}/M_y =	5.00000	M_{crd} =	101.05	kip-in		-	in
global	M_{cre}/M_y =	5.00000	M_{cre} =	101.05	kip-in		-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 20.21 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.43 (local-global slenderness)

$M_{n\ell}$ = 20.21 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.45 (distortional slenderness)

M_{nd} = 20.21 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 20.21 kip-in (local-global controls)

Date:	11/26/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1F						
	Gage:	20	GA					
	Strength:	60	KSI					
	M_y =	24.25	kip-in				Length:	
local	$M_{cr\ell}/M_y$ =	4.56430	$M_{cr\ell}$ =	110.68428	kip-in		1	in
dist.	M_{crd}/M_y =	5.00000	M_{crd} =	121.25	kip-in		-	in
global	M_{cre}/M_y =	5.00000	M_{cre} =	121.25	kip-in		-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **24.25** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.47 (local-global slenderness)

$M_{n\ell}$ = **24.25** kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.45 (distortional slenderness)

M_{nd} = **24.25** kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 24.25 kip-in (local-global controls)

Date:	11/26/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1F				
	Gage:	22 GA				
	Strength:	33 KSI				
	M_y =	11.06	kip-in		Length:	
local	$M_{cr\ell}/M_y$ =	5.68610	$M_{cr\ell}$ =	62.888266 kip-in	1	in
dist.	M_{crd}/M_y =	5.00000	M_{crd} =	55.3 kip-in	-	in
global	M_{cre}/M_y =	5.00000	M_{cre} =	55.3 kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 11.06 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.42 (local-global slenderness)

$M_{n\ell}$ = 11.06 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.45 (distortional slenderness)

M_{nd} = 11.06 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 11.06 kip-in (local-global controls)

Date:	11/26/14		calc by:	RKD	
			check by:	TS	

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1F				
	Gage:	22 GA				
	Strength:	40 KSI				
	M_y =	13.41	kip-in		Length:	
local	$M_{cr\ell}/M_y$ =	4.69110	$M_{cr\ell}$ =	62.907651 kip-in	1	in
dist.	M_{crd}/M_y =	5.00000	M_{crd} =	67.05 kip-in	-	in
global	M_{cre}/M_y =	5.00000	M_{cre} =	67.05 kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 13.41 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.46 (local-global slenderness)

$M_{n\ell}$ = 13.41 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.45 (distortional slenderness)

M_{nd} = 13.41 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 13.41 kip-in (local-global controls)

Date:	11/26/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1F				
	Gage:	22 GA				
	Strength:	50 KSI				
	M_y =	16.76	kip-in		Length:	
local	$M_{cr\ell}/M_y$ =	3.75280	$M_{cr\ell}$ =	62.896928 kip-in	1	in
dist.	M_{crd}/M_y =	5.00000	M_{crd} =	83.8 kip-in	-	in
global	M_{cre}/M_y =	5.00000	M_{cre} =	83.8 kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **16.76** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.52 (local-global slenderness)

$M_{n\ell}$ = **16.76** kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.45 (distortional slenderness)

M_{nd} = **16.76** kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 16.76 kip-in (local-global controls)

Date:	11/26/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1F					
	Gage:	22	GA				
	Strength:	60	KSI				
	M_y =	20.11	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	3.12740	$M_{cr\ell}$ =	62.892014	kip-in	1	in
dist.	M_{crd}/M_y =	5.00000	M_{crd} =	100.55	kip-in	-	in
global	M_{cre}/M_y =	5.00000	M_{cre} =	100.55	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **20.11** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.57 (local-global slenderness)

$M_{n\ell}$ = **20.11** kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.45 (distortional slenderness)

M_{nd} = **20.11** kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 20.11 kip-in (local-global controls)

Date: 11/26/14 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1F				
	Gage:	24 GA				
	Strength:	33 KSI				
	M_y =	8.97	kip-in		Length:	
local	$M_{cr\ell}/M_y$ =	3.72310	$M_{cr\ell}$ =	33.396207 kip-in	1	in
dist.	M_{crd}/M_y =	5.00000	M_{crd} =	44.85 kip-in	-	in
global	M_{cre}/M_y =	5.00000	M_{cre} =	44.85 kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **8.97** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.52 (local-global slenderness)

$M_{n\ell}$ = **8.97** kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.45 (distortional slenderness)

M_{nd} = **8.97** kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 8.97 kip-in (local-global controls)

Date: 11/26/14 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1F				
	Gage:	24 GA				
	Strength:	40 KSI				
	M_y =	10.88	kip-in		Length:	
local	$M_{cr\ell}/M_y$ =	3.07160	$M_{cr\ell}$ =	33.419008 kip-in	1	in
dist.	M_{crd}/M_y =	5.00000	M_{crd} =	54.4 kip-in	-	in
global	M_{cre}/M_y =	5.00000	M_{cre} =	54.4 kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1-\frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 10.88 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell}=M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell}=\left(1-0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.57 (local-global slenderness)

$M_{n\ell}$ = 10.88 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1-0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5}M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.45 (distortional slenderness)

M_{nd} = 10.88 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 10.88 kip-in (local-global controls)

Date: 11/26/14 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:							
	Deck:	1F					
	Gage:	24	GA				
	Strength:	50	KSI				
	M_y =	13.60	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	2.45730	$M_{cr\ell}$ =	33.41928	kip-in	1	in
dist.	M_{crd}/M_y =	5.00000	M_{crd} =	68	kip-in	-	in
global	M_{cre}/M_y =	5.00000	M_{cre} =	68	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq. 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq. 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq. 1.2.2\text{-}3)$$

M_{ne} = 13.60 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq. 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq. 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.64 (local-global slenderness)

$M_{n\ell}$ = 13.60 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq. 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq. 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.45 (distortional slenderness)

M_{nd} = 13.60 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 13.60 kip-in (local-global controls)

Date: 11/26/14 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1F					
	Gage:	24	GA				
	Strength:	60	KSI				
	M_y =	16.32	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	2.04770	$M_{cr\ell}$ =	33.418464	kip-in	1	in
dist.	M_{crd}/M_y =	5.00000	M_{crd} =	81.6	kip-in	-	in
global	M_{cre}/M_y =	5.00000	M_{cre} =	81.6	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1-\frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **16.32** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell}=M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell}=\left(1-0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.70 (local-global slenderness)

$M_{n\ell}$ = **16.32** kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1-0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5}M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.45 (distortional slenderness)

M_{nd} = **16.32** kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 16.32 kip-in (local-global controls)

Date:	11/26/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1F					
	Gage:	26 GA					
	Strength:	33 KSI					
	M_y =	6.79	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	2.12660	$M_{cr\ell}$ =	14.439614	kip-in	1	in
dist.	M_{crd}/M_y =	5.00000	M_{crd} =	33.95	kip-in	-	in
global	M_{cre}/M_y =	5.00000	M_{cre} =	33.95	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1-\frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **6.79** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1-0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ =	0.69	(local-global slenderness)
$M_{n\ell}$ =	**6.79** kip-in	(fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1-0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5}M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d =	0.45	(distortional slenderness)
M_{nd} =	**6.79** kip-in	(fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 6.79 kip-in (local-global controls)

Date: 11/26/14 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:							
	Deck:	1F					
	Gage:	26	GA				
	Strength:	40	KSI				
	M_y =	8.23	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	1.75450	$M_{cr\ell}$ =	14.439535	kip-in	1	in
dist.	M_{crd}/M_y =	5.00000	M_{crd} =	41.15	kip-in	-	in
global	M_{cre}/M_y =	5.00000	M_{cre} =	41.15	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **8.23** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.75 (local-global slenderness)

$M_{n\ell}$ = **8.23** kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.45 (distortional slenderness)

M_{nd} = **8.23** kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 8.23 kip-in (local-global controls)

Date: 11/26/14 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:							
	Deck:	1F					
	Gage:	26 GA					
	Strength:	50 KSI					
	M_y =	10.29	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	1.40360	$M_{cr\ell}$ =	14.443044	kip-in	1	in
dist.	M_{crd}/M_y =	5.00000	M_{crd} =	51.45	kip-in	-	in
global	M_{cre}/M_y =	5.00000	M_{cre} =	51.45	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **10.29** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ $(Eq.\ 1.2.2\text{-}7)$

λ_ℓ = 0.84 (local-global slenderness)

$M_{n\ell}$ = **9.76** kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ $(Eq.\ 1.2.2\text{-}10)$

λ_d = 0.45 (distortional slenderness)

M_{nd} = **10.29** kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 9.76 kip-in (local-global controls)

Date:	11/26/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1F					
	Gage:	26	GA				
	Strength:	60	KSI				
	M_y =	12.35	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	1.16970	$M_{cr\ell}$ =	14.445795	kip-in	1	in
dist.	M_{crd}/M_y =	5.00000	M_{crd} =	61.75	kip-in	-	in
global	M_{cre}/M_y =	5.00000	M_{cre} =	61.75	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1-\frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **12.35** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell}=M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell}=\left(1-0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (*Eq.* 1.2.2-7)

λ_ℓ = 0.92 (local-global slenderness)

$M_{n\ell}$ = **11.05** kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1-0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5}M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (*Eq.* 1.2.2-10)

λ_d = 0.45 (distortional slenderness)

M_{nd} = **12.35** kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 11.05 kip-in (local-global controls)

3.4 Effective Width Method Calculations

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1F	
Gage:	20	GA
Strength:	33	ksi
Thickness:	0.0358	in.
Total Height:	1.016	in.
Radius:	0.1429	in.
θ:	46.39	deg
θ:	0.810	rad
Curve I'_x:	0.000021	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.330	0.998
Corners	0.116	0.146
Bottom Flange	0.786	0.998
Web	1.244	0.508
Top Flange	0.786	0.018

Guess ȳ: 0.506 in.

Stress in Flange:	32.803	ksi
k:	4	
Fcr:	221.414	ksi
λ:	0.385	
ρ:	1.113	
Effective Width:	0.786	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	ΣLy² ($in.^3$)	ΣI'x ($in.^3$)
Lip	2	0.660	0.998	0.659	0.657	--
Bottom Corner	18	2.083	0.870	1.812	1.577	0.000
Web	18	22.386	0.508	11.370	5.775	1.513
Top Corner	18	2.083	0.146	0.303	0.044	0.000
Top Flange	9	7.071	0.018	0.127	0.002	--
Bottom Flange	8	6.286	0.998	6.272	6.259	--
	Σ	40.568		20.543	14.314	1.513

Solved ȳ =	ΣLy/ΣL =	0.506	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.509	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2 \Sigma L]t$ =	0.194	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.381	$in.^3$
Mn =	**Se*Fy =**	**12.58**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1F	
Gage:	20	GA
Strength:	40	ksi
Thickness:	0.0358	in.
Total Height:	1.016	in.
Radius:	0.1429	in.
θ:	46.39	deg
θ:	0.810	rad
Curve I'_x:	0.000021	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.330	0.998
Corners	0.116	0.146
Bottom Flange	0.786	0.998
Web	1.244	0.508
Top Flange	0.786	0.018

Guess ȳ: 0.506 in.

Stress in Flange:	39.762	ksi
k:	4	
Fcr:	221.414	ksi
λ:	0.424	
ρ:	1.135	
Effective Width:	0.786	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	ΣLy² ($in.^3$)	ΣI'ₓ ($in.^3$)
Lip	2	0.660	0.998	0.659	0.657	--
Bottom Corner	18	2.083	0.870	1.812	1.577	0.000
Web	18	22.386	0.508	11.370	5.775	1.513
Top Corner	18	2.083	0.146	0.303	0.044	0.000
Top Flange	9	7.071	0.018	0.127	0.002	--
Bottom Flange	8	6.286	0.998	6.272	6.259	--
	Σ	40.568		20.543	14.314	1.513

Solved ȳ =	ΣLy/ΣL =	0.506	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.509	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2 \Sigma L]t$ =	0.194	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.381	$in.^3$
Mn =	**Se*Fy =**	**15.25**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1F	
Gage:	20	GA
Strength:	50	ksi
Thickness:	0.0358	in.
Total Height:	1.016	in.
Radius:	0.1429	in.
θ:	46.39	deg
θ:	0.810	rad
Curve I'_x:	0.000021	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.330	0.998
Corners	0.116	0.146
Bottom Flange	0.786	0.998
Web	1.244	0.508
Top Flange	0.786	0.018

Guess ȳ: 0.506 in.

Stress in Flange:	49.702	ksi
k:	4	
Fcr:	221.414	ksi
λ:	0.474	
ρ:	1.131	
Effective Width:	0.786	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	0.660	0.998	0.659	0.657	--
Bottom Corner	18	2.083	0.870	1.812	1.577	0.000
Web	18	22.386	0.508	11.370	5.775	1.513
Top Corner	18	2.083	0.146	0.303	0.044	0.000
Top Flange	9	7.071	0.018	0.127	0.002	--
Bottom Flange	8	6.286	0.998	6.272	6.259	--
	Σ	40.568		20.543	14.314	1.513

Solved ȳ =	ΣLy/ΣL =	0.506	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.509	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	0.194	$in.^4$
S_e =	$I_x/ȳ$ =	0.381	$in.^3$
Mn =	**Se*Fy =**	**19.06**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1F	
Gage:	20	GA
Strength:	60	ksi
Thickness:	0.0358	in.
Total Height:	1.016	in.
Radius:	0.1429	in.
θ:	46.39	deg
θ:	0.810	rad
Curve I'_x:	0.000021	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.330	0.998
Corners	0.116	0.146
Bottom Flange	0.786	0.998
Web	1.244	0.508
Top Flange	0.786	0.018

Guess ȳ: 0.506 in.

Stress in Flange:	59.642	ksi
k:	4	
Fcr:	221.414	ksi
λ:	0.519	
ρ:	1.110	
Effective Width:	0.786	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	0.660	0.998	0.659	0.657	--
Bottom Corner	18	2.083	0.870	1.812	1.577	0.000
Web	18	22.386	0.508	11.370	5.775	1.513
Top Corner	18	2.083	0.146	0.303	0.044	0.000
Top Flange	9	7.071	0.018	0.127	0.002	--
Bottom Flange	8	6.286	0.998	6.272	6.259	--
	Σ	40.568		20.543	14.314	1.513

Solved ȳ =	ΣLy/ΣL =	0.506	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.509	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	0.194	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.381	$in.^3$
Mn =	**Se*Fy =**	**22.88**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1F	
Gage:	22	GA
Strength:	33	ksi
Thickness:	0.0295	in.
Total Height:	1.010	in.
Radius:	0.13975	in.
θ:	46.39	deg
θ:	0.810	rad
Curve I'_x:	0.000019	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.330	0.995
Corners	0.113	0.140
Bottom Flange	0.786	0.995
Web	1.244	0.505
Top Flange	0.786	0.015

Guess ȳ: 0.503 in.

Stress in Flange:	32.802	ksi
k:	4	
Fcr:	150.343	ksi
λ:	0.467	
ρ:	1.133	
Effective Width:	0.786	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	0.660	0.995	0.657	0.653	--
Bottom Corner	18	2.037	0.870	1.771	1.541	0.000
Web	18	22.386	0.505	11.299	5.703	1.513
Top Corner	18	2.037	0.140	0.285	0.040	0.000
Top Flange	9	7.071	0.015	0.104	0.002	--
Bottom Flange	8	6.286	0.995	6.253	6.220	--
	Σ	40.476		20.369	14.158	1.513

Solved ȳ =	ΣLy/ΣL =	0.503	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.506	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	0.160	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.316	$in.^3$
Mn =	**Se*Fy =**	**10.42**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1F	
Gage:	22	GA
Strength:	40	ksi
Thickness:	0.0295	in.
Total Height:	1.010	in.
Radius:	0.13975	in.
θ:	46.39	deg
θ:	0.810	rad
Curve I'_x:	0.000019	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.330	0.995
Corners	0.113	0.140
Bottom Flange	0.786	0.995
Web	1.244	0.505
Top Flange	0.786	0.015

Guess ȳ: 0.503 in.

Stress in Flange:	39.760	ksi
k:	4	
Fcr:	150.343	ksi
λ:	0.514	
ρ:	1.113	
Effective Width:	0.786	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	0.660	0.995	0.657	0.653	--
Bottom Corner	18	2.037	0.870	1.771	1.541	0.000
Web	18	22.386	0.505	11.299	5.703	1.513
Top Corner	18	2.037	0.140	0.285	0.040	0.000
Top Flange	9	7.071	0.015	0.104	0.002	--
Bottom Flange	8	6.286	0.995	6.253	6.220	--
	Σ	40.476		20.369	14.158	1.513

Solved ȳ =	ΣLy/ΣL =	0.503	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.506	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	0.160	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.316	$in.^3$
Mn =	**Se*Fy =**	**12.64**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1F	
Gage:	22	GA
Strength:	50	ksi
Thickness:	0.0295	in.
Total Height:	1.010	in.
Radius:	0.13975	in.
θ:	46.39	deg
θ:	0.810	rad
Curve I'_x:	0.000019	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.330	0.995
Corners	0.113	0.140
Bottom Flange	0.786	0.995
Web	1.244	0.505
Top Flange	0.786	0.015

Guess ȳ: 0.503 in.

Stress in Flange:	49.699	ksi
k:	4	
Fcr:	150.343	ksi
λ:	0.575	
ρ:	1.074	
Effective Width:	0.786	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	ΣLy^2 ($in.^3$)	$\Sigma I'_x$ ($in.^3$)
Lip	2	0.660	0.995	0.657	0.653	--
Bottom Corner	18	2.037	0.870	1.771	1.541	0.000
Web	18	22.386	0.505	11.299	5.703	1.513
Top Corner	18	2.037	0.140	0.285	0.040	0.000
Top Flange	9	7.071	0.015	0.104	0.002	--
Bottom Flange	8	6.286	0.995	6.253	6.220	--
	Σ	40.476		20.369	14.158	1.513

Solved ȳ =	ΣLy/ΣL =	0.503	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.506	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2 \Sigma L]t$ =	0.160	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.316	$in.^3$
Mn =	**Se*Fy =**	**15.80**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1F	
Gage:	22	GA
Strength:	60	ksi
Thickness:	0.0295	in.
Total Height:	1.010	in.
Radius:	0.13975	in.
θ:	46.39	deg
θ:	0.810	rad
Curve I'_x:	0.000019	in.^{3}

Element	L (in.)	y from top (in.)
Lip	0.330	0.995
Corners	0.113	0.140
Bottom Flange	0.786	0.995
Web	1.244	0.505
Top Flange	0.786	0.015

Guess ȳ: 0.503 in.

Stress in Flange:	59.639	ksi
k:	4	
Fcr:	150.343	ksi
λ:	0.630	
ρ:	1.033	
Effective Width:	0.786	in.

Element	Quantity	ΣL	y from top fiber	ΣLy (in.^{2})	ΣLy^2 (in.^{3})	ΣI'$_x$ (in.^{3})
Lip	2	0.660	0.995	0.657	0.653	--
Bottom Corner	18	2.037	0.870	1.771	1.541	0.000
Web	18	22.386	0.505	11.299	5.703	1.513
Top Corner	18	2.037	0.140	0.285	0.040	0.000
Top Flange	9	7.071	0.015	0.104	0.002	--
Bottom Flange	8	6.286	0.995	6.253	6.220	--
	Σ	40.476		20.369	14.158	1.513

Solved ȳ =	ΣLy/ΣL =	0.503	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.506	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2 \Sigma L]t$ =	0.160	in.^{4}
S_e =	$I_x/\bar{y}$ =	0.316	in.^{3}
Mn =	**Se*Fy =**	**18.95**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1F	
Gage:	24	GA
Strength:	33	ksi
Thickness:	0.0238	in.
Total Height:	1.004	in.
Radius:	0.1369	in.
θ:	46.39	deg
θ:	0.810	rad
Curve I'_x:	0.000018	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.330	0.992
Corners	0.111	0.134
Bottom Flange	0.786	0.992
Web	1.244	0.502
Top Flange	0.786	0.012

Guess ȳ: 0.500 in.

Stress in Flange:	32.800	ksi
k:	4	
Fcr:	97.857	ksi
λ:	0.579	
ρ:	1.071	
Effective Width:	0.786	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	0.660	0.992	0.655	0.649	--
Bottom Corner	18	1.995	0.869	1.735	1.508	0.000
Web	18	22.386	0.502	11.235	5.639	1.513
Top Corner	18	1.995	0.134	0.268	0.036	0.000
Top Flange	9	7.071	0.012	0.084	0.001	--
Bottom Flange	8	6.286	0.992	6.235	6.184	--
	Σ	40.393		20.212	14.018	1.513

Solved ȳ =	ΣLy/ΣL =	0.500	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.503	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	0.129	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.256	$in.^3$
Mn =	**Se*Fy =**	**8.45**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1F	
Gage:	24	GA
Strength:	40	ksi
Thickness:	0.0238	in.
Total Height:	1.004	in.
Radius:	0.1369	in.
θ:	46.39	deg
θ:	0.810	rad
Curve I'_x:	0.000018	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.330	0.992
Corners	0.111	0.134
Bottom Flange	0.786	0.992
Web	1.244	0.502
Top Flange	0.786	0.012

Guess ȳ: 0.500 in.

Stress in Flange:	39.758	ksi
k:	4	
Fcr:	97.857	ksi
λ:	0.637	
ρ:	1.027	
Effective Width:	0.786	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	ΣLy^2 ($in.^3$)	$\Sigma I'_x$ ($in.^3$)
Lip	2	0.660	0.992	0.655	0.649	--
Bottom Corner	18	1.995	0.869	1.735	1.508	0.000
Web	18	22.386	0.502	11.235	5.639	1.513
Top Corner	18	1.995	0.134	0.268	0.036	0.000
Top Flange	9	7.071	0.012	0.084	0.001	--
Bottom Flange	8	6.286	0.992	6.235	6.184	--
	Σ	40.393		20.212	14.018	1.513

Solved ȳ =	ΣLy/ΣL =	0.500	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.503	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2 \Sigma L]t$ =	0.129	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.256	$in.^3$
Mn =	**Se*Fy =**	**10.25**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1F	
Gage:	24	GA
Strength:	50	ksi
Thickness:	0.0238	in.
Total Height:	1.004	in.
Radius:	0.1369	in.
θ:	46.39	deg
θ:	0.810	rad
Curve I'_x:	0.000018	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.330	0.992
Corners	0.111	0.134
Bottom Flange	0.786	0.992
Web	1.244	0.502
Top Flange	0.786	0.012

Guess ȳ: 0.503 in.

Stress in Flange:	50.000	ksi
k:	4	
Fcr:	97.857	ksi
λ:	0.715	
ρ:	0.968	
Effective Width:	0.761	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	0.660	0.992	0.655	0.649	--
Bottom Corner	18	1.995	0.869	1.735	1.508	0.000
Web	18	22.386	0.502	11.235	5.639	1.513
Top Corner	18	1.995	0.134	0.268	0.036	0.000
Top Flange	9	6.848	0.012	0.081	0.001	--
Bottom Flange	8	6.286	0.992	6.235	6.184	--
	Σ	40.170		20.209	14.018	1.513

Solved ȳ =	ΣLy/ΣL =	0.503	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.503	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	0.128	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.254	$in.^3$
Mn =	**Se*Fy =**	**12.69**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1F	
Gage:	24	GA
Strength:	60	ksi
Thickness:	0.0238	in.
Total Height:	1.004	in.
Radius:	0.1369	in.
θ:	46.39	deg
θ:	0.810	rad
Curve I'_x:	0.000018	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.330	0.992
Corners	0.111	0.134
Bottom Flange	0.786	0.992
Web	1.244	0.502
Top Flange	0.786	0.012

Guess ȳ: 0.507 in.

Stress in Flange:	60.000	ksi
k:	4	
Fcr:	97.857	ksi
λ:	0.783	
ρ:	0.918	
Effective Width:	0.721	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	0.660	0.992	0.655	0.649	--
Bottom Corner	18	1.995	0.869	1.735	1.508	0.000
Web	18	22.386	0.502	11.235	5.639	1.513
Top Corner	18	1.995	0.134	0.268	0.036	0.000
Top Flange	9	6.493	0.012	0.077	0.001	--
Bottom Flange	8	6.286	0.992	6.235	6.184	--
	Σ	39.815		20.205	14.018	1.513

Solved ȳ =	ΣLy/ΣL =	0.507	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.507	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2 ΣL]t$ =	0.126	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.248	$in.^3$
Mn =	**Se*Fy =**	**14.85**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1F	
Gage:	26	GA
Strength:	33	ksi
Thickness:	0.0179	in.
Total Height:	0.998	in.
Radius:	0.13395	in.
θ:	46.39	deg
θ:	0.810	rad
Curve I'_x:	0.000017	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.330	0.989
Corners	0.108	0.129
Bottom Flange	0.786	0.989
Web	1.244	0.499
Top Flange	0.786	0.009

Guess ȳ: 0.504 in.

Stress in Flange:	33.000	ksi
k:	4	
Fcr:	55.353	ksi
λ:	0.772	
ρ:	0.926	
Effective Width:	0.728	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	0.660	0.989	0.653	0.645	--
Bottom Corner	18	1.952	0.869	1.697	1.475	0.000
Web	18	22.386	0.499	11.169	5.573	1.513
Top Corner	18	1.952	0.129	0.251	0.032	0.000
Top Flange	9	6.549	0.009	0.059	0.001	--
Bottom Flange	8	6.286	0.989	6.216	6.148	--
	Σ	39.784		20.045	13.874	1.513

Solved ȳ =	ΣLy/ΣL =	0.504	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.504	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2 \Sigma L]t$ =	0.095	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.188	$in.^3$
Mn =	**Se*Fy =**	**6.20**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1F	
Gage:	26	GA
Strength:	40	ksi
Thickness:	0.0179	in.
Total Height:	0.998	in.
Radius:	0.13395	in.
θ:	46.39	deg
θ:	0.810	rad
Curve I'_x:	0.000017	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.330	0.989
Corners	0.108	0.129
Bottom Flange	0.786	0.989
Web	1.244	0.499
Top Flange	0.786	0.009

Guess ȳ: 0.509 in.

Stress in Flange:	40.000	ksi
k:	4	
Fcr:	55.353	ksi
λ:	0.850	
ρ:	0.872	
Effective Width:	0.685	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	ΣLy^2 ($in.^3$)	$\Sigma I'_x$ ($in.^3$)
Lip	2	0.660	0.989	0.653	0.645	--
Bottom Corner	18	1.952	0.869	1.697	1.475	0.000
Web	18	22.386	0.499	11.169	5.573	1.513
Top Corner	18	1.952	0.129	0.251	0.032	0.000
Top Flange	9	6.166	0.009	0.055	0.000	--
Bottom Flange	8	6.286	0.989	6.216	6.148	--
	Σ	39.401		20.041	13.874	1.513

Solved ȳ =	ΣLy/ΣL =	0.509	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.509	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2\Sigma L]t$ =	0.093	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.183	$in.^3$
Mn =	**Se*Fy =**	**7.31**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1F	
Gage:	26	GA
Strength:	50	ksi
Thickness:	0.0179	in.
Total Height:	0.998	in.
Radius:	0.13395	in.
θ:	46.39	deg
θ:	0.810	rad
Curve I'_x:	0.000017	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.330	0.989
Corners	0.108	0.129
Bottom Flange	0.786	0.989
Web	1.244	0.499
Top Flange	0.786	0.009

Guess ȳ: 0.514 in.

Stress in Flange:	50.000	ksi
k:	4	
Fcr:	55.353	ksi
λ:	0.950	
ρ:	0.809	
Effective Width:	0.635	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	0.660	0.989	0.653	0.645	--
Bottom Corner	18	1.952	0.869	1.697	1.475	0.000
Web	18	22.386	0.499	11.169	5.573	1.513
Top Corner	18	1.952	0.129	0.251	0.032	0.000
Top Flange	9	5.718	0.009	0.051	0.000	--
Bottom Flange	8	6.286	0.989	6.216	6.148	--
	Σ	38.954		20.037	13.873	1.513

Solved ȳ =	ΣLy/ΣL =	0.514	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.514	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	0.091	$in.^4$
S_e =	I_x/ȳ =	0.177	$in.^3$
Mn =	**Se*Fy =**	**8.84**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1F	
Gage:	26	GA
Strength:	60	ksi
Thickness:	0.0179	in.
Total Height:	0.998	in.
Radius:	0.13395	in.
θ:	46.39	deg
θ:	0.810	rad
Curve I'_x:	0.000017	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.330	0.989
Corners	0.108	0.129
Bottom Flange	0.786	0.989
Web	1.244	0.499
Top Flange	0.786	0.009

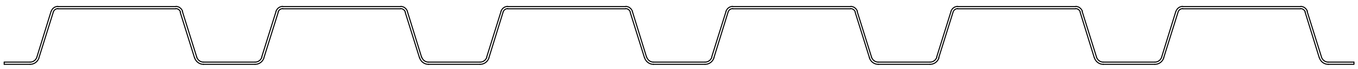
Guess ȳ: 0.519 in.

Stress in Flange:	60.000	ksi
k:	4	
Fcr:	55.353	ksi
λ:	1.041	
ρ:	0.758	
Effective Width:	0.595	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	0.660	0.989	0.653	0.645	--
Bottom Corner	18	1.952	0.869	1.697	1.475	0.000
Web	18	22.386	0.499	11.169	5.573	1.513
Top Corner	18	1.952	0.129	0.251	0.032	0.000
Top Flange	9	5.357	0.009	0.048	0.000	--
Bottom Flange	8	6.286	0.989	6.216	6.148	--
	Σ	38.592		20.034	13.873	1.513

Solved ȳ =	ΣLy/ΣL =	0.519	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.519	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	0.089	$in.^4$
S_e =	$I_x/ȳ$ =	0.172	$in.^3$
Mn =	**Se*Fy =**	**10.32**	**k-in.**

CHAPTER 4: ANALYSIS | 1.5B DECK | + BENDING



4.0 Executive Summary

The Direct Strength Method predicted lower strengths for the majority of the 1.5B Deck sections undergoing positive flexure. It appears that DSM predicts lower strengths for sections with thin and wide compression elements. DSM predicted higher strengths than the EWM for 16GA at each yield stress and 18GA at 33 KSI and 40 KSI. DSM predicted lower strengths for all other sections of 1.5B Deck analyzed for positive flexure in this study. The nominal moment capacity ratio (Mn_{DSM}/Mn_{EWM}) ranged between 0.756 and 1.072.

4.1 Mn_{DSM} / Mn_{EWM} vs. Thickness Plot

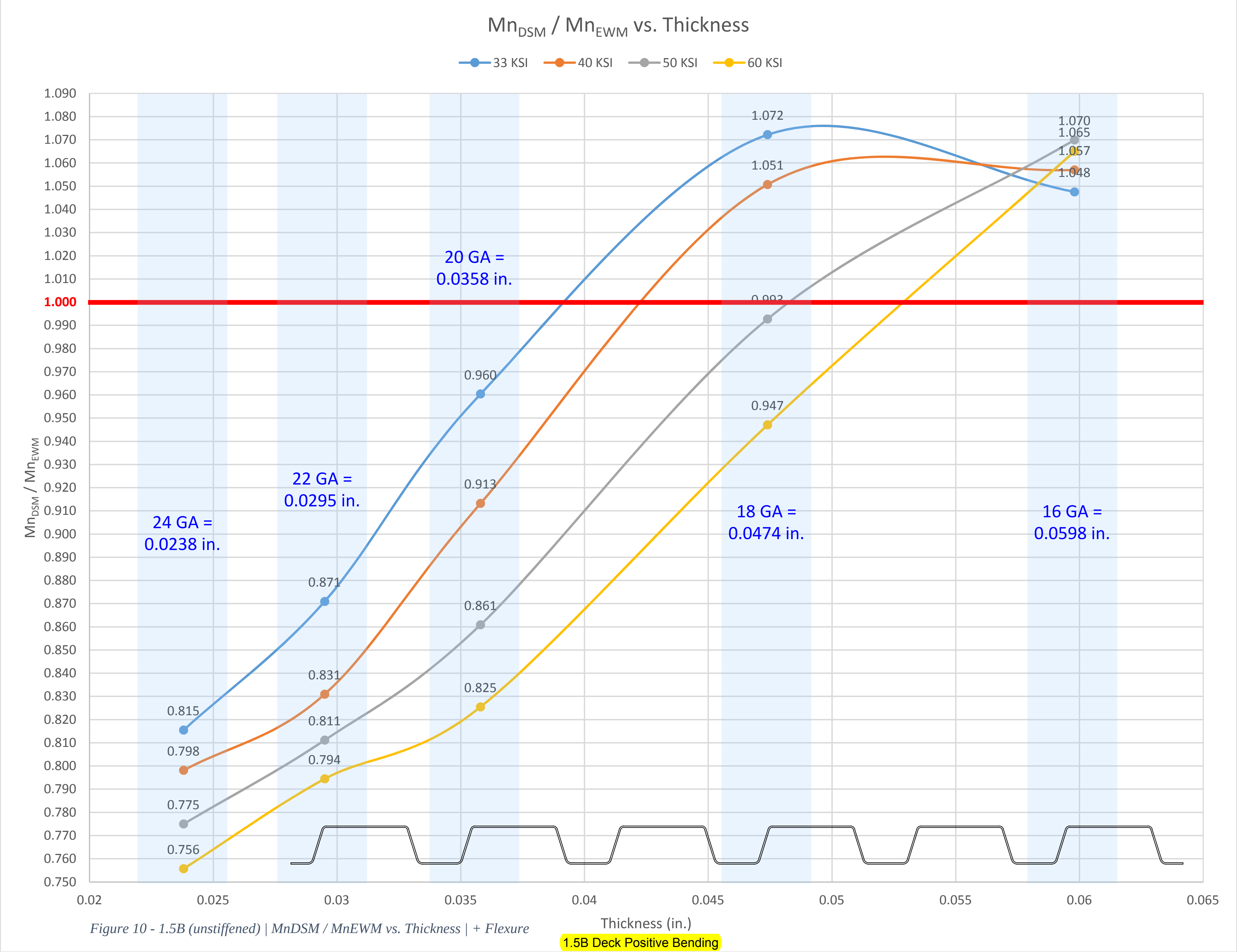


Figure 10 - 1.5B (unstiffened) | MnDSM / MnEWM vs. Thickness | + Flexure

4.2 Analysis Results Summary

Table 4 - 1.5B (unstiffened) | Analysis Results Summary | + Flexure

1.5 B DECK - 33 KSI					
Gage	**24**	**22**	**20**	**18**	**16**
Thickness	0.0238	0.0295	0.0358	0.0474	0.0598
CL Radius	0.1999	0.2028	0.2059	0.2117	0.2179
I_G	0.4345	0.5389	0.6543	0.8674	1.0957
$\bar{y}_G$	0.9127	0.9150	0.9176	0.9223	0.9273
S_{XX}	0.4761	0.5889	0.7131	0.9405	1.1816
My	15.71	19.43	23.53	31.04	38.99
Mn_{DSM}	10.43	14.85	20.36	31.04	38.99
Mn_{EWM}	12.79	17.05	21.2	28.95	37.22
% ERROR	-22.627%	-14.815%	-4.126%	6.733%	4.540%

1.5 B DECK - 33 KSI				
Thickness		Mn_{DSM}	Mn_{EWM}	Mn_{DSM} / Mn_{EWM}
24	0.0238	10.430	12.79	0.815
22	0.0295	14.850	17.05	0.871
20	0.0358	20.360	21.2	0.960
18	0.0474	31.040	28.95	1.072
16	0.0598	38.990	37.22	1.048

1.5 B DECK - 40 KSI					
Gage	**24**	**22**	**20**	**18**	**16**
Thickness	0.0238	0.0295	0.0358	0.0474	0.0598
CL Radius	0.1999	0.2028	0.2059	0.2117	0.2179
I_G	0.4345	0.5389	0.6543	0.8674	1.0957
$\bar{y}_G$	0.9127	0.9150	0.9176	0.9223	0.9273
S_{XX}	0.4761	0.5889	0.7131	0.9405	1.1816
My	19.04	23.56	28.53	37.62	47.26
Mn_{DSM}	11.82	16.86	23.16	36.45	47.26
Mn_{EWM}	14.81	20.29	25.36	34.69	44.71
% ERROR	-25.296%	-20.344%	-9.499%	4.829%	5.396%

1.5 B DECK - 40 KSI				
Thickness		Mn_{DSM}	Mn_{EWM}	Mn_{DSM} / Mn_{EWM}
24	0.0238	11.820	14.81	0.798
22	0.0295	16.860	20.29	0.831
20	0.0358	23.160	25.36	0.913
18	0.0474	36.450	34.69	1.051
16	0.0598	47.260	44.71	1.057

1.5 B DECK - 50 KSI					
Gage	**24**	**22**	**20**	**18**	**16**
Thickness	0.0238	0.0295	0.0358	0.0474	0.0598
CL Radius	0.1999	0.2028	0.2059	0.2117	0.2179
I_G	0.4345	0.5389	0.6543	0.8674	1.0957
$\bar{y}_G$	0.9127	0.9150	0.9176	0.9223	0.9273
S_{XX}	0.4761	0.5889	0.7131	0.9405	1.1816
My	23.80	29.45	35.66	47.02	59.08
Mn_{DSM}	13.64	19.50	26.86	42.43	59.08
Mn_{EWM}	17.6	24.04	31.2	42.74	55.22
% ERROR	-29.032%	-23.282%	-16.158%	-0.731%	6.534%

1.5 B DECK - 50 KSI				
Thickness		Mn_{DSM}	Mn_{EWM}	Mn_{DSM} / Mn_{EWM}
24	0.0238	13.640	17.6	0.775
22	0.0295	19.500	24.04	0.811
20	0.0358	26.860	31.2	0.861
18	0.0474	42.430	42.74	0.993
16	0.0598	59.080	55.22	1.070

1.5 B DECK - 60 KSI					
Gage	**24**	**22**	**20**	**18**	**16**
Thickness	0.0238	0.0295	0.0358	0.0474	0.0598
CL Radius	0.1999	0.2028	0.2059	0.2117	0.2179
I_G	0.4345	0.5389	0.6543	0.8674	1.0957
$\bar{y}_G$	0.9127	0.9150	0.9176	0.9223	0.9273
S_{XX}	0.4761	0.5889	0.7131	0.9405	1.1816
My	28.57	35.34	42.79	56.43	70.90
Mn_{DSM}	15.34	21.95	30.27	47.98	69.81
Mn_{EWM}	20.3	27.63	36.67	50.66	65.55
% ERROR	-32.334%	-25.877%	-21.143%	-5.586%	6.102%

1.5 B DECK - 60 KSI				
Thickness		Mn_{DSM}	Mn_{EWM}	Mn_{DSM} / Mn_{EWM}
24	0.0238	15.340	20.3	0.756
22	0.0295	21.950	27.63	0.794
20	0.0358	30.270	36.67	0.825
18	0.0474	47.980	50.66	0.947
16	0.0598	69.810	65.55	1.065

4.3 Direct Strength Method Calculations

Date:	11/25/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B					
	Gage:	16	GA				
	Strength:	33	KSI				
	M_y =	38.99	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	2.87320	$M_{cr\ell}$ =	112.02607	kip-in	2	in
dist.	M_{crd}/M_y =	10.00000	M_{crd} =	389.9	kip-in	-	in
global	M_{cre}/M_y =	10.00000	M_{cre} =	389.9	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1-\frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **38.99** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1-0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.59 (local-global slenderness)

$M_{n\ell}$ = **38.99** kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1-0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5}M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.32 (distortional slenderness)

M_{nd} = **38.99** kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 38.99 kip-in (local-global controls)

Date:	11/25/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B					
	Gage:	16	GA				
	Strength:	40	KSI				
	M_y =	47.26	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	2.37040	$M_{cr\ell}$ =	112.0251	kip-in	2	in
dist.	M_{crd}/M_y =	10.00000	M_{crd} =	472.6	kip-in	-	in
global	M_{cre}/M_y =	10.00000	M_{cre} =	472.6	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1-\frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 47.26 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell}=M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell}=\left(1-0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ $(Eq.\ 1.2.2\text{-}7)$

λ_ℓ = 0.65 (local-global slenderness)

$M_{n\ell}$ = 47.26 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1-0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5}M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ $(Eq.\ 1.2.2\text{-}10)$

λ_d = 0.32 (distortional slenderness)

M_{nd} = 47.26 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 47.26 kip-in (local-global controls)

Date:	11/25/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B					
	Gage:	16	GA				
	Strength:	50	KSI				
	M_y =	59.08	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	1.89630	$M_{cr\ell}$ =	112.0334	kip-in	2	in
dist.	M_{crd}/M_y =	10.00000	M_{crd} =	590.8	kip-in	-	in
global	M_{cre}/M_y =	10.00000	M_{cre} =	590.8	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1-\frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 59.08 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell}=M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell}=\left(1-0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.73 (local-global slenderness)

$M_{n\ell}$ = 59.08 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1-0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5}M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.32 (distortional slenderness)

M_{nd} = 59.08 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 59.08 kip-in (local-global controls)

Date: 11/25/14 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B					
	Gage:	16	GA				
	Strength:	60	KSI				
	M_y =	70.90	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	1.58030	$M_{cr\ell}$ =	112.04327	kip-in	2	in
dist.	M_{crd}/M_y =	10.00000	M_{crd} =	709	kip-in	-	in
global	M_{cre}/M_y =	10.00000	M_{cre} =	709	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 70.90 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.80 (local-global slenderness)

$M_{n\ell}$ = 69.81 kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.32 (distortional slenderness)

M_{nd} = 70.90 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 69.81 kip-in (local-global controls)

Date:	11/25/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B						
	Gage:	18	GA					
	Strength:	33	KSI					
	M_y =	31.04	kip-in				Length:	
local	$M_{cr\ell}/M_y$ =	1.82000		$M_{cr\ell}$ =	56.4928	kip-in	2	in
dist.	M_{crd}/M_y =	10.00000		M_{crd} =	310.4	kip-in	-	in
global	M_{cre}/M_y =	10.00000		M_{cre} =	310.4	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **31.04** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ $\qquad (Eq.\ 1.2.2\text{-}7)$

λ_ℓ = 0.74 (local-global slenderness)

$M_{n\ell}$ = **31.04** kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ $\qquad (Eq.\ 1.2.2\text{-}10)$

λ_d = 0.32 (distortional slenderness)

M_{nd} = **31.04** kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 31.04 kip-in (local-global controls)

Date:	11/25/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:								
	Deck:	1.5B						
	Gage:	18	GA					
	Strength:	40	KSI					
	M_y =	37.62	kip-in				Length:	
local	$M_{cr\ell}/M_y$ =	1.50150	$M_{cr\ell}$ =	56.48643	kip-in		2	in
dist.	M_{crd}/M_y =	10.00000	M_{crd} =	376.2	kip-in		-	in
global	M_{cre}/M_y =	10.00000	M_{cre} =	376.2	kip-in		-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **37.62** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.82 (local-global slenderness)

$M_{n\ell}$ = **36.45** kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.32 (distortional slenderness)

M_{nd} = **37.62** kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 36.45 kip-in (local-global controls)

Date: 11/25/14 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B				
	Gage:	18 GA				
	Strength:	50 KSI				
	M_y =	47.02	kip-in			Length:
local	$M_{cr\ell}/M_y$ =	1.20120	$M_{cr\ell}$ =	56.480424	kip-in	2 in
dist.	M_{crd}/M_y =	10.00000	M_{crd} =	470.2	kip-in	- in
global	M_{cre}/M_y =	10.00000	M_{cre} =	470.2	kip-in	- in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 47.02 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.91 (local-global slenderness)

$M_{n\ell}$ = 42.43 kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.32 (distortional slenderness)

M_{nd} = 47.02 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 42.43 kip-in (local-global controls)

Date: 11/25/14 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B				
	Gage:	18 GA				
	Strength:	60 KSI				
	$M_y =$	56.43	kip-in		Length:	
local	$M_{cr\ell}/M_y =$	1.00100	$M_{cr\ell} =$	56.48643 kip-in	2	in
dist.	$M_{crd}/M_y =$	10.00000	$M_{crd} =$	564.3 kip-in	-	in
global	$M_{cre}/M_y =$	10.00000	$M_{cre} =$	564.3 kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **56.43** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (*Eq.* 1.2.2-7)

λ_ℓ = 1.00 (local-global slenderness)

$M_{n\ell}$ = **47.98** kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (*Eq.* 1.2.2-10)

λ_d = 0.32 (distortional slenderness)

M_{nd} = **56.43** kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 47.98 kip-in (local-global controls)

Date: 11/25/14 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:						
	Deck:	1.5B				
	Gage:	20 GA				
	Strength:	33 KSI				
	M_y =	23.53	kip-in		Length:	
local	$M_{cr\ell}/M_y$ =	1.05570	$M_{cr\ell}$ =	24.840621 kip-in	2	in
dist.	M_{crd}/M_y =	10.00000	M_{crd} =	235.3 kip-in	-	in
global	M_{cre}/M_y =	10.00000	M_{cre} =	235.3 kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **23.53** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (*Eq.* 1.2.2-7)

λ_ℓ = 0.97 (local-global slenderness)

$M_{n\ell}$ = **20.36** kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (*Eq.* 1.2.2-10)

λ_d = 0.32 (distortional slenderness)

M_{nd} = **23.53** kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 20.36 kip-in (local-global controls)

Date: 11/25/14 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B					
	Gage:	20	GA				
	Strength:	40	KSI				
	M_y =	28.53	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	0.87099	$M_{cr\ell}$ =	24.849345	kip-in	2	in
dist.	M_{crd}/M_y =	10.00000	M_{crd} =	285.3	kip-in	-	in
global	M_{cre}/M_y =	10.00000	M_{cre} =	285.3	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **28.53** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 1.07 (local-global slenderness)

$M_{n\ell}$ = **23.16** kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.32 (distortional slenderness)

M_{nd} = **28.53** kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 23.16 kip-in (local-global controls)

Date: 11/25/14 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:							
	Deck:	1.5B					
	Gage:	20	GA				
	Strength:	50	KSI				
	M_y =	35.66	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	0.69678	$M_{cr\ell}$ =	24.847175	kip-in	2	in
dist.	M_{crd}/M_y =	10.00000	M_{crd} =	356.6	kip-in	-	in
global	M_{cre}/M_y =	10.00000	M_{cre} =	356.6	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1-\frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 35.66 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1-0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 1.20 (local-global slenderness)

$M_{n\ell}$ = 26.86 kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1-0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5}M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.32 (distortional slenderness)

M_{nd} = 35.66 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 26.86 kip-in (local-global controls)

Date:	11/25/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:							
	Deck:	1.5B					
	Gage:	20	GA				
	Strength:	60	KSI				
	M_y =	42.79	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	0.58066	$M_{cr\ell}$ =	24.846441	kip-in	2	in
dist.	M_{crd}/M_y =	10.00000	M_{crd} =	427.9	kip-in	-	in
global	M_{cre}/M_y =	10.00000	M_{cre} =	427.9	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 42.79 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 1.31 (local-global slenderness)

$M_{n\ell}$ = 30.27 kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.32 (distortional slenderness)

M_{nd} = 42.79 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 30.27 kip-in (local-global controls)

Date:	11/25/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:						
	Deck:	1.5B				
	Gage:	22 GA				
	Strength:	33 KSI				
	M_y =	19.43	kip-in		Length:	
local	$M_{cr\ell}/M_y$ =	0.72750	$M_{cr\ell}$ =	14.135325 kip-in	2	in
dist.	M_{crd}/M_y =	10.00000	M_{crd} =	194.3 kip-in	-	in
global	M_{cre}/M_y =	10.00000	M_{cre} =	194.3 kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **19.43** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 1.17 (local-global slenderness)

$M_{n\ell}$ = **14.85** kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.32 (distortional slenderness)

M_{nd} = **19.43** kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 14.85 kip-in (local-global controls)

Date:	11/25/14		calc by:	RKD
			check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B					
	Gage:	22 GA					
	Strength:	40 KSI					
	M_y =	23.56	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	0.60020	$M_{cr\ell}$ =	14.140712	kip-in	2	in
dist.	M_{crd}/M_y =	10.00000	M_{crd} =	235.6	kip-in	-	in
global	M_{cre}/M_y =	10.00000	M_{cre} =	235.6	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **23.56** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (*Eq.* 1.2.2-7)

λ_ℓ = 1.29 (local-global slenderness)

$M_{n\ell}$ = **16.86** kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (*Eq.* 1.2.2-10)

λ_d = 0.32 (distortional slenderness)

M_{nd} = **23.56** kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 16.86 kip-in (local-global controls)

Date:	11/25/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B					
	Gage:	22	GA				
	Strength:	50	KSI				
	M_y =	29.45	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	0.48016	$M_{cr\ell}$ =	14.140712	kip-in	2	in
dist.	M_{crd}/M_y =	10.00000	M_{crd} =	294.5	kip-in	-	in
global	M_{cre}/M_y =	10.00000	M_{cre} =	294.5	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 29.45 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ $\qquad (Eq.\ 1.2.2\text{-}7)$

λ_ℓ = 1.44 (local-global slenderness)

$M_{n\ell}$ = 19.50 kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ $\qquad (Eq.\ 1.2.2\text{-}10)$

λ_d = 0.32 (distortional slenderness)

M_{nd} = 29.45 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 19.50 kip-in (local-global controls)

Date:	11/25/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B					
	Gage:	22	GA				
	Strength:	60	KSI				
	M_y =	35.34	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	0.40013	$M_{cr\ell}$ =	14.140594	kip-in	2	in
dist.	M_{crd}/M_y =	10.00000	M_{crd} =	353.4	kip-in	-	in
global	M_{cre}/M_y =	10.00000	M_{cre} =	353.4	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **35.34** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (*Eq.* 1.2.2-7)

λ_ℓ = 1.58 (local-global slenderness)

$M_{n\ell}$ = **21.95** kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (*Eq.* 1.2.2-10)

λ_d = 0.32 (distortional slenderness)

M_{nd} = **35.34** kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 21.95 kip-in (local-global controls)

Date:	11/25/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:							
	Deck:	1.5B					
	Gage:	24	GA				
	Strength:	33	KSI				
	M_y =	15.71	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	0.48401	$M_{cr\ell}$ =	7.6037971	kip-in	2	in
dist.	M_{crd}/M_y =	10.00000	M_{crd} =	157.1	kip-in	-	in
global	M_{cre}/M_y =	10.00000	M_{cre} =	157.1	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **15.71** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 1.44 (local-global slenderness)

$M_{n\ell}$ = **10.43** kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.32 (distortional slenderness)

M_{nd} = **15.71** kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 10.43 kip-in (local-global controls)

Date:	11/25/14				calc by:	RKD
					check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B				
	Gage:	24	GA			
	Strength:	40	KSI			
	M_y =	19.04	kip-in		Length:	
local	$M_{cr\ell}/M_y$ =	0.39931	$M_{cr\ell}$ =	7.6028624 kip-in	2	in
dist.	M_{crd}/M_y =	10.00000	M_{crd} =	190.4 kip-in	-	in
global	M_{cre}/M_y =	10.00000	M_{cre} =	190.4 kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **19.04** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (*Eq.* 1.2.2-7)

λ_ℓ = 1.58 (local-global slenderness)

$M_{n\ell}$ = **11.82** kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (*Eq.* 1.2.2-10)

λ_d = 0.32 (distortional slenderness)

M_{nd} = **19.04** kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 11.82 kip-in (local-global controls)

Date:	11/25/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B					
	Gage:	24 GA					
	Strength:	50 KSI					
	M_y =	23.80	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	0.31945	$M_{cr\ell}$ =	7.60291	kip-in	2	in
dist.	M_{crd}/M_y =	10.00000	M_{crd} =	238	kip-in	-	in
global	M_{cre}/M_y =	10.00000	M_{cre} =	238	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1-\frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **23.80** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell}=M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell}=\left(1-0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 1.77 (local-global slenderness)

$M_{n\ell}$ = **13.64** kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1-0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5}M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.32 (distortional slenderness)

M_{nd} = **23.80** kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 13.64 kip-in (local-global controls)

Date:	11/25/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:								
	Deck:	1.5B						
	Gage:	24	GA					
	Strength:	60	KSI					
	$M_y =$	28.57	kip-in				Length:	
local	$M_{cr\ell}/M_y =$	0.26621	$M_{cr\ell} =$	7.6056197	kip-in		2	in
dist.	$M_{crd}/M_y =$	10.00000	$M_{crd} =$	285.7	kip-in		-	in
global	$M_{cre}/M_y =$	10.00000	$M_{cre} =$	285.7	kip-in		-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 28.57 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (*Eq.* 1.2.2-7)

λ_ℓ = 1.94 (local-global slenderness)

$M_{n\ell}$ = 15.34 kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (*Eq.* 1.2.2-10)

λ_d = 0.32 (distortional slenderness)

M_{nd} = 28.57 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 15.34 kip-in (local-global controls)

4.4 Effective Width Method Calculations

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	16	GA
Strength:	33	ksi
Thickness:	0.0598	in.
Total Height:	1.540	in.
Radius:	0.2179	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000592	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	1.510
Corners	0.276	0.194
Bottom Flange	1.377	1.510
Web	1.275	0.770
Top Flange	3.144	0.030

Guess ȳ: 0.626 in.

Stress in Flange:	22.588	ksi
k:	4	
Fcr:	38.589	ksi
λ:	0.765	
ρ:	0.931	
Effective Width:	2.928	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.378	1.510	2.080	3.141	--
Bottom Corner	12	3.309	1.346	4.452	5.991	0.007
Web	12	15.296	0.770	11.776	9.066	1.884
Top Corner	12	3.309	0.194	0.642	0.125	0.007
Top Flange	6	17.565	0.030	0.525	0.016	--
Bottom Flange	5	6.884	1.510	10.394	15.694	--
	Σ	47.739		29.870	34.033	1.898

Solved ȳ =	ΣLy/ΣL =	0.626	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.914	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	1.031	$in.^4$
S_e =	$I_x/\bar{y}$ =	1.128	$in.^3$
Mn =	**Se*Fy =**	**37.22**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	16	GA
Strength:	40	ksi
Thickness:	0.0598	in.
Total Height:	1.540	in.
Radius:	0.2179	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000592	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	1.510
Corners	0.276	0.194
Bottom Flange	1.377	1.510
Web	1.275	0.770
Top Flange	3.144	0.030

Guess ȳ: 0.642 in.

Stress in Flange:	28.581	ksi
k:	4	
Fcr:	38.589	ksi
λ:	0.861	
ρ:	0.865	
Effective Width:	2.719	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.378	1.510	2.080	3.141	--
Bottom Corner	12	3.309	1.346	4.452	5.991	0.007
Web	12	15.296	0.770	11.776	9.066	1.884
Top Corner	12	3.309	0.194	0.642	0.125	0.007
Top Flange	6	16.315	0.030	0.488	0.015	--
Bottom Flange	5	6.884	1.510	10.394	15.694	--
	Σ	46.489		29.832	34.031	1.898

Solved ȳ =	ΣLy/ΣL =	0.642	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.898	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	1.004	$in.^4$
S_e =	$I_x/ȳ$ =	1.118	$in.^3$
Mn =	**Se*Fy =**	**44.71**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	16	GA
Strength:	50	ksi
Thickness:	0.0598	in.
Total Height:	1.540	in.
Radius:	0.2179	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000592	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	1.510
Corners	0.276	0.194
Bottom Flange	1.377	1.510
Web	1.275	0.770
Top Flange	3.144	0.030

Guess ȳ: 0.662 in.

Stress in Flange:	37.686	ksi
k:	4	
Fcr:	38.589	ksi
λ:	0.988	
ρ:	0.787	
Effective Width:	2.473	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.378	1.510	2.080	3.141	--
Bottom Corner	12	3.309	1.346	4.452	5.991	0.007
Web	12	15.296	0.770	11.776	9.066	1.884
Top Corner	12	3.309	0.194	0.642	0.125	0.007
Top Flange	6	14.838	0.030	0.444	0.013	--
Bottom Flange	5	6.884	1.510	10.394	15.694	--
	Σ	45.012		29.788	34.030	1.898

Solved ȳ =	ΣLy/ΣL =	0.662	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.878	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	0.970	$in.^4$
S_e =	$I_x/\bar{y}$ =	1.104	$in.^3$
Mn =	**Se*Fy =**	**55.22**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	16	GA
Strength:	60	ksi
Thickness:	0.0598	in.
Total Height:	1.540	in.
Radius:	0.2179	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000592	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	1.510
Corners	0.276	0.194
Bottom Flange	1.377	1.510
Web	1.275	0.770
Top Flange	3.144	0.030

Guess ȳ: 0.679 in.

Stress in Flange:	47.317	ksi
k:	4	
Fcr:	38.589	ksi
λ:	1.107	
ρ:	0.724	
Effective Width:	2.275	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.378	1.510	2.080	3.141	--
Bottom Corner	12	3.309	1.346	4.452	5.991	0.007
Web	12	15.296	0.770	11.776	9.066	1.884
Top Corner	12	3.309	0.194	0.642	0.125	0.007
Top Flange	6	13.650	0.030	0.408	0.012	--
Bottom Flange	5	6.884	1.510	10.394	15.694	--
	Σ	43.824		29.753	34.029	1.898

Solved ȳ =	ΣLy/ΣL =	0.679	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.861	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	0.940	$in.^4$
S_e =	$I_x/\bar{y}$ =	1.092	$in.^3$
Mn =	**Se*Fy =**	**65.55**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	18	GA
Strength:	33	ksi
Thickness:	0.0474	in.
Total Height:	1.527	in.
Radius:	0.2117	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000543	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	1.504
Corners	0.268	0.183
Bottom Flange	1.377	1.504
Web	1.275	0.764
Top Flange	3.144	0.024

Guess ȳ: 0.659 in.

Stress in Flange:	25.070	ksi
k:	4	
Fcr:	24.245	ksi
λ:	1.017	
ρ:	0.771	
Effective Width:	2.423	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.378	1.504	2.072	3.115	--
Bottom Corner	12	3.215	1.344	4.321	5.808	0.007
Web	12	15.296	0.764	11.681	8.921	1.884
Top Corner	12	3.215	0.183	0.589	0.108	0.007
Top Flange	6	14.536	0.024	0.345	0.008	--
Bottom Flange	5	6.884	1.504	10.351	15.565	--
	Σ	44.522		29.358	33.525	1.897

Solved ȳ =	ΣLy/ΣL =	0.659	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.868	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	0.761	$in.^4$
S_e =	$I_x/ȳ$ =	0.877	$in.^3$
Mn =	**Se*Fy =**	**28.95**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	18	GA
Strength:	40	ksi
Thickness:	0.0474	in.
Total Height:	1.527	in.
Radius:	0.2117	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000543	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	1.504
Corners	0.268	0.183
Bottom Flange	1.377	1.504
Web	1.275	0.764
Top Flange	3.144	0.024

Guess ȳ: 0.678 in.

Stress in Flange:	31.905	ksi
k:	4	
Fcr:	24.245	ksi
λ:	1.147	
ρ:	0.705	
Effective Width:	2.215	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.378	1.504	2.072	3.115	--
Bottom Corner	12	3.215	1.344	4.321	5.808	0.007
Web	12	15.296	0.764	11.681	8.921	1.884
Top Corner	12	3.215	0.183	0.589	0.108	0.007
Top Flange	6	13.290	0.024	0.315	0.007	--
Bottom Flange	5	6.884	1.504	10.351	15.565	--
	Σ	43.276		29.329	33.524	1.897

Solved ȳ =	ΣLy/ΣL =	0.678	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.850	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	0.737	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.867	$in.^3$
Mn =	**Se*Fy =**	**34.69**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	18	GA
Strength:	50	ksi
Thickness:	0.0474	in.
Total Height:	1.527	in.
Radius:	0.2117	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000543	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	1.504
Corners	0.268	0.183
Bottom Flange	1.377	1.504
Web	1.275	0.764
Top Flange	3.144	0.024

Guess ȳ: 0.699 in.

Stress in Flange:	42.216	ksi
k:	4	
Fcr:	24.245	ksi
λ:	1.320	
ρ:	0.631	
Effective Width:	1.985	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	ΣLy^2 ($in.^3$)	$\Sigma I'_x$ ($in.^3$)
Lip	2	1.378	1.504	2.072	3.115	--
Bottom Corner	12	3.215	1.344	4.321	5.808	0.007
Web	12	15.296	0.764	11.681	8.921	1.884
Top Corner	12	3.215	0.183	0.589	0.108	0.007
Top Flange	6	11.911	0.024	0.282	0.007	--
Bottom Flange	5	6.884	1.504	10.351	15.565	--
	Σ	41.897		29.296	33.523	1.897

Solved ȳ =	ΣLy/ΣL =	0.699	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.828	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2 \Sigma L]t$ =	0.708	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.855	$in.^3$
Mn =	**Se*Fy =**	**42.74**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	18	GA
Strength:	60	ksi
Thickness:	0.0474	in.
Total Height:	1.527	in.
Radius:	0.2117	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000543	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	1.504
Corners	0.268	0.183
Bottom Flange	1.377	1.504
Web	1.275	0.764
Top Flange	3.144	0.024

Guess ȳ: 0.717 in.

Stress in Flange:	53.042	ksi
k:	4	
Fcr:	24.245	ksi
λ:	1.479	
ρ:	0.576	
Effective Width:	1.809	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.378	1.504	2.072	3.115	--
Bottom Corner	12	3.215	1.344	4.321	5.808	0.007
Web	12	15.296	0.764	11.681	8.921	1.884
Top Corner	12	3.215	0.183	0.589	0.108	0.007
Top Flange	6	10.856	0.024	0.257	0.006	--
Bottom Flange	5	6.884	1.504	10.351	15.565	--
	Σ	40.842		29.271	33.523	1.897

Solved ȳ =	ΣLy/ΣL =	0.717	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.811	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	0.685	$in.^4$
S_e =	$I_x/ȳ$ =	0.844	$in.^3$
Mn =	**Se*Fy =**	**50.66**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	20	GA
Strength:	33	ksi
Thickness:	0.0358	in.
Total Height:	1.516	in.
Radius:	0.2059	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000500	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	1.498
Corners	0.261	0.173
Bottom Flange	1.377	1.498
Web	1.275	0.758
Top Flange	3.144	0.018

Guess ȳ: 0.707 in.

Stress in Flange:	28.850	ksi
k:	4	
Fcr:	13.830	ksi
λ:	1.444	
ρ:	0.587	
Effective Width:	1.845	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.378	1.498	2.064	3.091	--
Bottom Corner	12	3.126	1.343	4.198	5.637	0.006
Web	12	15.296	0.758	11.592	8.786	1.884
Top Corner	12	3.126	0.173	0.541	0.094	0.006
Top Flange	6	11.071	0.018	0.198	0.004	--
Bottom Flange	5	6.884	1.498	10.311	15.445	--
	Σ	40.880		28.904	33.056	1.896

Solved ȳ =	ΣLy/ΣL =	0.707	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.809	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	0.520	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.643	$in.^3$
Mn =	**Se*Fy =**	**21.20**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	20	GA
Strength:	40	ksi
Thickness:	0.0358	in.
Total Height:	1.516	in.
Radius:	0.2059	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000500	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	1.498
Corners	0.261	0.173
Bottom Flange	1.377	1.498
Web	1.275	0.758
Top Flange	3.144	0.018

Guess ȳ: 0.725 in.

Stress in Flange:	36.702	ksi
k:	4	
Fcr:	13.830	ksi
λ:	1.629	
ρ:	0.531	
Effective Width:	1.669	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.378	1.498	2.064	3.091	--
Bottom Corner	12	3.126	1.343	4.198	5.637	0.006
Web	12	15.296	0.758	11.592	8.786	1.884
Top Corner	12	3.126	0.173	0.541	0.094	0.006
Top Flange	6	10.015	0.018	0.179	0.003	--
Bottom Flange	5	6.884	1.498	10.311	15.445	--
	Σ	39.825		28.886	33.056	1.896

Solved ȳ =	ΣLy/ΣL =	0.725	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.790	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2 ΣL]t$ =	0.501	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.634	$in.^3$
Mn =	**Se*Fy =**	**25.36**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	20	GA
Strength:	50	ksi
Thickness:	0.0358	in.
Total Height:	1.516	in.
Radius:	0.2059	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000500	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	1.498
Corners	0.261	0.173
Bottom Flange	1.377	1.498
Web	1.275	0.758
Top Flange	3.144	0.018

Guess ȳ: 0.746 in.

Stress in Flange:	48.426	ksi
k:	4	
Fcr:	13.830	ksi
λ:	1.871	
ρ:	0.472	
Effective Width:	1.483	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.378	1.498	2.064	3.091	--
Bottom Corner	12	3.126	1.343	4.198	5.637	0.006
Web	12	15.296	0.758	11.592	8.786	1.884
Top Corner	12	3.126	0.173	0.541	0.094	0.006
Top Flange	6	8.895	0.018	0.159	0.003	--
Bottom Flange	5	6.884	1.498	10.311	15.445	--
	Σ	38.705		28.866	33.055	1.896

Solved ȳ =	ΣLy/ΣL =	0.746	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.770	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	0.481	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.624	$in.^3$
Mn =	**Se*Fy =**	**31.20**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	20	GA
Strength:	60	ksi
Thickness:	0.0358	in.
Total Height:	1.516	in.
Radius:	0.2059	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000500	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	1.498
Corners	0.261	0.173
Bottom Flange	1.377	1.498
Web	1.275	0.758
Top Flange	3.144	0.018

Guess ȳ: 0.761 in.

Stress in Flange:	60.000	ksi
k:	4	
Fcr:	13.830	ksi
λ:	2.083	
ρ:	0.429	
Effective Width:	1.350	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.378	1.498	2.064	3.091	--
Bottom Corner	12	3.126	1.343	4.198	5.637	0.006
Web	12	15.296	0.758	11.592	8.786	1.884
Top Corner	12	3.126	0.173	0.541	0.094	0.006
Top Flange	6	8.100	0.018	0.145	0.003	--
Bottom Flange	5	6.884	1.498	10.311	15.445	--
	Σ	37.909		28.851	33.055	1.896

Solved ȳ =	ΣLy/ΣL =	0.761	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.761	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	0.465	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.611	$in.^3$
Mn =	**Se*Fy =**	**36.67**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	22	GA
Strength:	33	ksi
Thickness:	0.0295	in.
Total Height:	1.510	in.
Radius:	0.20275	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000477	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	1.495
Corners	0.257	0.168
Bottom Flange	1.377	1.495
Web	1.275	0.755
Top Flange	3.144	0.015

Guess ȳ: 0.740 in.

Stress in Flange:	31.737	ksi
k:	4	
Fcr:	9.391	ksi
λ:	1.838	
ρ:	0.479	
Effective Width:	1.505	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.378	1.495	2.059	3.078	--
Bottom Corner	12	3.079	1.342	4.131	5.544	0.006
Web	12	15.296	0.755	11.544	8.713	1.884
Top Corner	12	3.079	0.168	0.516	0.086	0.006
Top Flange	6	9.033	0.015	0.133	0.002	--
Bottom Flange	5	6.884	1.495	10.289	15.380	--
	Σ	38.747		28.673	32.804	1.895

Solved ȳ =	ΣLy/ΣL =	0.740	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.769	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	0.398	$in.^4$
S_e =	$I_x/ȳ$ =	0.517	$in.^3$
Mn =	**Se*Fy =**	**17.05**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	22	GA
Strength:	40	ksi
Thickness:	0.0295	in.
Total Height:	1.510	in.
Radius:	0.20275	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000477	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	1.495
Corners	0.257	0.168
Bottom Flange	1.377	1.495
Web	1.275	0.755
Top Flange	3.144	0.015

Guess ȳ: 0.757 in.

Stress in Flange:	40.000	ksi
k:	4	
Fcr:	9.391	ksi
λ:	2.064	
ρ:	0.433	
Effective Width:	1.361	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.378	1.495	2.059	3.078	--
Bottom Corner	12	3.079	1.342	4.131	5.544	0.006
Web	12	15.296	0.755	11.544	8.713	1.884
Top Corner	12	3.079	0.168	0.516	0.086	0.006
Top Flange	6	8.165	0.015	0.120	0.002	--
Bottom Flange	5	6.884	1.495	10.289	15.380	--
	Σ	37.879		28.661	32.804	1.895

Solved ȳ =	ΣLy/ΣL =	0.757	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.757	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	0.384	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.507	$in.^3$
Mn =	**Se*Fy =**	**20.29**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	22	GA
Strength:	50	ksi
Thickness:	0.0295	in.
Total Height:	1.510	in.
Radius:	0.20275	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000477	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	1.495
Corners	0.257	0.168
Bottom Flange	1.377	1.495
Web	1.275	0.755
Top Flange	3.144	0.015

Guess ȳ: 0.772 in.

Stress in Flange:	50.000	ksi
k:	4	
Fcr:	9.391	ksi
λ:	2.307	
ρ:	0.392	
Effective Width:	1.233	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.378	1.495	2.059	3.078	--
Bottom Corner	12	3.079	1.342	4.131	5.544	0.006
Web	12	15.296	0.755	11.544	8.713	1.884
Top Corner	12	3.079	0.168	0.516	0.086	0.006
Top Flange	6	7.395	0.015	0.109	0.002	--
Bottom Flange	5	6.884	1.495	10.289	15.380	--
	Σ	37.109		28.649	32.803	1.895

Solved ȳ =	ΣLy/ΣL =	0.772	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.772	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	0.371	$in.^4$
S_e =	I_x/ȳ =	0.481	$in.^3$
Mn =	**Se*Fy =**	**24.04**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	22	GA
Strength:	60	ksi
Thickness:	0.0295	in.
Total Height:	1.510	in.
Radius:	0.20275	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000477	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	1.495
Corners	0.257	0.168
Bottom Flange	1.377	1.495
Web	1.275	0.755
Top Flange	3.144	0.015

Guess ȳ: 0.784 in.

Stress in Flange:	60.000	ksi
k:	4	
Fcr:	9.391	ksi
λ:	2.528	
ρ:	0.361	
Effective Width:	1.135	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.378	1.495	2.059	3.078	--
Bottom Corner	12	3.079	1.342	4.131	5.544	0.006
Web	12	15.296	0.755	11.544	8.713	1.884
Top Corner	12	3.079	0.168	0.516	0.086	0.006
Top Flange	6	6.813	0.015	0.100	0.001	--
Bottom Flange	5	6.884	1.495	10.289	15.380	--
	Σ	36.527		28.641	32.803	1.895

Solved ȳ =	ΣLy/ΣL =	0.784	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.784	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	0.361	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.461	$in.^3$
Mn =	**Se*Fy =**	**27.63**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	24	GA
Strength:	33	ksi
Thickness:	0.0238	in.
Total Height:	1.504	in.
Radius:	0.1999	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000457	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	1.492
Corners	0.253	0.163
Bottom Flange	1.377	1.492
Web	1.275	0.752
Top Flange	3.144	0.012

Guess ȳ: 0.770 in.

Stress in Flange:	33.000	ksi
k:	4	
Fcr:	6.112	ksi
λ:	2.324	
ρ:	0.390	
Effective Width:	1.225	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.378	1.492	2.055	3.066	--
Bottom Corner	12	3.035	1.341	4.071	5.460	0.005
Web	12	15.296	0.752	11.501	8.647	1.884
Top Corner	12	3.035	0.163	0.493	0.080	0.005
Top Flange	6	7.349	0.012	0.087	0.001	--
Bottom Flange	5	6.884	1.492	10.270	15.322	--
	Σ	36.977		28.478	32.577	1.895

Solved ȳ =	ΣLy/ΣL =	0.770	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.770	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	0.298	$in.^4$
S_e =	I_x/ȳ =	0.387	$in.^3$
Mn =	**Se*Fy =**	**12.79**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	24	GA
Strength:	40	ksi
Thickness:	0.0238	in.
Total Height:	1.504	in.
Radius:	0.1999	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000457	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	1.492
Corners	0.253	0.163
Bottom Flange	1.377	1.492
Web	1.275	0.752
Top Flange	3.144	0.012

Guess ȳ: 0.783 in.

Stress in Flange:	40.000	ksi
k:	4	
Fcr:	6.112	ksi
λ:	2.558	
ρ:	0.357	
Effective Width:	1.123	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.378	1.492	2.055	3.066	--
Bottom Corner	12	3.035	1.341	4.071	5.460	0.005
Web	12	15.296	0.752	11.501	8.647	1.884
Top Corner	12	3.035	0.163	0.493	0.080	0.005
Top Flange	6	6.739	0.012	0.080	0.001	--
Bottom Flange	5	6.884	1.492	10.270	15.322	--
	Σ	36.367		28.471	32.577	1.895

Solved ȳ =	ΣLy/ΣL =	0.783	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.783	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	0.290	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.370	$in.^3$
Mn =	**Se*Fy =**	**14.81**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	24	GA
Strength:	50	ksi
Thickness:	0.0238	in.
Total Height:	1.504	in.
Radius:	0.1999	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000457	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	1.492
Corners	0.253	0.163
Bottom Flange	1.377	1.492
Web	1.275	0.752
Top Flange	3.144	0.012

Guess ȳ: 0.797 in.

Stress in Flange:	50.000	ksi
k:	4	
Fcr:	6.112	ksi
λ:	2.860	
ρ:	0.323	
Effective Width:	1.015	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	ΣLy^2 ($in.^3$)	$\Sigma I'_x$ ($in.^3$)
Lip	2	1.378	1.492	2.055	3.066	--
Bottom Corner	12	3.035	1.341	4.071	5.460	0.005
Web	12	15.296	0.752	11.501	8.647	1.884
Top Corner	12	3.035	0.163	0.493	0.080	0.005
Top Flange	6	6.088	0.012	0.072	0.001	--
Bottom Flange	5	6.884	1.492	10.270	15.322	--
	Σ	35.715		28.463	32.577	1.895

Solved ȳ =	ΣLy/ΣL =	0.797	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.797	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2 \Sigma L]t$ =	0.281	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.352	$in.^3$
Mn =	**Se*Fy =**	**17.60**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	24	GA
Strength:	60	ksi
Thickness:	0.0238	in.
Total Height:	1.504	in.
Radius:	0.1999	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000457	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	1.492
Corners	0.253	0.163
Bottom Flange	1.377	1.492
Web	1.275	0.752
Top Flange	3.144	0.012

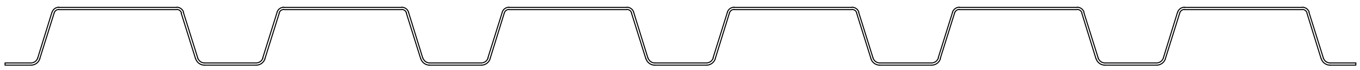
Guess ȳ: 0.808 in.

Stress in Flange:	60.000	ksi
k:	4	
Fcr:	6.112	ksi
λ:	3.133	
ρ:	0.297	
Effective Width:	0.933	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.378	1.492	2.055	3.066	--
Bottom Corner	12	3.035	1.341	4.071	5.460	0.005
Web	12	15.296	0.752	11.501	8.647	1.884
Top Corner	12	3.035	0.163	0.493	0.080	0.005
Top Flange	6	5.598	0.012	0.067	0.001	--
Bottom Flange	5	6.884	1.492	10.270	15.322	--
	Σ	35.225		28.457	32.577	1.895

Solved ȳ =	ΣLy/ΣL =	0.808	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.808	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	0.273	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.338	$in.^3$
Mn =	**Se*Fy =**	**20.30**	**k-in.**

CHAPTER 5: ANALYSIS | 1.5B DECK | - BENDING



5.0 Executive Summary

The Direct Strength Method predicted higher strengths for the majority of the 1.5B Deck sections undergoing negative flexure. The compression flange is the lower flange for this section and DSM is able to take advantage of the shorter compression flange width. DSM predicted lower strengths than the EWM for 24GA at yield stresses of 50 and 60 KSI and for 22GA at 60 KSI. DSM predicted higher strengths for all other sections of 1.5B Deck analyzed for negative flexure in this study. The nominal moment capacity ratio (Mn_{DSM}/Mn_{EWM}) ranged between 0.929 and 1.059.

5.1 Mn_{DSM} / Mn_{EWM} vs. Thickness Plot

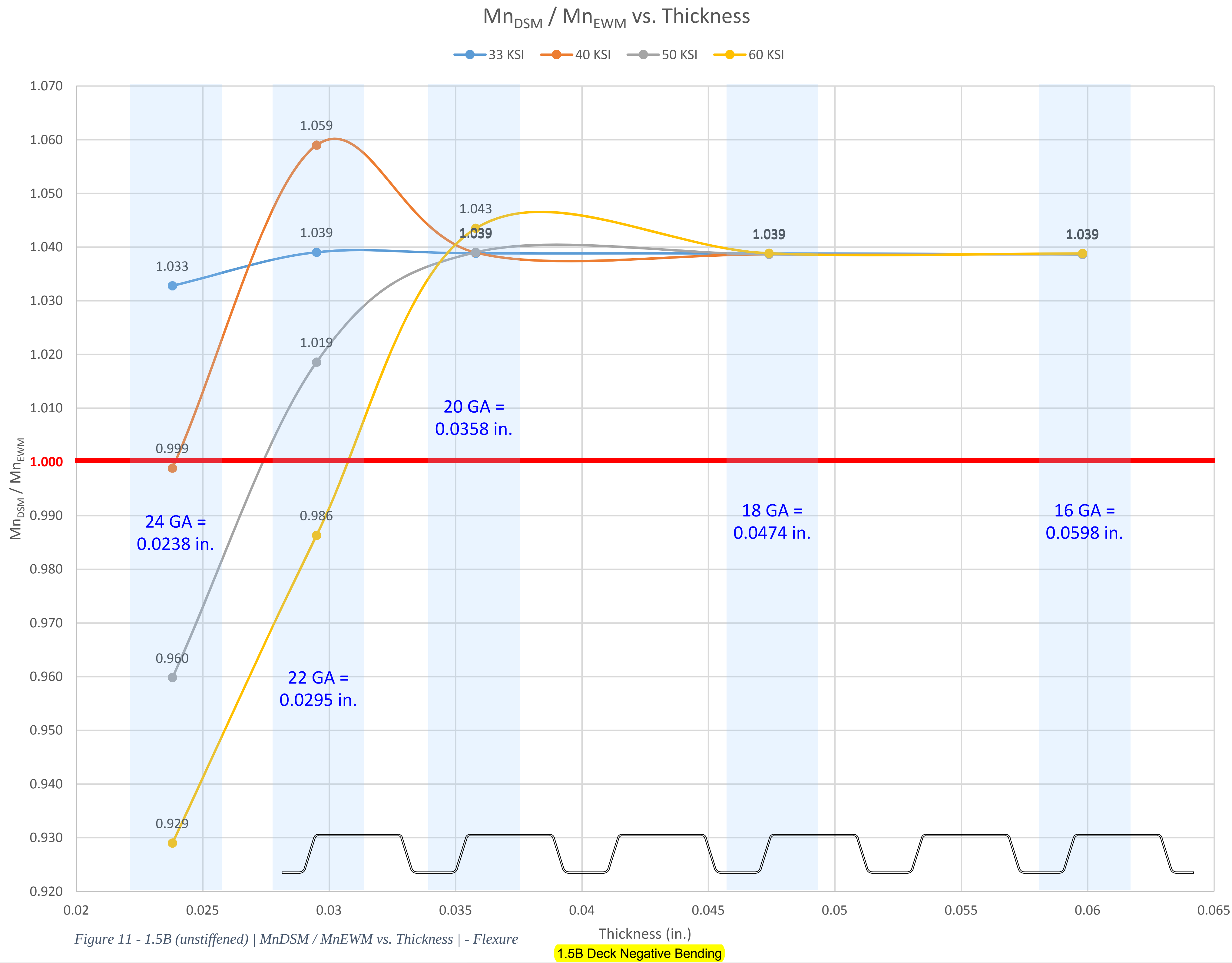


Figure 11 - 1.5B (unstiffened) | MnDSM / MnEWM vs. Thickness | - Flexure

5.2 Analysis Results Summary

Table 5 - 1.5B (unstiffened) | Analysis Results Summary | - Flexure

1.5 B DECK - 33 KSI					
Gage	**24**	**22**	**20**	**18**	**16**
Thickness	0.0238	0.0295	0.0358	0.0474	0.0598
CL Radius	0.1999	0.2028	0.2059	0.2117	0.2179
I_G	0.4345	0.5389	0.6543	0.8674	1.0957
$\bar{y}_G$	0.9127	0.9150	0.9176	0.9223	0.9273
S_{xx}	0.4761	0.5889	0.7131	0.9405	1.1816
My	15.71	19.43	23.53	31.04	38.99
Mn_{DSM}	14.80	19.43	23.53	31.04	38.99
Mn_{EWM}	14.33	18.7	22.65	29.88	37.54
% ERROR	3.176%	3.757%	3.740%	3.737%	3.719%

1.5 B DECK - 33 KSI				
Thickness		Mn_{DSM}	Mn_{EWM}	Mn_{DSM} / Mn_{EWM}
24	0.0238	14.800	14.33	1.033
22	0.0295	19.430	18.7	1.039
20	0.0358	23.530	22.65	1.039
18	0.0474	31.040	29.88	1.039
16	0.0598	38.990	37.54	1.039

1.5 B DECK - 40 KSI					
Gage	**24**	**22**	**20**	**18**	**16**
Thickness	0.0238	0.0295	0.0358	0.0474	0.0598
CL Radius	0.1999	0.2028	0.2059	0.2117	0.2179
I_G	0.4345	0.5389	0.6543	0.8674	1.0957
$\bar{y}_G$	0.9127	0.9150	0.9176	0.9223	0.9273
S_{xx}	0.4761	0.5889	0.7131	0.9405	1.1816
My	19.04	23.56	28.53	37.62	47.26
Mn_{DSM}	16.86	23.52	28.53	37.62	47.26
Mn_{EWM}	16.88	22.21	27.46	36.22	45.5
% ERROR	-0.119%	5.570%	3.750%	3.721%	3.724%

1.5 B DECK - 40 KSI				
Thickness		Mn_{DSM}	Mn_{EWM}	Mn_{DSM} / Mn_{EWM}
24	0.0238	16.860	16.88	0.999
22	0.0295	23.520	22.21	1.059
20	0.0358	28.530	27.46	1.039
18	0.0474	37.620	36.22	1.039
16	0.0598	47.260	45.5	1.039

1.5 B DECK - 50 KSI					
Gage	**24**	**22**	**20**	**18**	**16**
Thickness	0.0238	0.0295	0.0358	0.0474	0.0598
CL Radius	0.1999	0.2028	0.2059	0.2117	0.2179
I_G	0.4345	0.5389	0.6543	0.8674	1.0957
$\bar{y}_G$	0.9127	0.9150	0.9176	0.9223	0.9273
S_{xx}	0.4761	0.5889	0.7131	0.9405	1.1816
My	23.80	29.45	35.66	47.02	59.08
Mn_{DSM}	19.58	27.41	35.66	47.02	59.08
Mn_{EWM}	20.4	26.91	34.32	45.27	56.88
% ERROR	-4.188%	1.824%	3.758%	3.722%	3.724%

1.5 B DECK - 50 KSI				
Thickness		Mn_{DSM}	Mn_{EWM}	Mn_{DSM} / Mn_{EWM}
24	0.0238	19.580	20.4	0.960
22	0.0295	27.410	26.91	1.019
20	0.0358	35.660	34.32	1.039
18	0.0474	47.020	45.27	1.039
16	0.0598	59.080	56.88	1.039

1.5 B DECK - 60 KSI					
Gage	**24**	**22**	**20**	**18**	**16**
Thickness	0.0238	0.0295	0.0358	0.0474	0.0598
CL Radius	0.1999	0.2028	0.2059	0.2117	0.2179
I_G	0.4345	0.5389	0.6543	0.8674	1.0957
$\bar{y}_G$	0.9127	0.9150	0.9176	0.9223	0.9273
S_{xx}	0.4761	0.5889	0.7131	0.9405	1.1816
My	28.57	35.34	42.79	56.43	70.90
Mn_{DSM}	22.11	31.01	41.99	56.43	70.90
Mn_{EWM}	23.8	31.44	40.24	54.32	68.25
% ERROR	-7.644%	-1.387%	4.168%	3.739%	3.738%

1.5 B DECK - 60 KSI				
Thickness		Mn_{DSM}	Mn_{EWM}	Mn_{DSM} / Mn_{EWM}
24	0.0238	22.110	23.8	0.929
22	0.0295	31.010	31.44	0.986
20	0.0358	41.990	40.24	1.043
18	0.0474	56.430	54.32	1.039
16	0.0598	70.900	68.25	1.039

5.3 Direct Strength Method Calculations

Date:	11/25/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B					
	Gage:	16	GA				
	Strength:	-33	KSI				
	M_y =	38.99	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	7.42470	$M_{cr\ell}$ =	289.48905	kip-in	2	in
dist.	M_{crd}/M_y =	6.00000	M_{crd} =	233.94	kip-in	-	in
global	M_{cre}/M_y =	6.00000	M_{cre} =	233.94	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 38.99 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.37 (local-global slenderness)

$M_{n\ell}$ = 38.99 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.41 (distortional slenderness)

M_{nd} = 38.99 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 38.99 kip-in (local-global controls)

Date: 11/25/14 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:							
	Deck:	1.5B					
	Gage:	16	GA				
	Strength:	-40	KSI				
	M_y =	47.26	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	6.12540	$M_{cr\ell}$ =	289.4864	kip-in	2	in
dist.	M_{crd}/M_y =	6.00000	M_{crd} =	283.56	kip-in	-	in
global	M_{cre}/M_y =	6.00000	M_{cre} =	283.56	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1-\frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 47.26 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell}=M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell}=\left(1-0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.40 (local-global slenderness)

$M_{n\ell}$ = 47.26 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1-0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5}M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.41 (distortional slenderness)

M_{nd} = 47.26 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 47.26 kip-in (local-global controls)

Date: 11/25/14 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B				
	Gage:	16 GA				
	Strength:	-50 KSI				
	M_y =	59.08	kip-in		Length:	
local	$M_{cr\ell}/M_y$ =	4.90030	$M_{cr\ell}$ =	289.50972 kip-in	2	in
dist.	M_{crd}/M_y =	6.00000	M_{crd} =	354.48 kip-in	-	in
global	M_{cre}/M_y =	6.00000	M_{cre} =	354.48 kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 59.08 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.45 (local-global slenderness)

$M_{n\ell}$ = 59.08 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.41 (distortional slenderness)

M_{nd} = 59.08 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 59.08 kip-in (local-global controls)

Date:	11/25/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B					
	Gage:	16	GA				
	Strength:	-60	KSI				
	M_y =	70.90	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	4.08360	$M_{cr\ell}$ =	289.52724	kip-in	2	in
dist.	M_{crd}/M_y =	6.00000	M_{crd} =	425.4	kip-in	-	in
global	M_{cre}/M_y =	6.00000	M_{cre} =	425.4	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 70.90 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.49 (local-global slenderness)

$M_{n\ell}$ = 70.90 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.41 (distortional slenderness)

M_{nd} = 70.90 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 70.90 kip-in (local-global controls)

Date:	11/25/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B				
	Gage:	18 GA				
	Strength:	-33 KSI				
	M_y =	31.04	kip-in		Length:	
local	$M_{cr\ell}/M_y$ =	4.77600	$M_{cr\ell}$ =	148.24704 kip-in	2	in
dist.	M_{crd}/M_y =	6.00000	M_{crd} =	186.24 kip-in	-	in
global	M_{cre}/M_y =	6.00000	M_{cre} =	186.24 kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1-\frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 31.04 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell}=M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell}=\left(1-0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.46 (local-global slenderness)

$M_{n\ell}$ = 31.04 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1-0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5}M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.41 (distortional slenderness)

M_{nd} = 31.04 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 31.04 kip-in (local-global controls)

Date:	11/25/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:							
	Deck:	1.5B					
	Gage:	18	GA				
	Strength:	-40	KSI				
	$M_y =$	37.62	kip-in			Length:	
local	$M_{cr\ell}/M_y =$	3.94020	$M_{cr\ell} =$	148.23032	kip-in	2	in
dist.	$M_{crd}/M_y =$	6.00000	$M_{crd} =$	225.72	kip-in	-	in
global	$M_{cre}/M_y =$	6.00000	$M_{cre} =$	225.72	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9} M_y \left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 37.62 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.50 (local-global slenderness)

$M_{n\ell}$ = 37.62 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.41 (distortional slenderness)

M_{nd} = 37.62 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 37.62 kip-in (local-global controls)

Date:	11/25/14			calc by:	RKD
				check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B			
	Gage:	18 GA			
	Strength:	-50 KSI			
	M_y =	47.02 kip-in		Length:	
local	$M_{cr\ell}/M_y$ =	3.15220	$M_{cr\ell}$ = 148.21644 kip-in	2	in
dist.	M_{crd}/M_y =	6.00000	M_{crd} = 282.12 kip-in	-	in
global	M_{cre}/M_y =	6.00000	M_{cre} = 282.12 kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 47.02 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (*Eq.* 1.2.2-7)

λ_ℓ = 0.56 (local-global slenderness)

$M_{n\ell}$ = 47.02 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (*Eq.* 1.2.2-10)

λ_d = 0.41 (distortional slenderness)

M_{nd} = 47.02 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 47.02 kip-in (local-global controls)

Date:	11/25/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B						
	Gage:	18	GA					
	Strength:	-60	KSI					
	M_y =	56.43	kip-in				Length:	
local	$M_{cr\ell}/M_y$ =	2.62680		$M_{cr\ell}$ =	148.23032	kip-in	2	in
dist.	M_{crd}/M_y =	6.00000		M_{crd} =	338.58	kip-in	-	in
global	M_{cre}/M_y =	6.00000		M_{cre} =	338.58	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1-\frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **56.43** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1-0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.62 (local-global slenderness)

$M_{n\ell}$ = **56.43** kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1-0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5}M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.41 (distortional slenderness)

M_{nd} = **56.43** kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 56.43 kip-in (local-global controls)

Date: 11/26/14 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B					
	Gage:	20	GA				
	Strength:	-33	KSI				
	M_y =	23.53	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	2.84230	$M_{cr\ell}$ =	66.879319	kip-in	2	in
dist.	M_{crd}/M_y =	6.00000	M_{crd} =	141.18	kip-in	-	in
global	M_{cre}/M_y =	6.00000	M_{cre} =	141.18	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1-\frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 23.53 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell}=M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell}=\left(1-0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.59 (local-global slenderness)

$M_{n\ell}$ = 23.53 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1-0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5}M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.41 (distortional slenderness)

M_{nd} = 23.53 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 23.53 kip-in (local-global controls)

Date:	11/26/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:							
	Deck:	1.5B					
	Gage:	20	GA				
	Strength:	-40	KSI				
	M_y =	28.53	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	2.34490	$M_{cr\ell}$ =	66.899997	kip-in	2	in
dist.	M_{crd}/M_y =	6.00000	M_{crd} =	171.18	kip-in	-	in
global	M_{cre}/M_y =	6.00000	M_{cre} =	171.18	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 28.53 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.65 (local-global slenderness)

$M_{n\ell}$ = 28.53 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.41 (distortional slenderness)

M_{nd} = 28.53 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 28.53 kip-in (local-global controls)

Date:	11/26/14		calc by:	RKD
			check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B					
	Gage:	20	GA				
	Strength:	-50	KSI				
	M_y =	35.66	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	1.87590	$M_{cr\ell}$ =	66.894594	kip-in	2	in
dist.	M_{crd}/M_y =	6.00000	M_{crd} =	213.96	kip-in	-	in
global	M_{cre}/M_y =	6.00000	M_{cre} =	213.96	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1-\frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 35.66 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1-0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.73 (local-global slenderness)

$M_{n\ell}$ = 35.66 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1-0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5}M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.41 (distortional slenderness)

M_{nd} = 35.66 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 35.66 kip-in (local-global controls)

Date:	11/26/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B					
	Gage:	20	GA				
	Strength:	-60	KSI				
	M_y =	42.79	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	1.56320	$M_{cr\ell}$ =	66.889328	kip-in	2	in
dist.	M_{crd}/M_y =	6.00000	M_{crd} =	256.74	kip-in	-	in
global	M_{cre}/M_y =	6.00000	M_{cre} =	256.74	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 42.79 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.80 (local-global slenderness)

$M_{n\ell}$ = 41.99 kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.41 (distortional slenderness)

M_{nd} = 42.79 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 41.99 kip-in (local-global controls)

Date:	11/26/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B					
	Gage:	22	GA				
	Strength:	-33	KSI				
	M_y =	19.43	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	2.00370	$M_{cr\ell}$ =	38.931891	kip-in	2	in
dist.	M_{crd}/M_y =	6.00000	M_{crd} =	116.58	kip-in	-	in
global	M_{cre}/M_y =	6.00000	M_{cre} =	116.58	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **19.43** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.71 (local-global slenderness)

$M_{n\ell}$ = **19.43** kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.41 (distortional slenderness)

M_{nd} = **19.43** kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 19.43 kip-in (local-global controls)

Date:	11/26/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B					
	Gage:	22 GA					
	Strength:	-40 KSI					
	M_y =	23.56	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	1.65300	$M_{cr\ell}$ =	38.94468	kip-in	2	in
dist.	M_{crd}/M_y =	6.00000	M_{crd} =	141.36	kip-in	-	in
global	M_{cre}/M_y =	6.00000	M_{cre} =	141.36	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 23.56 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.78 (local-global slenderness)

$M_{n\ell}$ = 23.52 kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.41 (distortional slenderness)

M_{nd} = 23.56 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 23.52 kip-in (local-global controls)

Date:	11/26/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B					
	Gage:	22 GA					
	Strength:	-50 KSI					
	M_y =	29.45	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	1.32240	$M_{cr\ell}$ =	38.94468	kip-in	2	in
dist.	M_{crd}/M_y =	6.00000	M_{crd} =	176.7	kip-in	-	in
global	M_{cre}/M_y =	6.00000	M_{cre} =	176.7	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **29.45** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (*Eq.* 1.2.2-7)

λ_ℓ = 0.87 (local-global slenderness)

$M_{n\ell}$ = **27.41** kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (*Eq.* 1.2.2-10)

λ_d = 0.41 (distortional slenderness)

M_{nd} = **29.45** kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 27.41 kip-in (local-global controls)

Date: 11/26/14 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B					
	Gage:	22	GA				
	Strength:	-60	KSI				
	M_y =	35.34	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	1.10200	$M_{cr\ell}$ =	38.94468	kip-in	2	in
dist.	M_{crd}/M_y =	6.00000	M_{crd} =	212.04	kip-in	-	in
global	M_{cre}/M_y =	6.00000	M_{cre} =	212.04	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1-\frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 35.34 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell}=M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell}=\left(1-0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.95 (local-global slenderness)

$M_{n\ell}$ = 31.01 kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1-0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5}M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.41 (distortional slenderness)

M_{nd} = 35.34 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 31.01 kip-in (local-global controls)

Date:	11/26/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:								
	Deck:	1.5B						
	Gage:	24	GA					
	Strength:	-33	KSI					
	M_y =	15.71	kip-in				Length:	
local	$M_{cr\ell}/M_y$ =	1.37370		$M_{cr\ell}$ =	21.580827	kip-in	2	in
dist.	M_{crd}/M_y =	6.00000		M_{crd} =	94.26	kip-in	-	in
global	M_{cre}/M_y =	6.00000		M_{cre} =	94.26	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 15.71 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.85 (local-global slenderness)

$M_{n\ell}$ = 14.80 kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.41 (distortional slenderness)

M_{nd} = 15.71 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 14.80 kip-in (local-global controls)

Date:	11/26/14		calc by:	RKD
			check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B					
	Gage:	24 GA					
	Strength:	-40 KSI					
	$M_y =$	19.04	kip-in			Length:	
local	$M_{cr\ell}/M_y =$	1.13330	$M_{cr\ell} =$	21.578032	kip-in	2	in
dist.	$M_{crd}/M_y =$	6.00000	$M_{crd} =$	114.24	kip-in	-	in
global	$M_{cre}/M_y =$	6.00000	$M_{cre} =$	114.24	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **19.04** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.94 (local-global slenderness)

$M_{n\ell}$ = **16.86** kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.41 (distortional slenderness)

M_{nd} = **19.04** kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 16.86 kip-in (local-global controls)

Date:	11/26/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B						
	Gage:	24	GA					
	Strength:	-50	KSI					
	M_y =	23.80	kip-in				Length:	
local	$M_{cr\ell}/M_y$ =	0.90665	$M_{cr\ell}$ =	21.57827	kip-in		2	in
dist.	M_{crd}/M_y =	6.00000	M_{crd} =	142.8	kip-in		-	in
global	M_{cre}/M_y =	6.00000	M_{cre} =	142.8	kip-in		-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 23.80 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 1.05 (local-global slenderness)

$M_{n\ell}$ = 19.58 kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.41 (distortional slenderness)

M_{nd} = 23.80 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 19.58 kip-in (local-global controls)

Date: 11/26/14 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B				
	Gage:	24 GA				
	Strength:	-60 KSI				
	M_y =	28.57	kip-in		Length:	
local	$M_{cr\ell}/M_y$ =	0.75554	$M_{cr\ell}$ =	21.585778 kip-in	2	in
dist.	M_{crd}/M_y =	6.00000	M_{crd} =	171.42 kip-in	-	in
global	M_{cre}/M_y =	6.00000	M_{cre} =	171.42 kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 28.57 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 1.15 (local-global slenderness)

$M_{n\ell}$ = 22.11 kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.41 (distortional slenderness)

M_{nd} = 28.57 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 22.11 kip-in (local-global controls)

5.4 Effective Width Method Calculations

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	16	GA
Strength:	33	ksi
Thickness:	0.0598	in.
Total Height:	1.540	in.
Radius:	0.2179	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000592	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	0.030
Corners	0.276	0.194
Top Flange	1.377	0.030
Web	1.275	0.770
Bottom Flange	3.144	1.510

Guess ȳ: 0.610 in.

Stress in Flange:	21.644	ksi
k:	4	
Fcr:	201.212	ksi
λ:	0.328	
ρ:	1.004	
Effective Width:	1.377	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	0	0.000	0.030	0.000	0.000	--
Bottom Corner	12	3.309	1.346	4.452	5.991	0.007
Web	12	15.296	0.770	11.776	9.066	1.884
Top Corner	12	3.309	0.194	0.642	0.125	0.007
Top Flange	6	8.260	1.510	12.472	18.832	--
Bottom Flange	6	18.862	0.030	0.564	0.017	--
	Σ	49.036		29.907	34.032	1.898

Solved ȳ =	ΣLy/ΣL =	0.610	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.930	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	1.058	$in.^4$
S_e =	$I_x/ȳ$ =	1.138	$in.^3$
Mn =	**Se*Fy =**	**37.54**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	16	GA
Strength:	40	ksi
Thickness:	0.0598	in.
Total Height:	1.540	in.
Radius:	0.2179	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000592	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	0.030
Corners	0.276	0.194
Top Flange	1.377	0.030
Web	1.275	0.770
Bottom Flange	3.144	1.510

Guess ȳ: 0.610 in.

Stress in Flange:	26.235	ksi
k:	4	
Fcr:	201.212	ksi
λ:	0.361	
ρ:	1.082	
Effective Width:	1.377	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	0	0.000	0.030	0.000	0.000	--
Bottom Corner	12	3.309	1.346	4.452	5.991	0.007
Web	12	15.296	0.770	11.776	9.066	1.884
Top Corner	12	3.309	0.194	0.642	0.125	0.007
Top Flange	6	8.260	1.510	12.472	18.832	--
Bottom Flange	6	18.862	0.030	0.564	0.017	--
	Σ	49.036		29.907	34.032	1.898

Solved ȳ =	ΣLy/ΣL =	0.610	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.930	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	1.058	$in.^4$
S_e =	$I_x/\bar{y}$ =	1.138	$in.^3$
Mn =	**Se*Fy =**	**45.50**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	16	GA
Strength:	50	ksi
Thickness:	0.0598	in.
Total Height:	1.540	in.
Radius:	0.2179	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000592	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	0.030
Corners	0.276	0.194
Top Flange	1.377	0.030
Web	1.275	0.770
Bottom Flange	3.144	1.510

Guess ȳ: 0.610 in.

Stress in Flange:	32.794	ksi
k:	4	
Fcr:	201.212	ksi
λ:	0.404	
ρ:	1.127	
Effective Width:	1.377	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	0	0.000	0.030	0.000	0.000	--
Bottom Corner	12	3.309	1.346	4.452	5.991	0.007
Web	12	15.296	0.770	11.776	9.066	1.884
Top Corner	12	3.309	0.194	0.642	0.125	0.007
Top Flange	6	8.260	1.510	12.472	18.832	--
Bottom Flange	6	18.862	0.030	0.564	0.017	--
	Σ	49.036		29.907	34.032	1.898

Solved ȳ =	ΣLy/ΣL =	0.610	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.930	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	1.058	$in.^4$
S_e =	$I_x/\bar{y}$ =	1.138	$in.^3$
Mn =	**Se*Fy =**	**56.88**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	16	GA
Strength:	60	ksi
Thickness:	0.0598	in.
Total Height:	1.540	in.
Radius:	0.2179	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000592	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	0.030
Corners	0.276	0.194
Top Flange	1.377	0.030
Web	1.275	0.770
Bottom Flange	3.144	1.510

Guess ȳ: 0.610 in.

Stress in Flange:	39.353	ksi
k:	4	
Fcr:	201.212	ksi
λ:	0.442	
ρ:	1.136	
Effective Width:	1.377	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	0	0.000	0.030	0.000	0.000	--
Bottom Corner	12	3.309	1.346	4.452	5.991	0.007
Web	12	15.296	0.770	11.776	9.066	1.884
Top Corner	12	3.309	0.194	0.642	0.125	0.007
Top Flange	6	8.260	1.510	12.472	18.832	--
Bottom Flange	6	18.862	0.030	0.564	0.017	--
	Σ	49.036		29.907	34.032	1.898

Solved ȳ =	ΣLy/ΣL =	0.610	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.930	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	1.058	$in.^4$
S_e =	$I_x/\bar{y}$ =	1.138	$in.^3$
Mn =	**Se*Fy =**	**68.25**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	18	GA
Strength:	33	ksi
Thickness:	0.0474	in.
Total Height:	1.527	in.
Radius:	0.2117	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000543	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	0.024
Corners	0.268	0.183
Top Flange	1.377	0.024
Web	1.275	0.764
Bottom Flange	3.144	1.504

Guess ȳ: 0.603 in.

Stress in Flange:	21.532	ksi
k:	4	
Fcr:	126.418	ksi
λ:	0.413	
ρ:	1.131	
Effective Width:	1.377	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	0	0.000	0.024	0.000	0.000	--
Bottom Corner	12	3.215	1.344	4.321	5.808	0.007
Web	12	15.296	0.764	11.681	8.921	1.884
Top Corner	12	3.215	0.183	0.589	0.108	0.007
Top Flange	6	8.260	1.504	12.421	18.678	--
Bottom Flange	6	18.862	0.024	0.447	0.011	--
	Σ	48.848		29.459	33.525	1.897

Solved ȳ =	ΣLy/ΣL =	0.603	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.924	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2 \Sigma L]t$ =	0.837	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.905	$in.^3$
Mn =	**Se*Fy =**	**29.88**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	18	GA
Strength:	40	ksi
Thickness:	0.0474	in.
Total Height:	1.527	in.
Radius:	0.2117	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000543	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	0.024
Corners	0.268	0.183
Top Flange	1.377	0.024
Web	1.275	0.764
Bottom Flange	3.144	1.504

Guess ȳ: 0.603 in.

Stress in Flange:	26.099	ksi
k:	4	
Fcr:	126.418	ksi
λ:	0.454	
ρ:	1.135	
Effective Width:	1.377	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	0	0.000	0.024	0.000	0.000	--
Bottom Corner	12	3.215	1.344	4.321	5.808	0.007
Web	12	15.296	0.764	11.681	8.921	1.884
Top Corner	12	3.215	0.183	0.589	0.108	0.007
Top Flange	6	8.260	1.504	12.421	18.678	--
Bottom Flange	6	18.862	0.024	0.447	0.011	--
	Σ	48.848		29.459	33.525	1.897

Solved ȳ =	ΣLy/ΣL =	0.603	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.924	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	0.837	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.905	$in.^3$
Mn =	**Se*Fy =**	**36.22**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	18	GA
Strength:	50	ksi
Thickness:	0.0474	in.
Total Height:	1.527	in.
Radius:	0.2117	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000543	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	0.024
Corners	0.268	0.183
Top Flange	1.377	0.024
Web	1.275	0.764
Bottom Flange	3.144	1.504

Guess ȳ: 0.603 in.

Stress in Flange:	32.624	ksi
k:	4	
Fcr:	126.418	ksi
λ:	0.508	
ρ:	1.116	
Effective Width:	1.377	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	0	0.000	0.024	0.000	0.000	--
Bottom Corner	12	3.215	1.344	4.321	5.808	0.007
Web	12	15.296	0.764	11.681	8.921	1.884
Top Corner	12	3.215	0.183	0.589	0.108	0.007
Top Flange	6	8.260	1.504	12.421	18.678	--
Bottom Flange	6	18.862	0.024	0.447	0.011	--
	Σ	48.848		29.459	33.525	1.897

Solved ȳ =	ΣLy/ΣL =	0.603	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.924	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	0.837	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.905	$in.^3$
Mn =	**Se*Fy =**	**45.27**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	18	GA
Strength:	60	ksi
Thickness:	0.0474	in.
Total Height:	1.527	in.
Radius:	0.2117	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000543	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	0.024
Corners	0.268	0.183
Top Flange	1.377	0.024
Web	1.275	0.764
Bottom Flange	3.144	1.504

Guess ȳ: 0.603 in.

Stress in Flange:	39.148	ksi
k:	4	
Fcr:	126.418	ksi
λ:	0.556	
ρ:	1.087	
Effective Width:	1.377	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	0	0.000	0.024	0.000	0.000	--
Bottom Corner	12	3.215	1.344	4.321	5.808	0.007
Web	12	15.296	0.764	11.681	8.921	1.884
Top Corner	12	3.215	0.183	0.589	0.108	0.007
Top Flange	6	8.260	1.504	12.421	18.678	--
Bottom Flange	6	18.862	0.024	0.447	0.011	--
	Σ	48.848		29.459	33.525	1.897

Solved ȳ =	ΣLy/ΣL =	0.603	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.924	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	0.837	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.905	$in.^3$
Mn =	**Se*Fy =**	**54.32**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	20	GA
Strength:	33	ksi
Thickness:	0.0358	in.
Total Height:	1.516	in.
Radius:	0.2059	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000500	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	0.018
Corners	0.261	0.173
Top Flange	1.377	0.018
Web	1.275	0.758
Bottom Flange	3.144	1.498

Guess $\bar{y}$: 0.597 in.

Stress in Flange:	21.425	ksi
k:	4	
Fcr:	72.114	ksi
λ:	0.545	
ρ:	1.094	
Effective Width:	1.377	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	0	0.000	0.018	0.000	0.000	--
Bottom Corner	12	3.126	1.343	4.198	5.637	0.006
Web	12	15.296	0.758	11.592	8.786	1.884
Top Corner	12	3.126	0.173	0.541	0.094	0.006
Top Flange	6	8.260	1.498	12.373	18.534	--
Bottom Flange	6	18.862	0.018	0.338	0.006	--
	Σ	48.671		29.043	33.056	1.896

Solved $\bar{y}$ =	ΣLy/ΣL =	0.597	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max($\bar{y}$, h - $\bar{y}$) =	0.919	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	0.631	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.686	$in.^3$
Mn =	**Se*Fy =**	**22.65**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	20	GA
Strength:	40	ksi
Thickness:	0.0358	in.
Total Height:	1.516	in.
Radius:	0.2059	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000500	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	0.018
Corners	0.261	0.173
Top Flange	1.377	0.018
Web	1.275	0.758
Bottom Flange	3.144	1.498

Guess ȳ: 0.597 in.

Stress in Flange:	25.969	ksi
k:	4	
Fcr:	72.114	ksi
λ:	0.600	
ρ:	1.055	
Effective Width:	1.377	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	0	0.000	0.018	0.000	0.000	--
Bottom Corner	12	3.126	1.343	4.198	5.637	0.006
Web	12	15.296	0.758	11.592	8.786	1.884
Top Corner	12	3.126	0.173	0.541	0.094	0.006
Top Flange	6	8.260	1.498	12.373	18.534	--
Bottom Flange	6	18.862	0.018	0.338	0.006	--
	Σ	48.671		29.043	33.056	1.896

Solved ȳ =	ΣLy/ΣL =	0.597	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.919	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	0.631	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.686	$in.^3$
Mn =	**Se*Fy =**	**27.46**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	20	GA
Strength:	50	ksi
Thickness:	0.0358	in.
Total Height:	1.516	in.
Radius:	0.2059	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000500	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	0.018
Corners	0.261	0.173
Top Flange	1.377	0.018
Web	1.275	0.758
Bottom Flange	3.144	1.498

Guess ȳ: 0.597 in.

Stress in Flange:	32.462	ksi
k:	4	
Fcr:	72.114	ksi
λ:	0.671	
ρ:	1.002	
Effective Width:	1.377	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	0	0.000	0.018	0.000	0.000	--
Bottom Corner	12	3.126	1.343	4.198	5.637	0.006
Web	12	15.296	0.758	11.592	8.786	1.884
Top Corner	12	3.126	0.173	0.541	0.094	0.006
Top Flange	6	8.260	1.498	12.373	18.534	--
Bottom Flange	6	18.862	0.018	0.338	0.006	--
	Σ	48.671		29.043	33.056	1.896

Solved ȳ =	ΣLy/ΣL =	0.597	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.919	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	0.631	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.686	$in.^3$
Mn =	**Se*Fy =**	**34.32**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	20	GA
Strength:	60	ksi
Thickness:	0.0358	in.
Total Height:	1.516	in.
Radius:	0.2059	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000500	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	0.018
Corners	0.261	0.173
Top Flange	1.377	0.018
Web	1.275	0.758
Bottom Flange	3.144	1.498

Guess $\bar{y}$: 0.590 in.

Stress in Flange:	38.265	ksi
k:	4	
Fcr:	72.114	ksi
λ:	0.728	
ρ:	0.958	
Effective Width:	1.319	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	0	0.000	0.018	0.000	0.000	--
Bottom Corner	12	3.126	1.343	4.198	5.637	0.006
Web	12	15.296	0.758	11.592	8.786	1.884
Top Corner	12	3.126	0.173	0.541	0.094	0.006
Top Flange	6	7.915	1.498	11.856	17.759	--
Bottom Flange	6	18.862	0.018	0.338	0.006	--
	Σ	48.326		28.525	32.281	1.896

Solved $\bar{y}$ =	ΣLy/ΣL =	0.590	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max($\bar{y}$, h - $\bar{y}$) =	0.926	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	0.621	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.671	$in.^3$
Mn =	**Se*Fy =**	**40.24**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	22	GA
Strength:	33	ksi
Thickness:	0.0295	in.
Total Height:	1.510	in.
Radius:	0.20275	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000477	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	0.015
Corners	0.257	0.168
Top Flange	1.377	0.015
Web	1.275	0.755
Bottom Flange	3.144	1.495

Guess ȳ: 0.593 in.

Stress in Flange:	21.366	ksi
k:	4	
Fcr:	48.966	ksi
λ:	0.661	
ρ:	1.010	
Effective Width:	1.377	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	0	0.000	0.015	0.000	0.000	--
Bottom Corner	12	3.079	1.342	4.131	5.544	0.006
Web	12	15.296	0.755	11.544	8.713	1.884
Top Corner	12	3.079	0.168	0.516	0.086	0.006
Top Flange	6	8.260	1.495	12.347	18.456	--
Bottom Flange	6	18.862	0.015	0.278	0.004	--
	Σ	48.576		28.817	32.804	1.895

Solved ȳ =	ΣLy/ΣL =	0.593	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.916	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	0.519	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.567	$in.^3$
Mn =	**Se*Fy =**	**18.70**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	22	GA
Strength:	40	ksi
Thickness:	0.0295	in.
Total Height:	1.510	in.
Radius:	0.20275	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000477	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	0.015
Corners	0.257	0.168
Top Flange	1.377	0.015
Web	1.275	0.755
Bottom Flange	3.144	1.495

Guess ȳ: 0.588 in.

Stress in Flange:	25.494	ksi
k:	4	
Fcr:	48.966	ksi
λ:	0.722	
ρ:	0.963	
Effective Width:	1.326	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	0	0.000	0.015	0.000	0.000	--
Bottom Corner	12	3.079	1.342	4.131	5.544	0.006
Web	12	15.296	0.755	11.544	8.713	1.884
Top Corner	12	3.079	0.168	0.516	0.086	0.006
Top Flange	6	7.958	1.495	11.895	17.780	--
Bottom Flange	6	18.862	0.015	0.278	0.004	--
	Σ	48.273		28.364	32.127	1.895

Solved ȳ =	ΣLy/ΣL =	0.588	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.922	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	0.512	$in.^4$
S_e =	$I_x/ȳ$ =	0.555	$in.^3$
Mn =	**Se*Fy =**	**22.21**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	22	GA
Strength:	50	ksi
Thickness:	0.0295	in.
Total Height:	1.510	in.
Radius:	0.20275	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000477	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	0.015
Corners	0.257	0.168
Top Flange	1.377	0.015
Web	1.275	0.755
Bottom Flange	3.144	1.495

Guess ȳ: 0.579 in.

Stress in Flange:	31.109	ksi
k:	4	
Fcr:	48.966	ksi
λ:	0.797	
ρ:	0.908	
Effective Width:	1.251	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	0	0.000	0.015	0.000	0.000	--
Bottom Corner	12	3.079	1.342	4.131	5.544	0.006
Web	12	15.296	0.755	11.544	8.713	1.884
Top Corner	12	3.079	0.168	0.516	0.086	0.006
Top Flange	6	7.503	1.495	11.215	16.764	--
Bottom Flange	6	18.862	0.015	0.278	0.004	--
	Σ	47.818		27.685	31.112	1.895

Solved ȳ =	ΣLy/ΣL =	0.579	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.931	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	0.501	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.538	$in.^3$
Mn =	**Se*Fy =**	**26.91**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	22	GA
Strength:	60	ksi
Thickness:	0.0295	in.
Total Height:	1.510	in.
Radius:	0.20275	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000477	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	0.015
Corners	0.257	0.168
Top Flange	1.377	0.015
Web	1.275	0.755
Bottom Flange	3.144	1.495

Guess ȳ: 0.572 in.

Stress in Flange:	36.573	ksi
k:	4	
Fcr:	48.966	ksi
λ:	0.864	
ρ:	0.863	
Effective Width:	1.187	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	ΣLy^2 ($in.^3$)	$\Sigma I'_x$ ($in.^3$)
Lip	0	0.000	0.015	0.000	0.000	--
Bottom Corner	12	3.079	1.342	4.131	5.544	0.006
Web	12	15.296	0.755	11.544	8.713	1.884
Top Corner	12	3.079	0.168	0.516	0.086	0.006
Top Flange	6	7.125	1.495	10.650	15.919	--
Bottom Flange	6	18.862	0.015	0.278	0.004	--
	Σ	47.440		27.120	30.267	1.895

Solved ȳ =	ΣLy/ΣL =	0.572	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.938	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2 \Sigma L]t$ =	0.491	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.524	$in.^3$
Mn =	**Se*Fy =**	**31.44**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	24	GA
Strength:	33	ksi
Thickness:	0.0238	in.
Total Height:	1.504	in.
Radius:	0.1999	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000457	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	0.012
Corners	0.253	0.163
Top Flange	1.377	0.012
Web	1.275	0.752
Bottom Flange	3.144	1.492

Guess ȳ: 0.575 in.

Stress in Flange:	20.450	ksi
k:	4	
Fcr:	31.872	ksi
λ:	0.801	
ρ:	0.906	
Effective Width:	1.247	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	0	0.000	0.012	0.000	0.000	--
Bottom Corner	12	3.035	1.341	4.071	5.460	0.005
Web	12	15.296	0.752	11.501	8.647	1.884
Top Corner	12	3.035	0.163	0.493	0.080	0.005
Top Flange	6	7.480	1.492	11.160	16.649	--
Bottom Flange	6	18.862	0.012	0.224	0.003	--
	Σ	47.709		27.449	30.840	1.895

Solved ȳ =	ΣLy/ΣL =	0.575	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.928	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	0.403	$in.^4$
S_e =	I_x/ȳ =	0.434	$in.^3$
Mn =	**Se*Fy =**	**14.33**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	24	GA
Strength:	40	ksi
Thickness:	0.0238	in.
Total Height:	1.504	in.
Radius:	0.1999	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000457	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	0.012
Corners	0.253	0.163
Top Flange	1.377	0.012
Web	1.275	0.752
Bottom Flange	3.144	1.492

Guess ȳ: 0.568 in.

Stress in Flange:	24.253	ksi
k:	4	
Fcr:	31.872	ksi
λ:	0.872	
ρ:	0.857	
Effective Width:	1.180	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	ΣLy^2 ($in.^3$)	$\Sigma I'_x$ ($in.^3$)
Lip	0	0.000	0.012	0.000	0.000	--
Bottom Corner	12	3.035	1.341	4.071	5.460	0.005
Web	12	15.296	0.752	11.501	8.647	1.884
Top Corner	12	3.035	0.163	0.493	0.080	0.005
Top Flange	6	7.081	1.492	10.565	15.761	--
Bottom Flange	6	18.862	0.012	0.224	0.003	--
	Σ	47.310		26.854	29.952	1.895

Solved ȳ =	ΣLy/ΣL =	0.568	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.936	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2 \Sigma L]t$ =	0.395	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.422	$in.^3$
Mn =	**Se*Fy =**	**16.88**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	24	GA
Strength:	50	ksi
Thickness:	0.0238	in.
Total Height:	1.504	in.
Radius:	0.1999	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000457	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	0.012
Corners	0.253	0.163
Top Flange	1.377	0.012
Web	1.275	0.752
Bottom Flange	3.144	1.492

Guess ȳ: 0.559 in.

Stress in Flange:	29.542	ksi
k:	4	
Fcr:	31.872	ksi
λ:	0.963	
ρ:	0.801	
Effective Width:	1.103	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	0	0.000	0.012	0.000	0.000	--
Bottom Corner	12	3.035	1.341	4.071	5.460	0.005
Web	12	15.296	0.752	11.501	8.647	1.884
Top Corner	12	3.035	0.163	0.493	0.080	0.005
Top Flange	6	6.619	1.492	9.875	14.733	--
Bottom Flange	6	18.862	0.012	0.224	0.003	--
	Σ	46.848		26.165	28.924	1.895

Solved ȳ =	ΣLy/ΣL =	0.559	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.945	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	0.386	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.408	$in.^3$
Mn =	**Se*Fy =**	**20.40**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	1.5B	
Gage:	24	GA
Strength:	60	ksi
Thickness:	0.0238	in.
Total Height:	1.504	in.
Radius:	0.1999	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000457	$in.^3$

Element	L (in.)	y from top (in.)
Lip	0.689	0.012
Corners	0.253	0.163
Top Flange	1.377	0.012
Web	1.275	0.752
Bottom Flange	3.144	1.492

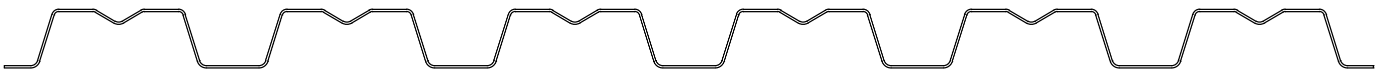
Guess ȳ: 0.551 in.

Stress in Flange:	34.702	ksi
k:	4	
Fcr:	31.872	ksi
λ:	1.043	
ρ:	0.756	
Effective Width:	1.041	in.

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	0	0.000	0.012	0.000	0.000	--
Bottom Corner	12	3.035	1.341	4.071	5.460	0.005
Web	12	15.296	0.752	11.501	8.647	1.884
Top Corner	12	3.035	0.163	0.493	0.080	0.005
Top Flange	6	6.247	1.492	9.320	13.905	--
Bottom Flange	6	18.862	0.012	0.224	0.003	--
	Σ	46.476		25.610	28.096	1.895

Solved ȳ =	ΣLy/ΣL =	0.551	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.953	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	0.378	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.397	$in.^3$
Mn =	**Se*Fy =**	**23.80**	**k-in.**

CHAPTER 6: ANALYSIS | 1.5B DECK (STIFFENED) | + BENDING



6.0 Executive Summary

To observe how a stiffened deck section performs compared to an unstiffened deck section, we created a non-standard shape by adding the compression flange stiffener from the 2C Deck to the 1.5B Deck. With the addition of the stiffener in the compression flange, the Direct Strength Method predicted higher strengths for each of the 1.5B Deck sections undergoing positive flexure. The nominal moment capacity ratio (Mn_{DSM}/Mn_{EWM}) ranged between 1.058 and 1.082. We believe that we see an increase in strength with DSM due to the decreased flat width of the compression flange.

6.1 Mn_{DSM} / Mn_{EWM} vs. Thickness Plot

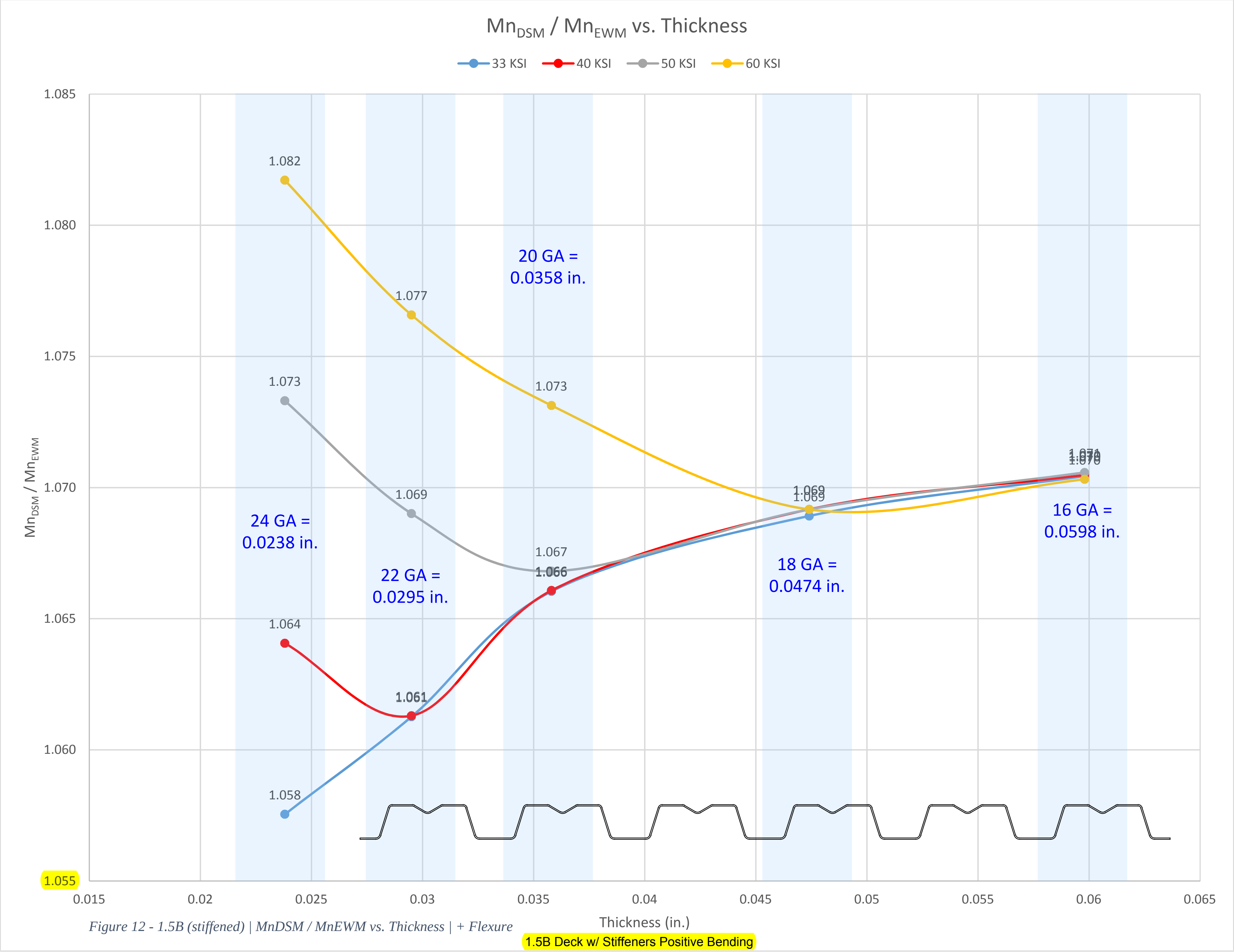


Figure 12 - 1.5B (stiffened) | MnDSM / MnEWM vs. Thickness | + Flexure

6.2 Analysis Results Summary

Table 6 - 1.5B (stiffened) | Analysis Results Summary | + Flexure

1.5B DECK (stiffeners) - 33 KSI					
Gage	**24**	**22**	**20**	**18**	**16**
Thickness	0.0238	0.0295	0.0358	0.0474	0.0598
Curve Radius	0.1999	0.2028	0.2059	0.2117	0.2179
I_G (CUFSM)	0.407	0.505	0.616	0.815	1.025
y-bar (CUFSM)	0.89116	0.89128	0.89289	0.89359	0.89359
Sxx	0.457	0.567	0.690	0.912	1.147
My	15.07	18.71	22.76	30.09	37.84
Mn_{DSM}	15.07	18.71	22.76	30.09	37.84
Mn_{EWM}	14.25	17.63	21.35	28.15	35.35
% ERROR	5.441%	5.772%	6.195%	6.447%	6.580%

1.5B DECK (stiffeners) - 33 KSI				
Thickness		Mn_{DSM}	Mn_{EWM}	Mn_{DSM} / Mn_{EWM}
24	0.0238	15.07	14.25	1.058
22	0.0295	18.71	17.63	1.061
20	0.0358	22.76	21.35	1.066
18	0.0474	30.09	28.15	1.069
16	0.0598	37.84	35.35	1.070

1.5B DECK (stiffeners) - 40 KSI					
Gage	**24**	**22**	**20**	**18**	**16**
Thickness	0.0238	0.0295	0.0358	0.0474	0.0598
Curve Radius	0.1999	0.2028	0.2059	0.2117	0.2179
I_G (CUFSM)	0.407	0.505	0.616	0.815	1.025
y-bar (CUFSM)	0.89116	0.89128	0.89289	0.89359	0.89359
Sxx	0.457	0.567	0.690	0.912	1.147
My	18.27	22.68	27.59	36.48	45.87
Mn_{DSM}	18.27	22.68	27.59	36.48	45.87
Mn_{EWM}	17.17	21.37	25.88	34.12	42.85
% ERROR	6.021%	5.776%	6.198%	6.469%	6.584%

1.5B DECK (stiffeners) - 40 KSI				
Thickness		Mn_{DSM}	Mn_{EWM}	Mn_{DSM} / Mn_{EWM}
24	0.0238	18.27	17.17	1.064
22	0.0295	22.68	21.37	1.061
20	0.0358	27.59	25.88	1.066
18	0.0474	36.48	34.12	1.069
16	0.0598	45.87	42.85	1.070

1.5B DECK (stiffeners) - 50 KSI					
Gage	**24**	**22**	**20**	**18**	**16**
Thickness	0.0238	0.0295	0.0358	0.0474	0.0598
Curve Radius	0.1999	0.2028	0.2059	0.2117	0.2179
I_G (CUFSM)	0.407	0.505	0.616	0.815	1.025
y-bar (CUFSM)	0.89116	0.89128	0.89289	0.89359	0.89359
Sxx	0.457	0.567	0.690	0.912	1.147
My	22.84	28.35	34.49	45.60	57.34
Mn_{DSM}	22.84	28.35	34.49	45.6	57.34
Mn_{EWM}	21.28	26.52	32.33	42.65	53.56
% ERROR	6.830%	6.455%	6.263%	6.469%	6.592%

1.5B DECK (stiffeners) - 50 KSI				
Thickness		Mn_{DSM}	Mn_{EWM}	Mn_{DSM} / Mn_{EWM}
24	0.0238	22.84	21.28	1.073
22	0.0295	28.35	26.52	1.069
20	0.0358	34.49	32.33	1.067
18	0.0474	45.60	42.65	1.069
16	0.0598	57.34	53.56	1.071

1.5B DECK (stiffeners) - 60 KSI					
Gage	**24**	**22**	**20**	**18**	**16**
Thickness	0.0238	0.0295	0.0358	0.0474	0.0598
Curve Radius	0.1999	0.2028	0.2059	0.2117	0.2179
I_G (CUFSM)	0.407	0.505	0.616	0.815	1.025
y-bar (CUFSM)	0.89116	0.89128	0.89289	0.89359	0.89359
Sxx	0.457	0.567	0.690	0.912	1.147
My	27.40	34.02	41.38	54.72	68.80
Mn_{DSM}	27.4	34.02	41.38	54.72	68.8
Mn_{EWM}	25.33	31.6	38.56	51.18	64.28
% ERROR	7.555%	7.113%	6.815%	6.469%	6.570%

1.5B DECK (stiffeners) - 60 KSI				
Thickness		Mn_{DSM}	Mn_{EWM}	Mn_{DSM} / Mn_{EWM}
24	0.0238	27.4	25.33	1.082
22	0.0295	34.02	31.60	1.077
20	0.0358	41.38	38.56	1.073
18	0.0474	54.72	51.18	1.069
16	0.0598	68.80	64.28	1.070

6.3 Direct Strength Method Calculations

Date: 1/11/2015 calc by: RKD

check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)				
	Gage:	16	GA				
	Strength:	33	KSI				
	M_y =	37.84	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	22.34510	$M_{cr\ell}$ =	845.5386	kip-in	1	in
dist.	M_{crd}/M_y =	14.43000	M_{crd} =	546.0312	kip-in	11	in
global	M_{cre}/M_y =	50.00000	M_{cre} =	1892	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 37.84 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (*Eq.* 1.2.2-7)

λ_ℓ = 0.21 (local-global slenderness)

$M_{n\ell}$ = 37.84 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (*Eq.* 1.2.2-10)

λ_d = 0.26 (distortional slenderness)

M_{nd} = 37.84 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 37.84 kip-in (local-global controls)

Date: 1/11/2015 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)				
	Gage:	16	GA				
	Strength:	40	KSI				
	M_y =	45.87	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	18.43470	$M_{cr\ell}$ =	845.5997	kip-in	1	in
dist.	M_{crd}/M_y =	11.90000	M_{crd} =	545.853	kip-in	11	in
global	M_{cre}/M_y =	50.00000	M_{cre} =	2293.5	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 45.87 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.23 (local-global slenderness)

$M_{n\ell}$ = 45.87 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.29 (distortional slenderness)

M_{nd} = 45.87 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 45.87 kip-in (local-global controls)

Date: 1/11/2015 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)					
	Gage:	16	GA					
	Strength:	50	KSI					
	M_y =	57.34	kip-in				Length:	
local	$M_{cr\ell}/M_y$ =	14.74770	$M_{cr\ell}$ =	845.6331	kip-in		1	in
dist.	M_{crd}/M_y =	9.52000	M_{crd} =	545.8768	kip-in		11	in
global	M_{cre}/M_y =	50.00000	M_{cre} =	2867	kip-in		-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 57.34 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.26 (local-global slenderness)

$M_{n\ell}$ = 57.34 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.32 (distortional slenderness)

M_{nd} = 57.34 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 57.34 kip-in (local-global controls)

Date:	1/11/2015	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)					
	Gage:	16	GA					
	Strength:	60	KSI					
	M_y =	68.80	kip-in				Length:	
local	$M_{cr\ell}/M_y$ =	12.28990	$M_{cr\ell}$ =	845.5451	kip-in		1	in
dist.	M_{crd}/M_y =	7.93000	M_{crd} =	545.584	kip-in		11	in
global	M_{cre}/M_y =	50.00000	M_{cre} =	3440	kip-in		-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 68.80 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.29 (local-global slenderness)

$M_{n\ell}$ = 68.80 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.36 (distortional slenderness)

M_{nd} = 68.80 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 68.80 kip-in (local-global controls)

Date:	1/11/2015	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)				
	Gage:	18	GA				
	Strength:	33	KSI				
	M_y =	30.09	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	14.84880	$M_{cr\ell}$ =	446.8004	kip-in	1	in
dist.	M_{crd}/M_y =	11.36000	M_{crd} =	341.8224	kip-in	12	in
global	M_{cre}/M_y =	50.00000	M_{cre} =	1504.5	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 30.09 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.26 (local-global slenderness)

$M_{n\ell}$ = 30.09 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.30 (distortional slenderness)

M_{nd} = 30.09 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 30.09 kip-in (local-global controls)

Date:	1/11/2015	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)			
	Gage:	18	GA			
	Strength:	40	KSI			
	M_y =	36.48	kip-in		Length:	
local	$M_{cr\ell}/M_y$ =	12.25040	$M_{cr\ell}$ =	446.8946 kip-in	1	in
dist.	M_{crd}/M_y =	9.38000	M_{crd} =	342.1824 kip-in	12	in
global	M_{cre}/M_y =	50.00000	M_{cre} =	1824 kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 36.48 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (*Eq.* 1.2.2-7)

λ_ℓ = 0.29 (local-global slenderness)

$M_{n\ell}$ = 36.48 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (*Eq.* 1.2.2-10)

λ_d = 0.33 (distortional slenderness)

M_{nd} = 36.48 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 36.48 kip-in (local-global controls)

Date:	1/11/2015	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)				
	Gage:	18	GA				
	Strength:	50	KSI				
	M_y =	45.60	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	9.80020	$M_{cr\ell}$ =	446.8891	kip-in	1	in
dist.	M_{crd}/M_y =	7.50000	M_{crd} =	342	kip-in	12	in
global	M_{cre}/M_y =	50.00000	M_{cre} =	2280	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 45.60 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.32 (local-global slenderness)

$M_{n\ell}$ = 45.60 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.37 (distortional slenderness)

M_{nd} = 45.60 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 45.60 kip-in (local-global controls)

Date:	1/11/2015	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)				
	Gage:	18	GA				
	Strength:	60	KSI				
	M_y =	54.72	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	8.16690	$M_{cr\ell}$ =	446.8928	kip-in	1	in
dist.	M_{crd}/M_y =	6.25000	M_{crd} =	342	kip-in	12	in
global	M_{cre}/M_y =	50.00000	M_{cre} =	2736	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 54.72 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.35 (local-global slenderness)

$M_{n\ell}$ = 54.72 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.40 (distortional slenderness)

M_{nd} = 54.72 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 54.72 kip-in (local-global controls)

Date:	1/11/2015	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)					
	Gage:	20	GA					
	Strength:	33	KSI					
	M_y =	22.76	kip-in				Length:	
local	$M_{cr\ell}/M_y$ =	8.94530	$M_{cr\ell}$ =	203.595	kip-in		1	in
dist.	M_{crd}/M_y =	8.76000	M_{crd} =	199.3776	kip-in		12	in
global	M_{cre}/M_y =	50.00000	M_{cre} =	1138	kip-in		-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 22.76 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.33 (local-global slenderness)

$M_{n\ell}$ = 22.76 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.34 (distortional slenderness)

M_{nd} = 22.76 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 22.76 kip-in (local-global controls)

Date: 1/11/2015 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:						
	Deck:	1.5B	(stiffeners)			
	Gage:	20	GA			
	Strength:	40	KSI			
	M_y =	27.59	kip-in		Length:	
local	$M_{cr\ell}/M_y$ =	7.37980	$M_{cr\ell}$ =	203.6087 kip-in	1	in
dist.	M_{crd}/M_y =	7.23000	M_{crd} =	199.4757 kip-in	12	in
global	M_{cre}/M_y =	50.00000	M_{cre} =	1379.5 kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 27.59 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.37 (local-global slenderness)

$M_{n\ell}$ = 27.59 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.37 (distortional slenderness)

M_{nd} = 27.59 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 27.59 kip-in (local-global controls)

Date:	1/11/2015		calc by:	RKD
			check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)					
	Gage:	20	GA					
	Strength:	50	KSI					
	M_y =	34.49	kip-in				Length:	
local	$M_{cr\ell}/M_y$ =	5.90380	$M_{cr\ell}$ =	203.6221	kip-in		1	in
dist.	M_{crd}/M_y =	5.78000	M_{crd} =	199.3522	kip-in		12	in
global	M_{cre}/M_y =	50.00000	M_{cre} =	1724.5	kip-in		-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 34.49 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.41 (local-global slenderness)

$M_{n\ell}$ = 34.49 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.42 (distortional slenderness)

M_{nd} = 34.49 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 34.49 kip-in (local-global controls)

Date: 1/11/2015 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)			
	Gage:	20	GA			
	Strength:	60	KSI			
	M_y =	41.38	kip-in			Length:
local	$M_{cr\ell}/M_y$ =	4.91990	$M_{cr\ell}$ =	203.5855	kip-in	1 in
dist.	M_{crd}/M_y =	4.82000	M_{crd} =	199.4516	kip-in	12 in
global	M_{cre}/M_y =	50.00000	M_{cre} =	2069	kip-in	- in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 41.38 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.45 (local-global slenderness)

$M_{n\ell}$ = 41.38 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.46 (distortional slenderness)

M_{nd} = 41.38 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 41.38 kip-in (local-global controls)

Date: 1/11/2015 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:							
	Deck:	1.5B	(stiffeners)				
	Gage:	22	GA				
	Strength:	33	KSI				
	$M_y =$	18.71	kip-in			Length:	
local	$M_{cr\ell}/M_y =$	6.31630	$M_{cr\ell} =$	118.178	kip-in	1	in
dist.	$M_{crd}/M_y =$	7.43000	$M_{crd} =$	139.0153	kip-in	12	in
global	$M_{cre}/M_y =$	50.00000	$M_{cre} =$	935.5	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 18.71 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.40 (local-global slenderness)

$M_{n\ell}$ = 18.71 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.37 (distortional slenderness)

M_{nd} = 18.71 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 18.71 kip-in (local-global controls)

Date:	1/11/2015	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:								
	Deck:	1.5B	(stiffeners)					
	Gage:	22	GA					
	Strength:	40	KSI					
	M_y =	22.68	kip-in				Length:	
local	$M_{cr\ell}/M_y$ =	5.21100	$M_{cr\ell}$ =	118.1855	kip-in		1	in
dist.	M_{crd}/M_y =	6.13000	M_{crd} =	139.0284	kip-in		12	in
global	M_{cre}/M_y =	50.00000	M_{cre} =	1134	kip-in		-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 22.68 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.44 (local-global slenderness)

$M_{n\ell}$ = 22.68 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.40 (distortional slenderness)

M_{nd} = 22.68 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 22.68 kip-in (local-global controls)

Date: 1/11/2015 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)			
	Gage:	22	GA			
	Strength:	50	KSI			
	M_y =	28.35	kip-in			Length:
local	$M_{cr\ell}/M_y$ =	4.16880	$M_{cr\ell}$ =	118.1855 kip-in	1	in
dist.	M_{crd}/M_y =	4.91000	M_{crd} =	139.1985 kip-in	12	in
global	M_{cre}/M_y =	50.00000	M_{cre} =	1417.5 kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 28.35 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.49 (local-global slenderness)

$M_{n\ell}$ = 28.35 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.45 (distortional slenderness)

M_{nd} = 28.35 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 28.35 kip-in (local-global controls)

Date:	1/11/2015	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)				
	Gage:	22	GA				
	Strength:	60	KSI				
	$M_y =$	34.02	kip-in			Length:	
local	$M_{cr\ell}/M_y =$	3.47400	$M_{cr\ell} =$	118.1855	kip-in	1	in
dist.	$M_{crd}/M_y =$	4.09000	$M_{crd} =$	139.1418	kip-in	12	in
global	$M_{cre}/M_y =$	50.00000	$M_{cre} =$	1701	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 34.02 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (*Eq.* 1.2.2-7)

λ_ℓ = 0.54 (local-global slenderness)

$M_{n\ell}$ = 34.02 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (*Eq.* 1.2.2-10)

λ_d = 0.49 (distortional slenderness)

M_{nd} = 34.02 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 34.02 kip-in (local-global controls)

Date:	1/11/2015	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)				
	Gage:	24	GA				
	Strength:	33	KSI				
	$M_y =$	15.07	kip-in			Length:	
local	$M_{cr\ell}/M_y =$	4.25890	$M_{cr\ell} =$	64.18162	kip-in	1	in
dist.	$M_{crd}/M_y =$	6.23000	$M_{crd} =$	93.8861	kip-in	24	in
global	$M_{cre}/M_y =$	50.00000	$M_{cre} =$	753.5	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9} M_y \left(1 - \frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 15.07 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

$\lambda_\ell =$ 0.48 (local-global slenderness)

$M_{n\ell}$ = 15.07 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

$\lambda_d =$ 0.40 (distortional slenderness)

M_{nd} = 15.07 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 15.07 kip-in (local-global controls)

Date: 1/11/2015 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)				
	Gage:	24	GA				
	Strength:	40	KSI				
	M_y =	18.27	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	3.51360	$M_{cr\ell}$ =	64.19347	kip-in	1	in
dist.	M_{crd}/M_y =	5.14000	M_{crd} =	93.9078	kip-in	24	in
global	M_{cre}/M_y =	50.00000	M_{cre} =	913.5	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **18.27** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.53 (local-global slenderness)

$M_{n\ell}$ = **18.27** kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.44 (distortional slenderness)

M_{nd} = **18.27** kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 18.27 kip-in (local-global controls)

Date:	1/11/2015	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)				
	Gage:	24	GA				
	Strength:	50	KSI				
	$M_y =$	22.84	kip-in			Length:	
local	$M_{cr\ell}/M_y =$	2.81090	$M_{cr\ell} =$	64.20096	kip-in	1	in
dist.	$M_{crd}/M_y =$	4.11000	$M_{crd} =$	93.8724	kip-in	24	in
global	$M_{cre}/M_y =$	50.00000	$M_{cre} =$	1142	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 22.84 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.60 (local-global slenderness)

$M_{n\ell}$ = 22.84 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.49 (distortional slenderness)

M_{nd} = 22.84 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 22.84 kip-in (local-global controls)

Date: 1/11/2015 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)					
	Gage:	24	GA					
	Strength:	60	KSI					
	M_y =	27.40	kip-in				Length:	
local	$M_{cr\ell}/M_y$ =	2.34240	$M_{cr\ell}$ =	64.18176	kip-in		1	in
dist.	M_{crd}/M_y =	3.42000	M_{crd} =	93.708	kip-in		24	in
global	M_{cre}/M_y =	50.00000	M_{cre} =	1370	kip-in		-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1-\frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 27.40 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell}=M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell}=\left(1-0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.65 (local-global slenderness)

$M_{n\ell}$ = 27.40 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1-0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5}M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.54 (distortional slenderness)

M_{nd} = 27.40 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 27.40 kip-in (local-global controls)

6.4 Effective Width Method Calculations

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)	bo:	3.144	in.
Gage:	16	GA	h:	1.275	in.
Strength:	33	ksi	bp:	0.895	in.
Thickness:	0.0598	in.	Ag:	0.199	$in.^2$
Total Height:	1.540	in.	n:	1	
Radius:	0.2179	in.			
θ:	72.5	deg	$θ_{Stiff}$:	30.625	deg
θ:	1.265	rad	$θ_{Stiff}$:	0.535	rad
Curve I'_x:	0.000592	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000010	$in.^3$

Guess ȳ: 0.629 in.

Stress in Flange:	22.812	ksi	δ:	1.056	
k:	13.475		Isp:	0.004	$in.^4$
Fcr:	130.002	ksi	γ:	63.186	
λ:	0.419		β:	3.359	
ρ:	1.000		kd:	7.896	
Effective Width:	3.320	in.	kloc:	49.309	
			R:	1.707	

Element	L (in.)	y from top (in.)
Lip	0.688	1.510
Corners	0.276	0.194
$Corners_{Stiff}$	0.116	0.238
Bottom Flange	1.377	1.510
Web	1.275	0.770
Web_{Stiff}	0.564	0.215
Top Flange	3.320	0.110

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.377	1.510	2.079	3.139	--
Bottom Corner	12	3.309	1.346	4.452	5.991	0.007
Web	12	15.296	0.770	11.776	9.066	1.884
Top Corner	12	3.309	0.194	0.642	0.125	0.007
Top Flange	6	19.920	0.110	2.183	0.239	--
Bottom Flange	5	6.884	1.510	10.394	15.694	--
Low $Corner_{Stiff}$	0	0.000	1.302	0.000	0.000	0.000
Web_{Stiff}	0	0.000	1.325	0.000	0.000	0.000
High $Corner_{Stiff}$	0	0.000	0.872	0.000	0.000	0.000
	Σ	50.093		31.527	34.254	1.898

Solved ȳ =	ΣLy/ΣL =	0.629	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.910	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	0.975	$in.^4$
S_e =	$I_x/ȳ$ =	1.071	$in.^3$
Mn =	**Se*Fy =**	**35.35**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)	bo:	3.144	in.
Gage:	16	GA	h:	1.275	in.
Strength:	40	ksi	bp:	0.895	in.
Thickness:	0.0598	in.	Ag:	0.199	$in.^2$
Total Height:	1.540	in.	n:	1	
Radius:	0.2179	in.			
θ:	72.5	deg	$θ_{Stiff}$:	30.625	deg
θ:	1.265	rad	$θ_{Stiff}$:	0.535	rad
Curve I'_x:	0.000592	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000010	$in.^3$

Guess ȳ: 0.629 in.

Stress in Flange:	27.651	ksi	δ:	1.056	
k:	13.475		Isp:	0.004	$in.^4$
Fcr:	130.002	ksi	γ:	63.186	
λ:	0.461		β:	3.359	
ρ:	1.000		kd:	7.896	
Effective Width:	3.320	in.	kloc:	49.309	
			R:	1.707	

Element	L (in.)	y from top (in.)
Lip	0.688	1.510
Corners	0.276	0.194
$Corners_{Stiff}$	0.116	0.238
Bottom Flange	1.377	1.510
Web	1.275	0.770
Web_{Stiff}	0.564	0.215
Top Flange	3.320	0.110

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.377	1.510	2.079	3.139	--
Bottom Corner	12	3.309	1.346	4.452	5.991	0.007
Web	12	15.296	0.770	11.776	9.066	1.884
Top Corner	12	3.309	0.194	0.642	0.125	0.007
Top Flange	6	19.920	0.110	2.183	0.239	--
Bottom Flange	5	6.884	1.510	10.394	15.694	--
Low $Corner_{Stiff}$	0	0.000	1.302	0.000	0.000	0.000
Web_{Stiff}	0	0.000	1.325	0.000	0.000	0.000
High $Corner_{Stiff}$	0	0.000	0.872	0.000	0.000	0.000
	Σ	50.093		31.527	34.254	1.898

Solved ȳ =	ΣLy/ΣL =	0.629	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.910	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	0.975	$in.^4$
S_e =	I_x/ȳ =	1.071	$in.^3$
Mn =	**Se*Fy =**	**42.85**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)	bo:	3.144	in.
Gage:	16	GA	h:	1.275	in.
Strength:	50	ksi	bp:	0.895	in.
Thickness:	0.0598	in.	Ag:	0.199	$in.^2$
Total Height:	1.540	in.	n:	1	
Radius:	0.2179	in.			
θ:	72.5	deg	θ_{Stiff}:	30.625	deg
θ:	1.265	rad	θ_{Stiff}:	0.535	rad
Curve I'_x:	0.000592	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000010	$in.^3$

Guess ȳ: 0.629 in.

Stress in Flange:	34.563	ksi	δ:	1.056	
k:	13.475		Isp:	0.004	$in.^4$
Fcr:	130.002	ksi	γ:	63.186	
λ:	0.516		β:	3.359	
ρ:	1.000		kd:	7.896	
Effective Width:	3.320	in.	kloc:	49.309	
			R:	1.707	

Element	L (in.)	y from top (in.)
Lip	0.688	1.510
Corners	0.276	0.194
$Corners_{Stiff}$	0.116	0.238
Bottom Flange	1.377	1.510
Web	1.275	0.770
Web_{Stiff}	0.564	0.215
Top Flange	3.320	0.110

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.377	1.510	2.079	3.139	--
Bottom Corner	12	3.309	1.346	4.452	5.991	0.007
Web	12	15.296	0.770	11.776	9.066	1.884
Top Corner	12	3.309	0.194	0.642	0.125	0.007
Top Flange	6	19.920	0.110	2.183	0.239	--
Bottom Flange	5	6.884	1.510	10.394	15.694	--
Low $Corner_{Stiff}$	0	0.000	1.302	0.000	0.000	0.000
Web_{Stiff}	0	0.000	1.325	0.000	0.000	0.000
High $Corner_{Stiff}$	0	0.000	0.872	0.000	0.000	0.000
	Σ	50.093		31.527	34.254	1.898

Solved ȳ =	ΣLy/ΣL =	0.629	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.910	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2 \Sigma L]t$ =	0.975	$in.^4$
S_e =	$I_x/\bar{y}$ =	1.071	$in.^3$
Mn =	**Se*Fy =**	**53.56**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)	bo:	3.144	in.
Gage:	16	GA	h:	1.275	in.
Strength:	60	ksi	bp:	0.895	in.
Thickness:	0.0598	in.	Ag:	0.199	$in.^2$
Total Height:	1.540	in.	n:	1	
Radius:	0.2179	in.			
θ:	72.5	deg	$θ_{Stiff}$:	30.625	deg
θ:	1.265	rad	$θ_{Stiff}$:	0.535	rad
Curve I'_x:	0.000592	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000010	$in.^3$

Guess ȳ: 0.629 in.

Stress in Flange:	41.476	ksi	δ:	1.056	
k:	13.475		Isp:	0.004	$in.^4$
Fcr:	130.002	ksi	γ:	63.186	
λ:	0.565		β:	3.359	
ρ:	1.000		kd:	7.896	
Effective Width:	3.320	in.	kloc:	49.309	
			R:	1.707	

Element	L (in.)	y from top (in.)
Lip	0.688	1.510
Corners	0.276	0.194
$Corners_{Stiff}$	0.116	0.238
Bottom Flange	1.377	1.510
Web	1.275	0.770
Web_{Stiff}	0.564	0.215
Top Flange	3.320	0.110

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.377	1.510	2.079	3.139	--
Bottom Corner	12	3.309	1.346	4.452	5.991	0.007
Web	12	15.296	0.770	11.776	9.066	1.884
Top Corner	12	3.309	0.194	0.642	0.125	0.007
Top Flange	6	19.920	0.110	2.183	0.239	--
Bottom Flange	5	6.884	1.510	10.394	15.694	--
Low $Corner_{Stiff}$	0	0.000	1.302	0.000	0.000	0.000
Web_{Stiff}	0	0.000	1.325	0.000	0.000	0.000
High $Corner_{Stiff}$	0	0.000	0.872	0.000	0.000	0.000
	Σ	50.093		31.527	34.254	1.898

Solved ȳ =	ΣLy/ΣL =	0.629	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.910	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	0.975	$in.^4$
S_e =	$I_x/ȳ$ =	1.071	$in.^3$
Mn =	**Se*Fy =**	**64.28**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)	bo:	3.144	in.
Gage:	18	GA	h:	1.275	in.
Strength:	33	ksi	bp:	0.895	in.
Thickness:	0.0474	in.	Ag:	0.157	$in.^2$
Total Height:	1.527	in.	n:	1	
Radius:	0.2117	in.			
θ:	72.5	deg	θ_{Stiff}:	30.625	deg
θ:	1.265	rad	θ_{Stiff}:	0.535	rad
Curve I'_x:	0.000543	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000009	$in.^3$

Guess ȳ: 0.622 in.

Stress in Flange:	22.701	ksi	δ:	1.056	
k:	16.607		Isp:	0.003	$in.^4$
Fcr:	100.657	ksi	γ:	99.481	
λ:	0.475		β:	3.760	
ρ:	1.000		kd:	9.730	
Effective Width:	3.320	in.	kloc:	49.309	
			R:	1.707	

Element	L (in.)	y from top (in.)
Lip	0.688	1.504
Corners	0.268	0.183
$Corners_{Stiff}$	0.113	0.225
Bottom Flange	1.377	1.504
Web	1.275	0.764
Web_{Stiff}	0.564	0.209
Top Flange	3.320	0.103

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	ΣLy^2 ($in.^3$)	$\Sigma I'_x$ ($in.^3$)
Lip	2	1.377	1.504	2.070	3.113	--
Bottom Corner	12	3.215	1.344	4.321	5.808	0.007
Web	12	15.296	0.764	11.681	8.921	1.884
Top Corner	12	3.215	0.183	0.589	0.108	0.007
Top Flange	6	19.920	0.103	2.054	0.212	--
Bottom Flange	5	6.884	1.504	10.351	15.565	--
Low $Corner_{Stiff}$	0	0.000	1.302	0.000	0.000	0.000
Web_{Stiff}	0	0.000	1.319	0.000	0.000	0.000
High $Corner_{Stiff}$	0	0.000	0.885	0.000	0.000	0.000
	Σ	49.905		31.066	33.726	1.897

Solved ȳ =	ΣLy/ΣL =	0.622	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.905	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2\Sigma L]t$ =	0.772	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.853	$in.^3$
Mn =	**Se*Fy =**	**28.15**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)	bo:	3.144	in.
Gage:	18	GA	h:	1.275	in.
Strength:	40	ksi	bp:	0.895	in.
Thickness:	0.0474	in.	Ag:	0.157	$in.^2$
Total Height:	1.527	in.	n:	1	
Radius:	0.2117	in.			
θ:	72.5	deg	$θ_{Stiff}$:	30.625	deg
θ:	1.265	rad	$θ_{Stiff}$:	0.535	rad
Curve I'_x:	0.000543	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000009	$in.^3$

Guess ȳ: 0.622 in.

Stress in Flange:	27.517	ksi	δ:	1.056	
k:	16.607		Isp:	0.003	$in.^4$
Fcr:	100.657	ksi	γ:	99.481	
λ:	0.523		β:	3.760	
ρ:	1.000		kd:	9.730	
Effective Width:	3.320	in.	kloc:	49.309	
			R:	1.707	

Element	L (in.)	y from top (in.)
Lip	0.688	1.504
Corners	0.268	0.183
$Corners_{Stiff}$	0.113	0.225
Bottom Flange	1.377	1.504
Web	1.275	0.764
Web_{Stiff}	0.564	0.209
Top Flange	3.320	0.103

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.377	1.504	2.070	3.113	--
Bottom Corner	12	3.215	1.344	4.321	5.808	0.007
Web	12	15.296	0.764	11.681	8.921	1.884
Top Corner	12	3.215	0.183	0.589	0.108	0.007
Top Flange	6	19.920	0.103	2.054	0.212	--
Bottom Flange	5	6.884	1.504	10.351	15.565	--
Low $Corner_{Stiff}$	0	0.000	1.302	0.000	0.000	0.000
Web_{Stiff}	0	0.000	1.319	0.000	0.000	0.000
High $Corner_{Stiff}$	0	0.000	0.885	0.000	0.000	0.000
	Σ	49.905		31.066	33.726	1.897

Solved ȳ =	ΣLy/ΣL =	0.622	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.905	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	0.772	$in.^4$
S_e =	I_x/ȳ =	0.853	$in.^3$
Mn =	**Se*Fy =**	**34.12**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)	bo:	3.144	in.
Gage:	18	GA	h:	1.275	in.
Strength:	50	ksi	bp:	0.895	in.
Thickness:	0.0474	in.	Ag:	0.157	$in.^2$
Total Height:	1.527	in.	n:	1	
Radius:	0.2117	in.			
θ:	72.5	deg	$θ_{Stiff}$:	30.625	deg
θ:	1.265	rad	$θ_{Stiff}$:	0.535	rad
Curve I'_x:	0.000543	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000009	$in.^3$

Guess ȳ: 0.622 in.

Stress in Flange:	34.396	ksi	δ:	1.056	
k:	16.607		Isp:	0.003	$in.^4$
Fcr:	100.657	ksi	γ:	99.481	
λ:	0.585		β:	3.760	
ρ:	1.000		kd:	9.730	
Effective Width:	3.320	in.	kloc:	49.309	
			R:	1.707	

Element	L (in.)	y from top (in.)
Lip	0.688	1.504
Corners	0.268	0.183
$Corners_{Stiff}$	0.113	0.225
Bottom Flange	1.377	1.504
Web	1.275	0.764
Web_{Stiff}	0.564	0.209
Top Flange	3.320	0.103

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.377	1.504	2.070	3.113	--
Bottom Corner	12	3.215	1.344	4.321	5.808	0.007
Web	12	15.296	0.764	11.681	8.921	1.884
Top Corner	12	3.215	0.183	0.589	0.108	0.007
Top Flange	6	19.920	0.103	2.054	0.212	--
Bottom Flange	5	6.884	1.504	10.351	15.565	--
Low $Corner_{Stiff}$	0	0.000	1.302	0.000	0.000	0.000
Web_{Stiff}	0	0.000	1.319	0.000	0.000	0.000
High $Corner_{Stiff}$	0	0.000	0.885	0.000	0.000	0.000
	Σ	49.905		31.066	33.726	1.897

Solved ȳ =	ΣLy/ΣL =	0.622	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.905	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	0.772	$in.^4$
S_e =	$I_x/ȳ$ =	0.853	$in.^3$
Mn =	**Se*Fy =**	**42.65**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)	bo:	3.144	in.
Gage:	18	GA	h:	1.275	in.
Strength:	60	ksi	bp:	0.895	in.
Thickness:	0.0474	in.	Ag:	0.157	$in.^2$
Total Height:	1.527	in.	n:	1	
Radius:	0.2117	in.			
θ:	72.5	deg	$θ_{Stiff}$:	30.625	deg
θ:	1.265	rad	$θ_{Stiff}$:	0.535	rad
Curve I'_x:	0.000543	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000009	$in.^3$

Guess ȳ: 0.622 in.

Stress in Flange:	41.275	ksi	δ:	1.056	
k:	16.607		Isp:	0.003	$in.^4$
Fcr:	100.657	ksi	γ:	99.481	
λ:	0.640		β:	3.760	
ρ:	1.000		kd:	9.730	
Effective Width:	3.320	in.	kloc:	49.309	
			R:	1.707	

Element	L (in.)	y from top (in.)
Lip	0.688	1.504
Corners	0.268	0.183
$Corners_{Stiff}$	0.113	0.225
Bottom Flange	1.377	1.504
Web	1.275	0.764
Web_{Stiff}	0.564	0.209
Top Flange	3.320	0.103

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.377	1.504	2.070	3.113	--
Bottom Corner	12	3.215	1.344	4.321	5.808	0.007
Web	12	15.296	0.764	11.681	8.921	1.884
Top Corner	12	3.215	0.183	0.589	0.108	0.007
Top Flange	6	19.920	0.103	2.054	0.212	--
Bottom Flange	5	6.884	1.504	10.351	15.565	--
Low $Corner_{Stiff}$	0	0.000	1.302	0.000	0.000	0.000
Web_{Stiff}	0	0.000	1.319	0.000	0.000	0.000
High $Corner_{Stiff}$	0	0.000	0.885	0.000	0.000	0.000
	Σ	49.905		31.066	33.726	1.897

Solved ȳ =	ΣLy/ΣL =	0.622	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.905	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	0.772	$in.^4$
S_e =	$I_x/ȳ$ =	0.853	$in.^3$
Mn =	**Se*Fy =**	**51.18**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)	bo:	3.144	in.
Gage:	20	GA	h:	1.275	in.
Strength:	33	ksi	bp:	0.895	in.
Thickness:	0.0358	in.	Ag:	0.119	$in.^2$
Total Height:	1.516	in.	n:	1	
Radius:	0.2059	in.			
θ:	72.5	deg	$θ_{Stiff}$:	30.625	deg
θ:	1.265	rad	$θ_{Stiff}$:	0.535	rad
Curve I'_x:	0.000500	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000008	$in.^3$

Guess ȳ: 0.616 in.

Stress in Flange:	22.597	ksi	δ:	1.056	
k:	21.505		Isp:	0.002	$in.^4$
Fcr:	74.355	ksi	γ:	172.608	
λ:	0.551		β:	4.314	
ρ:	1.000		kd:	12.600	
Effective Width:	3.320	in.	kloc:	49.309	
			R:	1.707	

Element	L (in.)	y from top (in.)
Lip	0.688	1.498
Corners	0.261	0.173
$Corners_{Stiff}$	0.110	0.214
Bottom Flange	1.377	1.498
Web	1.275	0.758
Web_{Stiff}	0.564	0.203
Top Flange	3.320	0.097

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.377	1.498	2.062	3.089	--
Bottom Corner	12	3.126	1.343	4.198	5.637	0.006
Web	12	15.296	0.758	11.592	8.786	1.884
Top Corner	12	3.126	0.173	0.541	0.094	0.006
Top Flange	6	19.920	0.097	1.932	0.187	--
Bottom Flange	5	6.884	1.498	10.311	15.445	--
Low $Corner_{Stiff}$	0	0.000	1.302	0.000	0.000	0.000
Web_{Stiff}	0	0.000	1.313	0.000	0.000	0.000
High $Corner_{Stiff}$	0	0.000	0.896	0.000	0.000	0.000
	Σ	49.729		30.637	33.238	1.896

Solved ȳ =	ΣLy/ΣL =	0.616	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.900	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	0.582	$in.^4$
S_e =	I_x/ȳ =	0.647	$in.^3$
Mn =	**Se*Fy =**	**21.35**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)	bo:	3.144	in.
Gage:	20	GA	h:	1.275	in.
Strength:	40	ksi	bp:	0.895	in.
Thickness:	0.0358	in.	Ag:	0.119	$in.^2$
Total Height:	1.516	in.	n:	1	
Radius:	0.2059	in.			
θ:	72.5	deg	$θ_{Stiff}$:	30.625	deg
θ:	1.265	rad	$θ_{Stiff}$:	0.535	rad
Curve I'_x:	0.000500	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000008	$in.^3$

Guess ȳ: 0.616 in.

Stress in Flange:	27.390	ksi	δ:	1.056	
k:	21.505		Isp:	0.002	$in.^4$
Fcr:	74.355	ksi	γ:	172.608	
λ:	0.607		β:	4.314	
ρ:	1.000		kd:	12.600	
Effective Width:	3.320	in.	kloc:	49.309	
			R:	1.707	

Element	L (in.)	y from top (in.)
Lip	0.688	1.498
Corners	0.261	0.173
$Corners_{Stiff}$	0.110	0.214
Bottom Flange	1.377	1.498
Web	1.275	0.758
Web_{Stiff}	0.564	0.203
Top Flange	3.320	0.097

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.377	1.498	2.062	3.089	--
Bottom Corner	12	3.126	1.343	4.198	5.637	0.006
Web	12	15.296	0.758	11.592	8.786	1.884
Top Corner	12	3.126	0.173	0.541	0.094	0.006
Top Flange	6	19.920	0.097	1.932	0.187	--
Bottom Flange	5	6.884	1.498	10.311	15.445	--
Low $Corner_{Stiff}$	0	0.000	1.302	0.000	0.000	0.000
Web_{Stiff}	0	0.000	1.313	0.000	0.000	0.000
High $Corner_{Stiff}$	0	0.000	0.896	0.000	0.000	0.000
	Σ	49.729		30.637	33.238	1.896

Solved ȳ =	ΣLy/ΣL =	0.616	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.900	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	0.582	$in.^4$
S_e =	I_x/ȳ =	0.647	$in.^3$
Mn =	**Se*Fy =**	**25.88**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)	bo:	3.144	in.
Gage:	20	GA	h:	1.275	in.
Strength:	50	ksi	bp:	0.895	in.
Thickness:	0.0358	in.	Ag:	0.119	$in.^2$
Total Height:	1.516	in.	n:	1	
Radius:	0.2059	in.			
θ:	72.5	deg	$θ_{Stiff}$:	30.625	deg
θ:	1.265	rad	$θ_{Stiff}$:	0.535	rad
Curve I'_x:	0.000500	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000008	$in.^3$

Guess ȳ: 0.617 in.

Stress in Flange:	34.332	ksi	δ:	1.056	
k:	21.505		Isp:	0.002	$in.^4$
Fcr:	74.355	ksi	γ:	172.608	
λ:	0.680		β:	4.314	
ρ:	0.995		kd:	12.600	
Effective Width:	3.304	in.	kloc:	49.309	
			R:	1.707	

Element	L (in.)	y from top (in.)
Lip	0.688	1.498
Corners	0.261	0.173
$Corners_{Stiff}$	0.110	0.214
Bottom Flange	1.377	1.498
Web	1.275	0.758
Web_{Stiff}	0.564	0.203
Top Flange	3.304	0.097

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.377	1.498	2.062	3.089	--
Bottom Corner	12	3.126	1.343	4.198	5.637	0.006
Web	12	15.296	0.758	11.592	8.786	1.884
Top Corner	12	3.126	0.173	0.541	0.094	0.006
Top Flange	6	19.824	0.097	1.923	0.187	--
Bottom Flange	5	6.884	1.498	10.311	15.445	--
$Low\ Corner_{Stiff}$	0	0.000	1.302	0.000	0.000	0.000
Web_{Stiff}	0	0.000	1.313	0.000	0.000	0.000
$High\ Corner_{Stiff}$	0	0.000	0.896	0.000	0.000	0.000
	Σ	49.633		30.628	33.237	1.896

Solved ȳ =	ΣLy/ΣL =	0.617	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.899	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	0.581	$in.^4$
S_e =	$I_x/ȳ$ =	0.647	$in.^3$
Mn =	**Se*Fy =**	**32.33**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)	bo:	3.144	in.
Gage:	20	GA	h:	1.275	in.
Strength:	60	ksi	bp:	0.895	in.
Thickness:	0.0358	in.	Ag:	0.119	$in.^2$
Total Height:	1.516	in.	n:	1	
Radius:	0.2059	in.			
θ:	72.5	deg	$θ_{Stiff}$:	30.625	deg
θ:	1.265	rad	$θ_{Stiff}$:	0.535	rad
Curve I'_x:	0.000500	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000008	$in.^3$

Guess ȳ: 0.630 in.

Stress in Flange:	42.617	ksi	δ:	1.056	
k:	21.505		Isp:	0.002	$in.^4$
Fcr:	74.355	ksi	γ:	172.608	
λ:	0.757		β:	4.314	
ρ:	0.937		kd:	12.600	
Effective Width:	3.111	in.	kloc:	49.309	
			R:	1.707	

Element	L (in.)	y from top (in.)
Lip	0.688	1.498
Corners	0.261	0.173
$Corners_{Stiff}$	0.110	0.214
Bottom Flange	1.377	1.498
Web	1.275	0.758
Web_{Stiff}	0.564	0.203
Top Flange	3.111	0.097

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.377	1.498	2.062	3.089	--
Bottom Corner	12	3.126	1.343	4.198	5.637	0.006
Web	12	15.296	0.758	11.592	8.786	1.884
Top Corner	12	3.126	0.173	0.541	0.094	0.006
Top Flange	6	18.666	0.097	1.811	0.176	--
Bottom Flange	5	6.884	1.498	10.311	15.445	--
Low $Corner_{Stiff}$	0	0.000	1.302	0.000	0.000	0.000
Web_{Stiff}	0	0.000	1.313	0.000	0.000	0.000
High $Corner_{Stiff}$	0	0.000	0.896	0.000	0.000	0.000
	Σ	48.475		30.516	33.226	1.896

Solved ȳ =	ΣLy/ΣL =	0.630	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.886	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	0.570	$in.^4$
S_e =	$I_x/ȳ$ =	0.643	$in.^3$
Mn =	**Se*Fy =**	**38.56**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)	bo:	3.144	in.
Gage:	22	GA	h:	1.275	in.
Strength:	33	ksi	bp:	0.895	in.
Thickness:	0.0295	in.	Ag:	0.098	$in.^2$
Total Height:	1.510	in.	n:	1	
Radius:	0.20275	in.			
θ:	72.5	deg	$θ_{Stiff}$:	30.625	deg
θ:	1.265	rad	$θ_{Stiff}$:	0.535	rad
Curve I'_x:	0.000477	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000008	$in.^3$

Guess ȳ: 0.613 in.

Stress in Flange:	22.539	ksi	δ:	1.056	
k:	25.794		Isp:	0.002	$in.^4$
Fcr:	60.558	ksi	γ:	253.018	
λ:	0.610		β:	4.745	
ρ:	1.000		kd:	15.113	
Effective Width:	3.320	in.	kloc:	49.309	
			R:	1.707	

Element	L (in.)	y from top (in.)
Lip	0.688	1.495
Corners	0.257	0.168
$Corners_{Stiff}$	0.108	0.208
Bottom Flange	1.377	1.495
Web	1.275	0.755
Web_{Stiff}	0.564	0.200
Top Flange	3.320	0.094

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.377	1.495	2.058	3.076	--
Bottom Corner	12	3.079	1.342	4.131	5.544	0.006
Web	12	15.296	0.755	11.544	8.713	1.884
Top Corner	12	3.079	0.168	0.516	0.086	0.006
Top Flange	6	19.920	0.094	1.866	0.175	--
Bottom Flange	5	6.884	1.495	10.289	15.380	--
$Low Corner_{Stiff}$	0	0.000	1.302	0.000	0.000	0.000
Web_{Stiff}	0	0.000	1.310	0.000	0.000	0.000
$High Corner_{Stiff}$	0	0.000	0.902	0.000	0.000	0.000
	Σ	49.633		30.405	32.975	1.895

Solved ȳ =	ΣLy/ΣL =	0.613	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.897	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	0.479	$in.^4$
S_e =	$I_x/ȳ$ =	0.534	$in.^3$
Mn =	**Se*Fy =**	**17.63**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)	bo:	3.144	in.
Gage:	22	GA	h:	1.275	in.
Strength:	40	ksi	bp:	0.895	in.
Thickness:	0.0295	in.	Ag:	0.098	$in.^2$
Total Height:	1.510	in.	n:	1	
Radius:	0.20275	in.			
θ:	72.5	deg	$θ_{Stiff}$:	30.625	deg
θ:	1.265	rad	$θ_{Stiff}$:	0.535	rad
Curve I'_x:	0.000477	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000008	$in.^3$

Guess ȳ: 0.613 in.

Stress in Flange:	27.320	ksi	δ:	1.056	
k:	25.794		Isp:	0.002	$in.^4$
Fcr:	60.558	ksi	γ:	253.018	
λ:	0.672		β:	4.745	
ρ:	1.000		kd:	15.113	
Effective Width:	3.320	in.	kloc:	49.309	
			R:	1.707	

Element	L (in.)	y from top (in.)
Lip	0.688	1.495
Corners	0.257	0.168
$Corners_{Stiff}$	0.108	0.208
Bottom Flange	1.377	1.495
Web	1.275	0.755
Web_{Stiff}	0.564	0.200
Top Flange	3.320	0.094

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.377	1.495	2.058	3.076	--
Bottom Corner	12	3.079	1.342	4.131	5.544	0.006
Web	12	15.296	0.755	11.544	8.713	1.884
Top Corner	12	3.079	0.168	0.516	0.086	0.006
Top Flange	6	19.920	0.094	1.866	0.175	--
Bottom Flange	5	6.884	1.495	10.289	15.380	--
$Low Corner_{Stiff}$	0	0.000	1.302	0.000	0.000	0.000
Web_{Stiff}	0	0.000	1.310	0.000	0.000	0.000
$High Corner_{Stiff}$	0	0.000	0.902	0.000	0.000	0.000
	Σ	49.633		30.405	32.975	1.895

Solved ȳ =	ΣLy/ΣL =	0.613	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.897	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	0.479	$in.^4$
S_e =	$I_x/ȳ$ =	0.534	$in.^3$
Mn =	**Se*Fy =**	**21.37**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)	bo:	3.144	in.
Gage:	22	GA	h:	1.275	in.
Strength:	50	ksi	bp:	0.895	in.
Thickness:	0.0295	in.	Ag:	0.098	$in.^2$
Total Height:	1.510	in.	n:	1	
Radius:	0.20275	in.			
θ:	72.5	deg	$θ_{Stiff}$:	30.625	deg
θ:	1.265	rad	$θ_{Stiff}$:	0.535	rad
Curve I'_x:	0.000477	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000008	$in.^3$

Guess ȳ: 0.628 in.

Stress in Flange:	35.578	ksi	δ:	1.056	
k:	25.794		Isp:	0.002	$in.^4$
Fcr:	60.558	ksi	γ:	253.018	
λ:	0.766		β:	4.745	
ρ:	0.930		kd:	15.113	
Effective Width:	3.088	in.	kloc:	49.309	
			R:	1.707	

Element	L (in.)	y from top (in.)
Lip	0.688	1.495
Corners	0.257	0.168
$Corners_{Stiff}$	0.108	0.208
Bottom Flange	1.377	1.495
Web	1.275	0.755
Web_{Stiff}	0.564	0.200
Top Flange	3.088	0.094

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.377	1.495	2.058	3.076	--
Bottom Corner	12	3.079	1.342	4.131	5.544	0.006
Web	12	15.296	0.755	11.544	8.713	1.884
Top Corner	12	3.079	0.168	0.516	0.086	0.006
Top Flange	6	18.529	0.094	1.736	0.163	--
Bottom Flange	5	6.884	1.495	10.289	15.380	--
Low $Corner_{Stiff}$	0	0.000	1.302	0.000	0.000	0.000
Web_{Stiff}	0	0.000	1.310	0.000	0.000	0.000
High $Corner_{Stiff}$	0	0.000	0.902	0.000	0.000	0.000
	Σ	48.243		30.275	32.962	1.895

Solved ȳ =	ΣLy/ΣL =	0.628	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.882	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	0.468	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.530	$in.^3$
Mn =	**Se*Fy =**	**26.52**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)	bo:	3.144	in.
Gage:	22	GA	h:	1.275	in.
Strength:	60	ksi	bp:	0.895	in.
Thickness:	0.0295	in.	Ag:	0.098	$in.^2$
Total Height:	1.510	in.	n:	1	
Radius:	0.20275	in.			
θ:	72.5	deg	$θ_{Stiff}$:	30.625	deg
θ:	1.265	rad	$θ_{Stiff}$:	0.535	rad
Curve I'_x:	0.000477	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000008	$in.^3$

Guess ȳ: 0.642 in.

Stress in Flange:	44.356	ksi	δ:	1.056	
k:	25.794		Isp:	0.002	$in.^4$
Fcr:	60.558	ksi	γ:	253.018	
λ:	0.856		β:	4.745	
ρ:	0.868		kd:	15.113	
Effective Width:	2.882	in.	kloc:	49.309	
			R:	1.707	

Element	L (in.)	y from top (in.)
Lip	0.688	1.495
Corners	0.257	0.168
$Corners_{Stiff}$	0.108	0.208
Bottom Flange	1.377	1.495
Web	1.275	0.755
Web_{Stiff}	0.564	0.200
Top Flange	2.882	0.094

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.377	1.495	2.058	3.076	--
Bottom Corner	12	3.079	1.342	4.131	5.544	0.006
Web	12	15.296	0.755	11.544	8.713	1.884
Top Corner	12	3.079	0.168	0.516	0.086	0.006
Top Flange	6	17.292	0.094	1.620	0.152	--
Bottom Flange	5	6.884	1.495	10.289	15.380	--
Low $Corner_{Stiff}$	0	0.000	1.302	0.000	0.000	0.000
Web_{Stiff}	0	0.000	1.310	0.000	0.000	0.000
High $Corner_{Stiff}$	0	0.000	0.902	0.000	0.000	0.000
	Σ	47.006		30.159	32.951	1.895

Solved ȳ =	ΣLy/ΣL =	0.642	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.868	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	0.457	$in.^4$
S_e =	$I_x/ȳ$ =	0.527	$in.^3$
Mn =	**Se*Fy =**	**31.60**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)	bo:	3.144	in.
Gage:	24	GA	h:	1.275	in.
Strength:	33	ksi	bp:	0.895	in.
Thickness:	0.0238	in.	Ag:	0.079	$in.^2$
Total Height:	1.504	in.	n:	1	
Radius:	0.1999	in.			
θ:	72.5	deg	θ_{Stiff}:	30.625	deg
θ:	1.265	rad	θ_{Stiff}:	0.535	rad
Curve I'_x:	0.000457	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000007	$in.^3$

Guess ȳ: 0.610 in.

Stress in Flange:	22.494	ksi	δ:	1.056	
k:	32.413		Isp:	0.002	$in.^4$
Fcr:	49.531	ksi	γ:	407.100	
λ:	0.674		β:	5.343	
ρ:	0.999		kd:	18.991	
Effective Width:	3.318	in.	kloc:	49.309	
			R:	1.707	

Element	L (in.)	y from top (in.)
Lip	0.688	1.492
Corners	0.253	0.163
$Corners_{Stiff}$	0.107	0.202
Bottom Flange	1.377	1.492
Web	1.275	0.752
Web_{Stiff}	0.564	0.197
Top Flange	3.318	0.091

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	ΣLy^2 ($in.^3$)	$\Sigma I'_x$ ($in.^3$)
Lip	2	1.377	1.492	2.054	3.064	--
Bottom Corner	12	3.035	1.341	4.071	5.460	0.005
Web	12	15.296	0.752	11.501	8.647	1.884
Top Corner	12	3.035	0.163	0.493	0.080	0.005
Top Flange	6	19.909	0.091	1.806	0.164	--
Bottom Flange	5	6.884	1.492	10.270	15.322	--
Low $Corner_{Stiff}$	0	0.000	1.301	0.000	0.000	0.000
Web_{Stiff}	0	0.000	1.307	0.000	0.000	0.000
High $Corner_{Stiff}$	0	0.000	0.908	0.000	0.000	0.000
	Σ	49.536		30.195	32.738	1.895

Solved ȳ =	ΣLy/ΣL =	0.610	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.894	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2\Sigma L]t$ =	0.386	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.432	$in.^3$
Mn =	**Se*Fy =**	**14.25**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)	bo:	3.144	in.
Gage:	24	GA	h:	1.275	in.
Strength:	40	ksi	bp:	0.895	in.
Thickness:	0.0238	in.	Ag:	0.079	$in.^2$
Total Height:	1.504	in.	n:	1	
Radius:	0.1999	in.			
θ:	72.5	deg	$θ_{Stiff}$:	30.625	deg
θ:	1.265	rad	$θ_{Stiff}$:	0.535	rad
Curve I'_x:	0.000457	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000007	$in.^3$

Guess ȳ: 0.623 in.

Stress in Flange:	28.265	ksi	δ:	1.056	
k:	32.413		Isp:	0.002	$in.^4$
Fcr:	49.531	ksi	γ:	407.100	
λ:	0.755		β:	5.343	
ρ:	0.938		kd:	18.991	
Effective Width:	3.115	in.	kloc:	49.309	
			R:	1.707	

Element	L (in.)	y from top (in.)
Lip	0.688	1.492
Corners	0.253	0.163
$Corners_{Stiff}$	0.107	0.202
Bottom Flange	1.377	1.492
Web	1.275	0.752
Web_{Stiff}	0.564	0.197
Top Flange	3.115	0.091

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.377	1.492	2.054	3.064	--
Bottom Corner	12	3.035	1.341	4.071	5.460	0.005
Web	12	15.296	0.752	11.501	8.647	1.884
Top Corner	12	3.035	0.163	0.493	0.080	0.005
Top Flange	6	18.690	0.091	1.695	0.154	--
Bottom Flange	5	6.884	1.492	10.270	15.322	--
Low $Corner_{Stiff}$	0	0.000	1.301	0.000	0.000	0.000
Web_{Stiff}	0	0.000	1.307	0.000	0.000	0.000
High $Corner_{Stiff}$	0	0.000	0.908	0.000	0.000	0.000
	Σ	48.317		30.084	32.728	1.895

Solved ȳ =	ΣLy/ΣL =	0.623	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.881	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	0.378	$in.^4$
S_e =	$I_x/ȳ$ =	0.429	$in.^3$
Mn =	**Se*Fy =**	**17.17**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)	bo:	3.144	in.
Gage:	24	GA	h:	1.275	in.
Strength:	50	ksi	bp:	0.895	in.
Thickness:	0.0238	in.	Ag:	0.079	$in.^2$
Total Height:	1.504	in.	n:	1	
Radius:	0.1999	in.			
θ:	72.5	deg	$θ_{Stiff}$:	30.625	deg
θ:	1.265	rad	$θ_{Stiff}$:	0.535	rad
Curve I'_x:	0.000457	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000007	$in.^3$

Guess ȳ: 0.640 in.

Stress in Flange:	37.030	ksi	δ:	1.056	
k:	32.413		Isp:	0.002	$in.^4$
Fcr:	49.531	ksi	γ:	407.100	
λ:	0.865		β:	5.343	
ρ:	0.862		kd:	18.991	
Effective Width:	2.863	in.	kloc:	49.309	
			R:	1.707	

Element	L (in.)	y from top (in.)
Lip	0.688	1.492
Corners	0.253	0.163
$Corners_{Stiff}$	0.107	0.202
Bottom Flange	1.377	1.492
Web	1.275	0.752
Web_{Stiff}	0.564	0.197
Top Flange	2.863	0.091

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.377	1.492	2.054	3.064	--
Bottom Corner	12	3.035	1.341	4.071	5.460	0.005
Web	12	15.296	0.752	11.501	8.647	1.884
Top Corner	12	3.035	0.163	0.493	0.080	0.005
Top Flange	6	17.176	0.091	1.558	0.141	--
Bottom Flange	5	6.884	1.492	10.270	15.322	--
Low $Corner_{Stiff}$	0	0.000	1.301	0.000	0.000	0.000
Web_{Stiff}	0	0.000	1.307	0.000	0.000	0.000
High $Corner_{Stiff}$	0	0.000	0.908	0.000	0.000	0.000
	Σ	46.803		29.947	32.715	1.895

Solved ȳ =	ΣLy/ΣL =	0.640	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.864	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	0.368	$in.^4$
S_e =	$I_x/ȳ$ =	0.426	$in.^3$
Mn =	**Se*Fy =**	**21.28**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)	bo:	3.144	in.
Gage:	24	GA	h:	1.275	in.
Strength:	60	ksi	bp:	0.895	in.
Thickness:	0.0238	in.	Ag:	0.079	$in.^2$
Total Height:	1.504	in.	n:	1	
Radius:	0.1999	in.			
θ:	72.5	deg	$θ_{Stiff}$:	30.625	deg
θ:	1.265	rad	$θ_{Stiff}$:	0.535	rad
Curve I'_x:	0.000457	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000007	$in.^3$

Guess ȳ: 0.655 in.

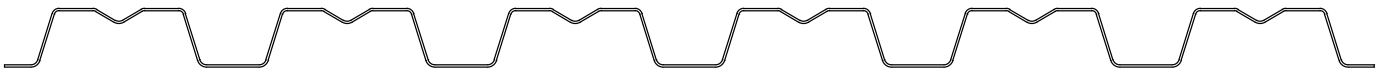
Stress in Flange:	46.309	ksi	δ:	1.056	
k:	32.413		Isp:	0.002	$in.^4$
Fcr:	49.531	ksi	γ:	407.100	
λ:	0.967		β:	5.343	
ρ:	0.799		kd:	18.991	
Effective Width:	2.652	in.	kloc:	49.309	
			R:	1.707	

Element	L (in.)	y from top (in.)
Lip	0.688	1.492
Corners	0.253	0.163
$Corners_{Stiff}$	0.107	0.202
Bottom Flange	1.377	1.492
Web	1.275	0.752
Web_{Stiff}	0.564	0.197
Top Flange	2.652	0.091

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	1.377	1.492	2.054	3.064	--
Bottom Corner	12	3.035	1.341	4.071	5.460	0.005
Web	12	15.296	0.752	11.501	8.647	1.884
Top Corner	12	3.035	0.163	0.493	0.080	0.005
Top Flange	6	15.914	0.091	1.443	0.131	--
Bottom Flange	5	6.884	1.492	10.270	15.322	--
Low $Corner_{Stiff}$	0	0.000	1.301	0.000	0.000	0.000
Web_{Stiff}	0	0.000	1.307	0.000	0.000	0.000
High $Corner_{Stiff}$	0	0.000	0.908	0.000	0.000	0.000
	Σ	45.541		29.832	32.705	1.895

Solved ȳ =	ΣLy/ΣL =	0.655	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.849	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	0.358	$in.^4$
S_e =	$I_x/ȳ$ =	0.422	$in.^3$
Mn =	**Se*Fy =**	**25.33**	**k-in.**

CHAPTER 7: ANALYSIS | 1.5B DECK (STIFFENED) | - BENDING



7.0 Executive Summary

The Direct Strength Method predicted higher strengths for each of the stiffened 1.5B Deck sections undergoing negative flexure. The compression flange is the lower unstiffened flange for this section and DSM is able to take advantage of the shorter compression flange width. The nominal moment capacity ratio (Mn_{DSM}/Mn_{EWM}) ranged between 1.037 and 1.157.

7.1 Mn_{DSM} / Mn_{EWM} vs. Thickness Plot

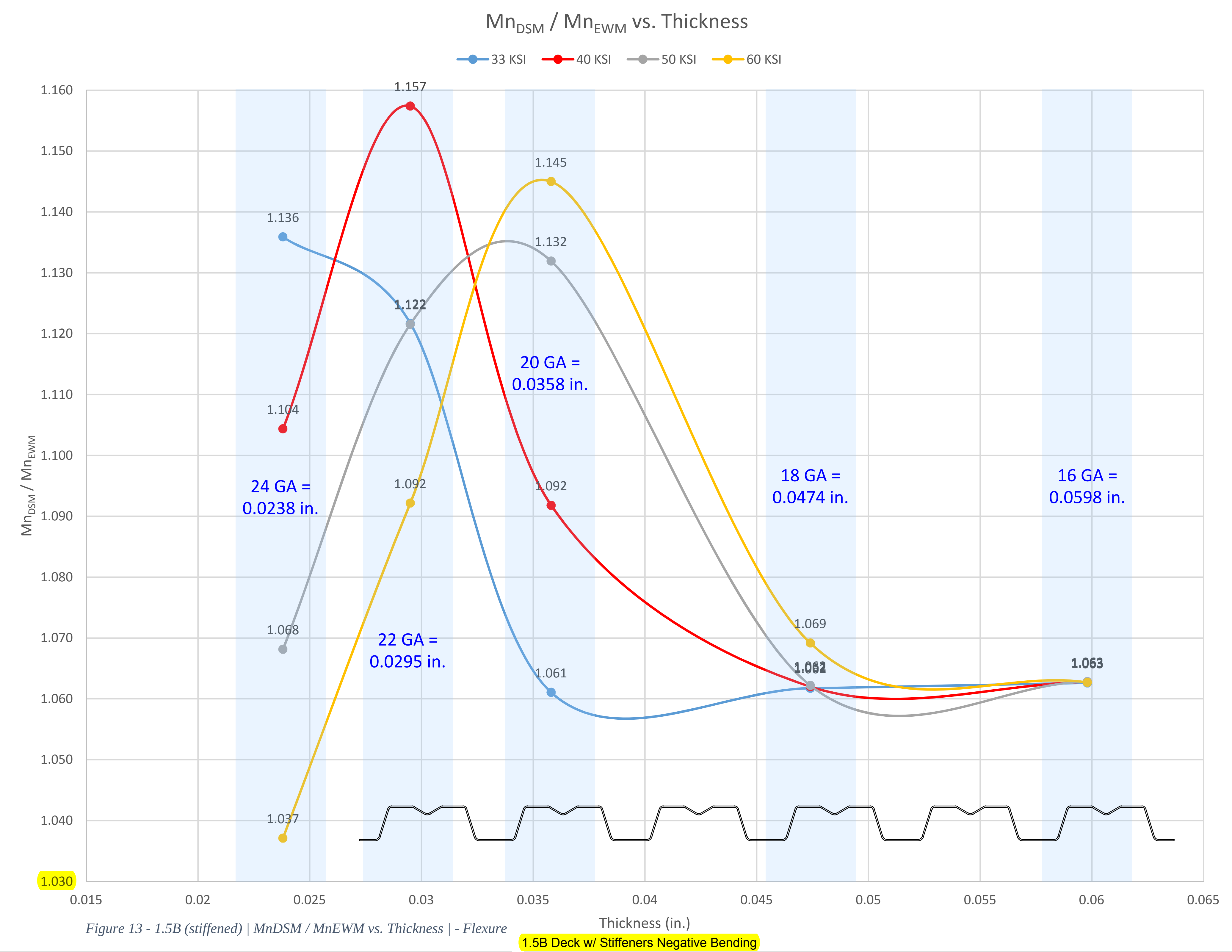


Figure 13 - 1.5B (stiffened) | MnDSM / MnEWM vs. Thickness | - Flexure

7.2 Analysis Results Summary

Table 7 - 1.5B (stiffened) | Analysis Results Summary | - Flexure

1.5B DECK (stiffeners) - 33 KSI					
Gage	**24**	**22**	**20**	**18**	**16**
Thickness	0.0238	0.0295	0.0358	0.0474	0.0598
Curve Radius	0.1999	0.2028	0.2059	0.2117	0.2179
I_G (CUFSM)	0.407	0.505	0.616	0.815	1.025
y-bar (CUFSM)	0.89116	0.89128	0.89289	0.89359	0.89359
Sxx	0.457	0.567	0.690	0.912	1.147
My	15.07	18.71	22.76	30.09	37.84
Mn_{DSM}	14.21	18.71	22.76	30.09	37.84
Mn_{EWM}	12.51	16.68	21.45	28.34	35.61
% ERROR	11.963%	10.850%	5.756%	5.816%	5.893%

1.5B DECK (stiffeners) - 33 KSI				
Thickness		Mn_{DSM}	Mn_{EWM}	Mn_{DSM} / Mn_{EWM}
24	0.0238	14.21	12.51	1.136
22	0.0295	18.71	16.68	1.122
20	0.0358	22.76	21.45	1.061
18	0.0474	30.09	28.34	1.062
16	0.0598	37.84	35.61	1.063

1.5B DECK (stiffeners) - 40 KSI					
Gage	**24**	**22**	**20**	**18**	**16**
Thickness	0.0238	0.0295	0.0358	0.0474	0.0598
Curve Radius	0.1999	0.2028	0.2059	0.2117	0.2179
I_G (CUFSM)	0.407	0.505	0.616	0.815	1.025
y-bar (CUFSM)	0.89116	0.89128	0.89289	0.89359	0.89359
Sxx	0.457	0.567	0.690	0.912	1.147
My	18.27	22.68	27.59	36.48	45.87
Mn_{DSM}	16.19	22.65	27.59	36.48	45.87
Mn_{EWM}	14.66	19.57	25.27	34.35	43.16
% ERROR	9.450%	13.598%	8.409%	5.839%	5.908%

1.5B DECK (stiffeners) - 40 KSI				
Thickness		Mn_{DSM}	Mn_{EWM}	Mn_{DSM} / Mn_{EWM}
24	0.0238	16.19	14.66	1.104
22	0.0295	22.65	19.57	1.157
20	0.0358	27.59	25.27	1.092
18	0.0474	36.48	34.35	1.062
16	0.0598	45.87	43.16	1.063

1.5B DECK (stiffeners) - 50 KSI					
Gage	**24**	**22**	**20**	**18**	**16**
Thickness	0.0238	0.0295	0.0358	0.0474	0.0598
Curve Radius	0.1999	0.2028	0.2059	0.2117	0.2179
I_G (CUFSM)	0.407	0.505	0.616	0.815	1.025
y-bar (CUFSM)	0.89116	0.89128	0.89289	0.89359	0.89359
Sxx	0.457	0.567	0.690	0.912	1.147
My	22.84	28.35	34.49	45.60	57.34
Mn_{DSM}	18.81	26.39	34.49	45.6	57.34
Mn_{EWM}	17.61	23.53	30.47	42.93	53.95
% ERROR	6.380%	10.837%	11.656%	5.855%	5.912%

1.5B DECK (stiffeners) - 50 KSI				
Thickness		Mn_{DSM}	Mn_{EWM}	Mn_{DSM} / Mn_{EWM}
24	0.0238	18.81	17.61	1.068
22	0.0295	26.39	23.53	1.122
20	0.0358	34.49	30.47	1.132
18	0.0474	45.60	42.93	1.062
16	0.0598	57.34	53.95	1.063

1.5B DECK (stiffeners) - 60 KSI					
Gage	**24**	**22**	**20**	**18**	**16**
Thickness	0.0238	0.0295	0.0358	0.0474	0.0598
Curve Radius	0.1999	0.2028	0.2059	0.2117	0.2179
I_G (CUFSM)	0.407	0.505	0.616	0.815	1.025
y-bar (CUFSM)	0.89116	0.89128	0.89289	0.89359	0.89359
Sxx	0.457	0.567	0.690	0.912	1.147
My	27.40	34.02	41.38	54.72	68.80
Mn_{DSM}	21.23	29.86	40.59	54.72	68.8
Mn_{EWM}	20.47	27.34	35.45	51.18	64.74
% ERROR	3.580%	8.439%	12.663%	6.469%	5.901%

1.5B DECK (stiffeners) - 60 KSI				
Thickness		Mn_{DSM}	Mn_{EWM}	Mn_{DSM} / Mn_{EWM}
24	0.0238	21.23	20.47	1.037
22	0.0295	29.86	27.34	1.092
20	0.0358	40.59	35.45	1.145
18	0.0474	54.72	51.18	1.069
16	0.0598	68.80	64.74	1.063

7.3 Direct Strength Method Calculations

Date:	1/11/2015	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)					
	Gage:	16	GA					
	Strength:	-33	KSI					
	M_y =	37.84	kip-in				Length:	
local	$M_{cr\ell}/M_y$ =	7.43230	$M_{cr\ell}$ =	281.2382	kip-in		1	in
dist.	M_{crd}/M_y =	27.74000	M_{crd} =	1049.682	kip-in		12	in
global	M_{cre}/M_y =	50.00000	M_{cre} =	1892	kip-in		-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 37.84 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.37 (local-global slenderness)

$M_{n\ell}$ = 37.84 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.19 (distortional slenderness)

M_{nd} = 37.84 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 37.84 kip-in (local-global controls)

Date: 1/11/2015 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)			
	Gage:	16	GA			
	Strength:	-40	KSI			
	$M_y =$	45.87	kip-in		Length:	
local	$M_{cr\ell}/M_y =$	6.13170	$M_{cr\ell} =$ 281.2611 kip-in		1	in
dist.	$M_{crd}/M_y =$	22.89000	$M_{crd} =$ 1049.964 kip-in		12	in
global	$M_{cre}/M_y =$	50.00000	$M_{cre} =$ 2293.5 kip-in		-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 45.87 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

$\lambda_\ell =$ 0.40 (local-global slenderness)

$M_{n\ell}$ = 45.87 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

$\lambda_d =$ 0.21 (distortional slenderness)

M_{nd} = 45.87 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 45.87 kip-in (local-global controls)

Date: 1/11/2015 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)				
	Gage:	16	GA				
	Strength:	-50	KSI				
	M_y =	57.34	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	4.90530	$M_{cr\ell}$ =	281.2699	kip-in	1	in
dist.	M_{crd}/M_y =	18.31000	M_{crd} =	1049.895	kip-in	12	in
global	M_{cre}/M_y =	50.00000	M_{cre} =	2867	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 57.34 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.45 (local-global slenderness)

$M_{n\ell}$ = 57.34 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.23 (distortional slenderness)

M_{nd} = 57.34 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 57.34 kip-in (local-global controls)

Date:	1/11/2015	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)					
	Gage:	16	GA					
	Strength:	-60	KSI					
	M_y =	68.80	kip-in				Length:	
local	$M_{cr\ell}/M_y$ =	4.08780	$M_{cr\ell}$ =	281.2406	kip-in		1	in
dist.	M_{crd}/M_y =	15.26000	M_{crd} =	1049.888	kip-in		12	in
global	M_{cre}/M_y =	50.00000	M_{cre} =	3440	kip-in		-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 68.80 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.49 (local-global slenderness)

$M_{n\ell}$ = 68.80 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.26 (distortional slenderness)

M_{nd} = 68.80 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 68.80 kip-in (local-global controls)

Date:	1/11/2015		calc by:	RKD
			check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)				
	Gage:	18	GA				
	Strength:	-33	KSI				
	$M_y =$	30.09	kip-in			Length:	
local	$M_{cr\ell}/M_y =$	4.78110	$M_{cr\ell} =$	143.8633	kip-in	1	in
dist.	$M_{crd}/M_y =$	22.85000	$M_{crd} =$	687.5565	kip-in	12	in
global	$M_{cre}/M_y =$	50.00000	$M_{cre} =$	1504.5	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq. 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq. 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq. 1.2.2\text{-}3)$$

M_{ne} = 30.09 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq. 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq. 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (*Eq.* 1.2.2-7)

$\lambda_\ell =$	0.46	(local-global slenderness)
$M_{n\ell}$ =	**30.09** kip-in	(fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq. 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq. 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (*Eq.* 1.2.2-10)

$\lambda_d =$	0.21	(distortional slenderness)
M_{nd} =	**30.09** kip-in	(fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 30.09 kip-in (local-global controls)

Date:	1/11/2015	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)				
	Gage:	18	GA				
	Strength:	-40	KSI				
	$M_y =$	36.48	kip-in			Length:	
local	$M_{cr\ell}/M_y =$	3.94440	$M_{cr\ell} =$	143.8917	kip-in	1	in
dist.	$M_{crd}/M_y =$	18.85000	$M_{crd} =$	687.648	kip-in	12	in
global	$M_{cre}/M_y =$	50.00000	$M_{cre} =$	1824	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 36.48 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (*Eq.* 1.2.2-7)

λ_ℓ = 0.50 (local-global slenderness)

$M_{n\ell}$ = 36.48 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (*Eq.* 1.2.2-10)

λ_d = 0.23 (distortional slenderness)

M_{nd} = 36.48 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 36.48 kip-in (local-global controls)

Date: 1/11/2015 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)				
	Gage:	18	GA				
	Strength:	-50	KSI				
	M_y =	45.60	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	3.15560	$M_{cr\ell}$ =	143.8954	kip-in	1	in
dist.	M_{crd}/M_y =	15.08000	M_{crd} =	687.648	kip-in	12	in
global	M_{cre}/M_y =	50.00000	M_{cre} =	2280	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 45.60 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (*Eq.* 1.2.2-7)

λ_ℓ = 0.56 (local-global slenderness)

$M_{n\ell}$ = 45.60 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (*Eq.* 1.2.2-10)

λ_d = 0.26 (distortional slenderness)

M_{nd} = 45.60 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 45.60 kip-in (local-global controls)

Date: 1/11/2015 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)				
	Gage:	18	GA				
	Strength:	-60	KSI				
	M_y =	54.72	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	2.62960	$M_{cr\ell}$ =	143.8917	kip-in	1	in
dist.	M_{crd}/M_y =	12.57000	M_{crd} =	687.8304	kip-in	12	in
global	M_{cre}/M_y =	50.00000	M_{cre} =	2736	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1-\frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 54.72 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell}=M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell}=\left(1-0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ $\qquad (Eq.\ 1.2.2\text{-}7)$

λ_ℓ = 0.62 (local-global slenderness)

$M_{n\ell}$ = 54.72 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1-0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5}M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ $\qquad (Eq.\ 1.2.2\text{-}10)$

λ_d = 0.28 (distortional slenderness)

M_{nd} = 54.72 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 54.72 kip-in (local-global controls)

Date:	1/11/2015	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)				
	Gage:	20	GA				
	Strength:	-33	KSI				
	M_y =	22.76	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	2.83930	$M_{cr\ell}$ =	64.62247	kip-in	1	in
dist.	M_{crd}/M_y =	19.26000	M_{crd} =	438.3576	kip-in	12	in
global	M_{cre}/M_y =	50.00000	M_{cre} =	1138	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 22.76 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (*Eq.* 1.2.2-7)

λ_ℓ = 0.59 (local-global slenderness)

$M_{n\ell}$ = 22.76 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (*Eq.* 1.2.2-10)

λ_d = 0.23 (distortional slenderness)

M_{nd} = 22.76 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 22.76 kip-in (local-global controls)

Date:	1/11/2015	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)					
	Gage:	20	GA					
	Strength:	-40	KSI					
	$M_y =$	27.59	kip-in				Length:	
local	$M_{cr\ell}/M_y =$	2.34250	$M_{cr\ell} =$	64.62958	kip-in		1	in
dist.	$M_{crd}/M_y =$	15.89000	$M_{crd} =$	438.4051	kip-in		12	in
global	$M_{cre}/M_y =$	50.00000	$M_{cre} =$	1379.5	kip-in		-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **27.59** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

$\lambda_\ell =$ 0.65 (local-global slenderness)

$M_{n\ell}$ = **27.59** kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

$\lambda_d =$ 0.25 (distortional slenderness)

M_{nd} = **27.59** kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 27.59 kip-in (local-global controls)

Date:	1/11/2015		calc by:	RKD
			check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:								
	Deck:	1.5B	(stiffeners)					
	Gage:	20	GA					
	Strength:	-50	KSI					
	M_y =	34.49	kip-in				Length:	
local	$M_{cr\ell}/M_y$ =	1.87400	$M_{cr\ell}$ =	64.63426	kip-in		1	in
dist.	M_{crd}/M_y =	12.71000	M_{crd} =	438.3679	kip-in		12	in
global	M_{cre}/M_y =	50.00000	M_{cre} =	1724.5	kip-in		-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 34.49 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.73 (local-global slenderness)

$M_{n\ell}$ = 34.49 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.28 (distortional slenderness)

M_{nd} = 34.49 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 34.49 kip-in (local-global controls)

Date: 1/11/2015 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)			
	Gage:	20	GA			
	Strength:	-60	KSI			
	M_y =	41.38	kip-in		Length:	
local	$M_{cr\ell}/M_y$ =	1.56170	$M_{cr\ell}$ =	64.62315 kip-in	1	in
dist.	M_{crd}/M_y =	10.59000	M_{crd} =	438.2142 kip-in	12	in
global	M_{cre}/M_y =	50.00000	M_{cre} =	2069 kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 41.38 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.80 (local-global slenderness)

$M_{n\ell}$ = 40.59 kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.31 (distortional slenderness)

M_{nd} = 41.38 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 40.59 kip-in (local-global controls)

Date: 1/11/2015 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)			
	Gage:	22	GA			
	Strength:	-33	KSI			
	M_y =	18.71	kip-in		Length:	
local	$M_{cr\ell}/M_y$ =	2.00510	$M_{cr\ell}$ =	37.51542 kip-in	1	in
dist.	M_{crd}/M_y =	16.27000	M_{crd} =	304.4117 kip-in	24	in
global	M_{cre}/M_y =	50.00000	M_{cre} =	935.5 kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1-\frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 18.71 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell}=M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell}=\left(1-0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (*Eq.* 1.2.2-7)

λ_ℓ = 0.71 (local-global slenderness)

$M_{n\ell}$ = 18.71 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1-0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5}M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (*Eq.* 1.2.2-10)

λ_d = 0.25 (distortional slenderness)

M_{nd} = 18.71 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 18.71 kip-in (local-global controls)

Date:	1/11/2015		calc by:	RKD
			check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)			
	Gage:	22	GA			
	Strength:	-40	KSI			
	M_y =	22.68	kip-in		Length:	
local	$M_{cr\ell}/M_y$ =	1.65420	$M_{cr\ell}$ =	37.51726 kip-in	1	in
dist.	M_{crd}/M_y =	13.43000	M_{crd} =	304.5924 kip-in	24	in
global	M_{cre}/M_y =	50.00000	M_{cre} =	1134 kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1-\frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 22.68 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell}=M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell}=\left(1-0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (*Eq.* 1.2.2-7)

λ_ℓ = 0.78 (local-global slenderness)

$M_{n\ell}$ = 22.65 kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1-0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5}M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (*Eq.* 1.2.2-10)

λ_d = 0.27 (distortional slenderness)

M_{nd} = 22.68 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 22.65 kip-in (local-global controls)

Date:	1/11/2015	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)				
	Gage:	22	GA				
	Strength:	-50	KSI				
	M_y =	28.35	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	1.32340	$M_{cr\ell}$ =	37.51839	kip-in	1	in
dist.	M_{crd}/M_y =	10.74000	M_{crd} =	304.479	kip-in	24	in
global	M_{cre}/M_y =	50.00000	M_{cre} =	1417.5	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 28.35 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.87 (local-global slenderness)

$M_{n\ell}$ = 26.39 kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.31 (distortional slenderness)

M_{nd} = 28.35 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 26.39 kip-in (local-global controls)

Date: 1/11/2015 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)			
	Gage:	22	GA			
	Strength:	-60	KSI			
	M_y =	34.02	kip-in		Length:	
local	$M_{cr\ell}/M_y$ =	1.10280	$M_{cr\ell}$ = 37.51726	kip-in	1	in
dist.	M_{crd}/M_y =	8.95000	M_{crd} = 304.479	kip-in	24	in
global	M_{cre}/M_y =	50.00000	M_{cre} = 1701	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 34.02 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.95 (local-global slenderness)

$M_{n\ell}$ = 29.86 kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.33 (distortional slenderness)

M_{nd} = 34.02 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 29.86 kip-in (local-global controls)

Date:	1/11/2015	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)				
	Gage:	24	GA				
	Strength:	-33	KSI				
	M_y =	15.07	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	1.37710	$M_{cr\ell}$ =	20.7529	kip-in	1	in
dist.	M_{crd}/M_y =	12.01000	M_{crd} =	180.9907	kip-in	24	in
global	M_{cre}/M_y =	50.00000	M_{cre} =	753.5	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 15.07 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.85 (local-global slenderness)

$M_{n\ell}$ = 14.21 kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.29 (distortional slenderness)

M_{nd} = 15.07 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 14.21 kip-in (local-global controls)

Date: 1/11/2015 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)				
	Gage:	24	GA				
	Strength:	-40	KSI				
	M_y =	18.27	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	1.13610	$M_{cr\ell}$ =	20.75655	kip-in	1	in
dist.	M_{crd}/M_y =	9.91000	M_{crd} =	181.0557	kip-in	24	in
global	M_{cre}/M_y =	50.00000	M_{cre} =	913.5	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 18.27 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (*Eq.* 1.2.2-7)

λ_ℓ = 0.94 (local-global slenderness)

$M_{n\ell}$ = 16.19 kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (*Eq.* 1.2.2-10)

λ_d = 0.32 (distortional slenderness)

M_{nd} = 18.27 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 16.19 kip-in (local-global controls)

Date:	1/11/2015	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)				
	Gage:	24	GA				
	Strength:	-50	KSI				
	M_y =	22.84	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	0.90890	$M_{cr\ell}$ =	20.75928	kip-in	1	in
dist.	M_{crd}/M_y =	7.92000	M_{crd} =	180.8928	kip-in	24	in
global	M_{cre}/M_y =	50.00000	M_{cre} =	1142	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 22.84 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 1.05 (local-global slenderness)

$M_{n\ell}$ = 18.81 kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.36 (distortional slenderness)

M_{nd} = 22.84 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 18.81 kip-in (local-global controls)

Date:	1/11/2015			calc by:	RKD
				check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	1.5B	(stiffeners)				
	Gage:	24	GA				
	Strength:	-60	KSI				
	$M_y =$	27.40	kip-in			Length:	
local	$M_{cr\ell}/M_y =$	0.75742	$M_{cr\ell} =$	20.75331	kip-in	1	in
dist.	$M_{crd}/M_y =$	6.60000	$M_{crd} =$	180.84	kip-in	24	in
global	$M_{cre}/M_y =$	50.00000	$M_{cre} =$	1370	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 27.40 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

$\lambda_\ell =$ 1.15 (local-global slenderness)

$M_{n\ell}$ = 21.23 kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

$\lambda_d =$ 0.39 (distortional slenderness)

M_{nd} = 27.40 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 21.23 kip-in (local-global controls)

7.4 Effective Width Method Calculations

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)			
Gage:	16	GA			
Strength:	33	ksi			
Thickness:	0.0598	in.			
Total Height:	1.540	in.	Comp Flange:	1.377	in
Radius:	0.2179	in.			
θ:	72.5	deg	θ_{Stiff}:	30.625	deg
θ:	1.265	rad	θ_{Stiff}:	0.535	rad
Curve I'_x:	0.000592	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000010	$in.^3$

Guess ȳ: 0.898 in.

Stress in Flange:	33.000	ksi
k:	4.000	
Fcr:	201.212	ksi
λ:	0.405	
ρ:	1.128	
Effective Width:	1.377	in.

Element	L (in.)	y from top (in.)
Lip	0.688	0.030
Corners	0.276	0.194
$Corners_{Stiff}$	0.116	0.238
Bottom Flange	1.377	0.030
Web	1.275	0.770
Web_{Stiff}	0.564	0.215
Top Flange	0.895	1.510

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	ΣLy^2 ($in.^3$)	$\Sigma I'_x$ ($in.^3$)
Lip	0	0.000	0.030	0.000	0.000	--
Bottom Corner	12	3.309	0.194	0.642	0.125	0.007
Web	12	15.296	0.770	11.776	9.066	1.884
Top Corner	12	3.309	1.346	4.452	5.991	0.007
Top Flange	12	10.745	1.510	16.224	24.496	--
Bottom Flange	6	8.260	0.030	0.247	0.007	--
High $Corner_{Stiff}$	12	1.398	1.302	1.820	2.370	0.000
Web_{Stiff}	12	6.762	1.325	8.959	11.870	0.046
Low $Corner_{Stiff}$	12	1.398	0.872	1.219	1.064	0.000
	Σ	50.475		45.340	54.989	1.944

Solved ȳ =	ΣLy/ΣL =	0.898	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.898	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2\Sigma L]t$ =	0.969	$in.^4$
S_e =	$I_x/\bar{y}$ =	1.079	$in.^3$
Mn =	**Se*Fy =**	**35.61**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)			
Gage:	16	GA			
Strength:	40	ksi			
Thickness:	0.0598	in.			
Total Height:	1.540	in.	Comp Flange:	1.377	in
Radius:	0.2179	in.			
θ:	72.5	deg	θ_{Stiff}:	30.625	deg
θ:	1.265	rad	θ_{Stiff}:	0.535	rad
Curve I'_x:	0.000592	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000010	$in.^3$

Guess $\bar{y}$: 0.898 in.

Stress in Flange:	40.000	ksi
k:	4.000	
Fcr:	201.212	ksi
λ:	0.446	
ρ:	1.136	
Effective Width:	1.377	in.

Element	L (in.)	y from top (in.)
Lip	0.688	0.030
Corners	0.276	0.194
$Corners_{Stiff}$	0.116	0.238
Bottom Flange	1.377	0.030
Web	1.275	0.770
Web_{Stiff}	0.564	0.215
Top Flange	0.895	1.510

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	ΣLy^2 ($in.^3$)	$\Sigma I'_x$ ($in.^3$)
Lip	0	0.000	0.030	0.000	0.000	--
Bottom Corner	12	3.309	0.194	0.642	0.125	0.007
Web	12	15.296	0.770	11.776	9.066	1.884
Top Corner	12	3.309	1.346	4.452	5.991	0.007
Top Flange	12	10.745	1.510	16.224	24.496	--
Bottom Flange	6	8.260	0.030	0.247	0.007	--
High $Corner_{Stiff}$	12	1.398	1.302	1.820	2.370	0.000
Web_{Stiff}	12	6.762	1.325	8.959	11.870	0.046
Low $Corner_{Stiff}$	12	1.398	0.872	1.219	1.064	0.000
	Σ	50.475		45.340	54.989	1.944

Solved $\bar{y}$ =	$\Sigma Ly/\Sigma L$ =	0.898	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max($\bar{y}$, h - $\bar{y}$) =	0.898	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2 \Sigma L]t$ =	0.969	$in.^4$
S_e =	$I_x/\bar{y}$ =	1.079	$in.^3$
Mn =	**Se*Fy =**	**43.16**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)				
Gage:	16	GA				
Strength:	50	ksi				
Thickness:	0.0598	in.				
Total Height:	1.540	in.		Comp Flange:	1.377	in
Radius:	0.2179	in.				
θ:	72.5	deg		θ_{Stiff}:	30.625	deg
θ:	1.265	rad		θ_{Stiff}:	0.535	rad
Curve I'_x:	0.000592	$in.^3$		$Curve_{Stiff}$ I'_x:	0.000010	$in.^3$

Guess ȳ: 0.898 in.

Stress in Flange:	50.000	ksi
k:	4.000	
Fcr:	201.212	ksi
λ:	0.498	
ρ:	1.121	
Effective Width:	1.377	in.

Element	L (in.)	y from top (in.)
Lip	0.688	0.030
Corners	0.276	0.194
$Corners_{Stiff}$	0.116	0.238
Bottom Flange	1.377	0.030
Web	1.275	0.770
Web_{Stiff}	0.564	0.215
Top Flange	0.895	1.510

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	ΣLy^2 ($in.^3$)	$\Sigma I'_x$ ($in.^3$)
Lip	0	0.000	0.030	0.000	0.000	--
Bottom Corner	12	3.309	0.194	0.642	0.125	0.007
Web	12	15.296	0.770	11.776	9.066	1.884
Top Corner	12	3.309	1.346	4.452	5.991	0.007
Top Flange	12	10.745	1.510	16.224	24.496	--
Bottom Flange	6	8.260	0.030	0.247	0.007	--
High $Corner_{Stiff}$	12	1.398	1.302	1.820	2.370	0.000
Web_{Stiff}	12	6.762	1.325	8.959	11.870	0.046
Low $Corner_{Stiff}$	12	1.398	0.872	1.219	1.064	0.000
	Σ	50.475		45.340	54.989	1.944

Solved ȳ =	ΣLy/ΣL =	0.898	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.898	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2 \Sigma L]t$ =	0.969	$in.^4$
S_e =	$I_x/\bar{y}$ =	1.079	$in.^3$
Mn =	**Se*Fy =**	**53.95**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)			
Gage:	16	GA			
Strength:	60	ksi			
Thickness:	0.0598	in.			
Total Height:	1.540	in.	Comp Flange:	1.377	in
Radius:	0.2179	in.			
θ:	72.5	deg	$θ_{Stiff}$:	30.625	deg
θ:	1.265	rad	$θ_{Stiff}$:	0.535	rad
Curve I'_x:	0.000592	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000010	$in.^3$

Guess ȳ: 0.898 in.

Stress in Flange:	60.000	ksi
k:	4.000	
Fcr:	201.212	ksi
λ:	0.546	
ρ:	1.093	
Effective Width:	1.377	in.

Element	L (in.)	y from top (in.)
Lip	0.688	0.030
Corners	0.276	0.194
$Corners_{Stiff}$	0.116	0.238
Bottom Flange	1.377	0.030
Web	1.275	0.770
Web_{Stiff}	0.564	0.215
Top Flange	0.895	1.510

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	0	0.000	0.030	0.000	0.000	--
Bottom Corner	12	3.309	0.194	0.642	0.125	0.007
Web	12	15.296	0.770	11.776	9.066	1.884
Top Corner	12	3.309	1.346	4.452	5.991	0.007
Top Flange	12	10.745	1.510	16.224	24.496	--
Bottom Flange	6	8.260	0.030	0.247	0.007	--
High $Corner_{Stiff}$	12	1.398	1.302	1.820	2.370	0.000
Web_{Stiff}	12	6.762	1.325	8.959	11.870	0.046
Low $Corner_{Stiff}$	12	1.398	0.872	1.219	1.064	0.000
	Σ	50.475		45.340	54.989	1.944

Solved ȳ =	ΣLy/ΣL =	0.898	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.898	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	0.969	$in.^4$
S_e =	$I_x/ȳ$ =	1.079	$in.^3$
Mn =	**Se*Fy =**	**64.74**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)			
Gage:	18	GA			
Strength:	33	ksi			
Thickness:	0.0474	in.			
Total Height:	1.527	in.	Comp Flange:	1.377	in
Radius:	0.2117	in.			
θ:	72.5	deg	θ_{Stiff}:	30.625	deg
θ:	1.265	rad	θ_{Stiff}:	0.535	rad
Curve I'_x:	0.000543	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000009	$in.^3$

Guess ȳ: 0.893 in.

Stress in Flange:	33.000	ksi
k:	4.000	
Fcr:	126.418	ksi
λ:	0.511	
ρ:	1.114	
Effective Width:	1.377	in.

Element	L (in.)	y from top (in.)
Lip	0.688	0.024
Corners	0.268	0.183
$Corners_{Stiff}$	0.113	0.225
Bottom Flange	1.377	0.024
Web	1.275	0.764
Web_{Stiff}	0.564	0.209
Top Flange	0.895	1.504

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	0	0.000	0.024	0.000	0.000	--
Bottom Corner	12	3.215	0.183	0.589	0.108	0.007
Web	12	15.296	0.764	11.681	8.921	1.884
Top Corner	12	3.215	1.344	4.321	5.808	0.007
Top Flange	12	10.745	1.504	16.157	24.295	--
Bottom Flange	6	8.260	0.024	0.196	0.005	--
High $Corner_{Stiff}$	12	1.358	1.302	1.768	2.302	0.000
Web_{Stiff}	12	6.762	1.319	8.917	11.759	0.046
Low $Corner_{Stiff}$	12	1.358	0.885	1.201	1.062	0.000
	Σ	50.208		44.830	54.259	1.943

Solved ȳ =	ΣLy/ΣL =	0.893	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.893	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2 \Sigma L]t$ =	0.767	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.859	$in.^3$
Mn =	**Se*Fy =**	**28.34**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)			
Gage:	18	GA			
Strength:	40	ksi			
Thickness:	0.0474	in.			
Total Height:	1.527	in.	Comp Flange:	1.377	in
Radius:	0.2117	in.			
θ:	72.5	deg	θ_{Stiff}:	30.625	deg
θ:	1.265	rad	θ_{Stiff}:	0.535	rad
Curve I'_x:	0.000543	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000009	$in.^3$

Guess $\bar{y}$: 0.893 in.

Stress in Flange:	40.000	ksi
k:	4.000	
Fcr:	126.418	ksi
λ:	0.563	
ρ:	1.082	
Effective Width:	1.377	in.

Element	L (in.)	y from top (in.)
Lip	0.688	0.024
Corners	0.268	0.183
$Corners_{Stiff}$	0.113	0.225
Bottom Flange	1.377	0.024
Web	1.275	0.764
Web_{Stiff}	0.564	0.209
Top Flange	0.895	1.504

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	0	0.000	0.024	0.000	0.000	--
Bottom Corner	12	3.215	0.183	0.589	0.108	0.007
Web	12	15.296	0.764	11.681	8.921	1.884
Top Corner	12	3.215	1.344	4.321	5.808	0.007
Top Flange	12	10.745	1.504	16.157	24.295	--
Bottom Flange	6	8.260	0.024	0.196	0.005	--
High $Corner_{Stiff}$	12	1.358	1.302	1.768	2.302	0.000
Web_{Stiff}	12	6.762	1.319	8.917	11.759	0.046
Low $Corner_{Stiff}$	12	1.358	0.885	1.201	1.062	0.000
	Σ	50.208		44.830	54.259	1.943

Solved $\bar{y}$ =	ΣLy/ΣL =	0.893	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max($\bar{y}$, h - $\bar{y}$) =	0.893	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	0.767	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.859	$in.^3$
Mn =	**Se*Fy =**	**34.35**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)
Gage:	18	GA
Strength:	50	ksi
Thickness:	0.0474	in.
Total Height:	1.527	in.
Radius:	0.2117	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000543	$in.^3$

Comp Flange:	1.377	in
$θ_{Stiff}$:	30.625	deg
$θ_{Stiff}$:	0.535	rad
$Curve_{Stiff}$ I'_x:	0.000009	$in.^3$

Guess ȳ: 0.893 in.

Stress in Flange:	50.000	ksi
k:	4.000	
Fcr:	126.418	ksi
λ:	0.629	
ρ:	1.034	
Effective Width:	1.377	in.

Element	L (in.)	y from top (in.)
Lip	0.688	0.024
Corners	0.268	0.183
$Corners_{Stiff}$	0.113	0.225
Bottom Flange	1.377	0.024
Web	1.275	0.764
Web_{Stiff}	0.564	0.209
Top Flange	0.895	1.504

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	0	0.000	0.024	0.000	0.000	--
Bottom Corner	12	3.215	0.183	0.589	0.108	0.007
Web	12	15.296	0.764	11.681	8.921	1.884
Top Corner	12	3.215	1.344	4.321	5.808	0.007
Top Flange	12	10.745	1.504	16.157	24.295	--
Bottom Flange	6	8.260	0.024	0.196	0.005	--
High $Corner_{Stiff}$	12	1.358	1.302	1.768	2.302	0.000
Web_{Stiff}	12	6.762	1.319	8.917	11.759	0.046
Low $Corner_{Stiff}$	12	1.358	0.885	1.201	1.062	0.000
	Σ	50.208		44.830	54.259	1.943

Solved ȳ =	ΣLy/ΣL =	0.893	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.893	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	0.767	$in.^4$
S_e =	$I_x/ȳ$ =	0.859	$in.^3$
Mn =	**Se*Fy =**	**42.93**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)
Gage:	18	GA
Strength:	60	ksi
Thickness:	0.0474	in.
Total Height:	1.527	in.
Radius:	0.2117	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000543	$in.^3$

Comp Flange:	1.377	in
θ_{Stiff}:	30.625	deg
θ_{Stiff}:	0.535	rad
$Curve_{Stiff}$ I'_x:	0.000009	$in.^3$

Guess $\bar{y}$: 0.895 in.

Stress in Flange:	60.000	ksi
k:	4.000	
Fcr:	126.418	ksi
λ:	0.689	
ρ:	0.988	
Effective Width:	1.360	in.

Element	L (in.)	y from top (in.)
Lip	0.688	0.024
Corners	0.268	0.183
$Corners_{Stiff}$	0.113	0.225
Bottom Flange	1.360	0.024
Web	1.275	0.764
Web_{Stiff}	0.564	0.209
Top Flange	0.895	1.504

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	ΣLy^2 ($in.^3$)	$\Sigma I'_x$ ($in.^3$)
Lip	0	0.000	0.024	0.000	0.000	--
Bottom Corner	12	3.215	0.183	0.589	0.108	0.007
Web	12	15.296	0.764	11.681	8.921	1.884
Top Corner	12	3.215	1.344	4.321	5.808	0.007
Top Flange	12	10.745	1.504	16.157	24.295	--
Bottom Flange	6	8.161	0.024	0.193	0.005	--
High $Corner_{Stiff}$	12	1.358	1.302	1.768	2.302	0.000
Web_{Stiff}	12	6.762	1.319	8.917	11.759	0.046
Low $Corner_{Stiff}$	12	1.358	0.885	1.201	1.062	0.000
	Σ	50.108		44.827	54.259	1.943

Solved $\bar{y}$ =	ΣLy/ΣL =	0.895	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max($\bar{y}$, h - $\bar{y}$) =	0.895	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2 \Sigma L]t$ =	0.763	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.853	$in.^3$
Mn =	**Se*Fy =**	**51.18**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)			
Gage:	20	GA			
Strength:	33	ksi			
Thickness:	0.0358	in.			
Total Height:	1.516	in.	Comp Flange:	1.377	in
Radius:	0.2059	in.			
θ:	72.5	deg	θ_{Stiff}:	30.625	deg
θ:	1.265	rad	θ_{Stiff}:	0.535	rad
Curve I'_x:	0.000500	in.^{3}	$Curve_{Stiff}$ I'_x:	0.000008	in.^{3}

Guess $\bar{y}$: 0.888 in.

Stress in Flange:	33.000	ksi
k:	4.000	
Fcr:	72.114	ksi
λ:	0.676	
ρ:	0.998	
Effective Width:	1.373	in.

Element	L (in.)	y from top (in.)
Lip	0.688	0.018
Corners	0.261	0.173
$Corners_{Stiff}$	0.110	0.214
Bottom Flange	1.373	0.018
Web	1.275	0.758
Web_{Stiff}	0.564	0.203
Top Flange	0.895	1.498

Element	Quantity	ΣL	y from top fiber	ΣLy (in.^{2})	ΣLy^2 (in.^{3})	ΣI'$_x$ (in.^{3})
Lip	0	0.000	0.018	0.000	0.000	--
Bottom Corner	12	3.126	0.173	0.541	0.094	0.006
Web	12	15.296	0.758	11.592	8.786	1.884
Top Corner	12	3.126	1.343	4.198	5.637	0.006
Top Flange	12	10.745	1.498	16.095	24.108	--
Bottom Flange	6	8.240	0.018	0.147	0.003	--
High $Corner_{Stiff}$	12	1.321	1.302	1.719	2.238	0.000
Web_{Stiff}	12	6.762	1.313	8.878	11.656	0.046
Low $Corner_{Stiff}$	12	1.321	0.896	1.183	1.060	0.000
	Σ	49.936		44.354	53.580	1.942

Solved $\bar{y}$ =	ΣLy/ΣL =	0.888	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max($\bar{y}$, h - $\bar{y}$) =	0.888	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2 \Sigma L]t$ =	0.577	in.^{4}
S_e =	$I_x/\bar{y}$ =	0.650	in.^{3}
Mn =	**Se*Fy =**	**21.45**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)			
Gage:	20	GA			
Strength:	40	ksi			
Thickness:	0.0358	in.			
Total Height:	1.516	in.	Comp Flange:	1.377	in
Radius:	0.2059	in.			
θ:	72.5	deg	$θ_{Stiff}$:	30.625	deg
θ:	1.265	rad	$θ_{Stiff}$:	0.535	rad
Curve I'_x:	0.000500	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000008	$in.^3$

Guess $\bar{y}$: 0.896 in.

Stress in Flange:	40.000	ksi
k:	4.000	
Fcr:	72.114	ksi
λ:	0.745	
ρ:	0.946	
Effective Width:	1.303	in.

Element	L (in.)	y from top (in.)
Lip	0.688	0.018
Corners	0.261	0.173
$Corners_{Stiff}$	0.110	0.214
Bottom Flange	1.303	0.018
Web	1.275	0.758
Web_{Stiff}	0.564	0.203
Top Flange	0.895	1.498

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	0	0.000	0.018	0.000	0.000	--
Bottom Corner	12	3.126	0.173	0.541	0.094	0.006
Web	12	15.296	0.758	11.592	8.786	1.884
Top Corner	12	3.126	1.343	4.198	5.637	0.006
Top Flange	12	10.745	1.498	16.095	24.108	--
Bottom Flange	6	7.815	0.018	0.140	0.003	--
High $Corner_{Stiff}$	12	1.321	1.302	1.719	2.238	0.000
Web_{Stiff}	12	6.762	1.313	8.878	11.656	0.046
Low $Corner_{Stiff}$	12	1.321	0.896	1.183	1.060	0.000
	Σ	49.512		44.346	53.580	1.942

Solved $\bar{y}$ =	ΣLy/ΣL =	0.896	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max($\bar{y}$, h - $\bar{y}$) =	0.896	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	0.566	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.632	$in.^3$
Mn =	**Se*Fy =**	**25.27**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)			
Gage:	20	GA			
Strength:	50	ksi			
Thickness:	0.0358	in.			
Total Height:	1.516	in.	Comp Flange:	1.377	in
Radius:	0.2059	in.			
θ:	72.5	deg	θ_{Stiff}:	30.625	deg
θ:	1.265	rad	θ_{Stiff}:	0.535	rad
Curve I'_x:	0.000500	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000008	$in.^3$

Guess $\bar{y}$: 0.905 in.

Stress in Flange:	50.000	ksi
k:	4.000	
Fcr:	72.114	ksi
λ:	0.833	
ρ:	0.884	
Effective Width:	1.217	in.

Element	L (in.)	y from top (in.)
Lip	0.688	0.018
Corners	0.261	0.173
$Corners_{Stiff}$	0.110	0.214
Bottom Flange	1.217	0.018
Web	1.275	0.758
Web_{Stiff}	0.564	0.203
Top Flange	0.895	1.498

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	ΣLy^2 ($in.^3$)	$\Sigma I'_x$ ($in.^3$)
Lip	0	0.000	0.018	0.000	0.000	--
Bottom Corner	12	3.126	0.173	0.541	0.094	0.006
Web	12	15.296	0.758	11.592	8.786	1.884
Top Corner	12	3.126	1.343	4.198	5.637	0.006
Top Flange	12	10.745	1.498	16.095	24.108	--
Bottom Flange	6	7.299	0.018	0.131	0.002	--
High $Corner_{Stiff}$	12	1.321	1.302	1.719	2.238	0.000
Web_{Stiff}	12	6.762	1.313	8.878	11.656	0.046
Low $Corner_{Stiff}$	12	1.321	0.896	1.183	1.060	0.000
	Σ	48.996		44.337	53.580	1.942

Solved $\bar{y}$ =	ΣLy/ΣL =	0.905	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max($\bar{y}$, h - $\bar{y}$) =	0.905	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2 \Sigma L]t$ =	0.551	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.609	$in.^3$
Mn =	**Se*Fy =**	**30.47**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)			
Gage:	20	GA			
Strength:	60	ksi			
Thickness:	0.0358	in.			
Total Height:	1.516	in.	Comp Flange:	1.377	in
Radius:	0.2059	in.			
θ:	72.5	deg	θ_{Stiff}:	30.625	deg
θ:	1.265	rad	θ_{Stiff}:	0.535	rad
Curve I'_x:	0.000500	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000008	$in.^3$

Guess $\bar{y}$: 0.913 in.

Stress in Flange:	60.000	ksi
k:	4.000	
Fcr:	72.114	ksi
λ:	0.912	
ρ:	0.832	
Effective Width:	1.145	in.

Element	L (in.)	y from top (in.)
Lip	0.688	0.018
Corners	0.261	0.173
$Corners_{Stiff}$	0.110	0.214
Bottom Flange	1.145	0.018
Web	1.275	0.758
Web_{Stiff}	0.564	0.203
Top Flange	0.895	1.498

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	ΣLy^2 ($in.^3$)	$\Sigma I'_x$ ($in.^3$)
Lip	0	0.000	0.018	0.000	0.000	--
Bottom Corner	12	3.126	0.173	0.541	0.094	0.006
Web	12	15.296	0.758	11.592	8.786	1.884
Top Corner	12	3.126	1.343	4.198	5.637	0.006
Top Flange	12	10.745	1.498	16.095	24.108	--
Bottom Flange	6	6.872	0.018	0.123	0.002	--
High $Corner_{Stiff}$	12	1.321	1.302	1.719	2.238	0.000
Web_{Stiff}	12	6.762	1.313	8.878	11.656	0.046
Low $Corner_{Stiff}$	12	1.321	0.896	1.183	1.060	0.000
	Σ	48.568		44.329	53.580	1.942

Solved $\bar{y}$ =	ΣLy/ΣL =	0.913	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max($\bar{y}$, h - $\bar{y}$) =	0.913	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2\Sigma L]t$ =	0.539	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.591	$in.^3$
Mn =	**Se*Fy =**	**35.45**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)
Gage:	22	GA
Strength:	33	ksi
Thickness:	0.0295	in.
Total Height:	1.510	in.
Radius:	0.20275	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000477	$in.^3$

Comp Flange:	1.377	in
θ_{Stiff}:	30.625	deg
θ_{Stiff}:	0.535	rad
$Curve_{Stiff}$ I'_x:	0.000008	$in.^3$

Guess ȳ: 0.901 in.

Stress in Flange:	33.000	ksi
k:	4.000	
Fcr:	48.966	ksi
λ:	0.821	
ρ:	0.892	
Effective Width:	1.228	in.

Element	L (in.)	y from top (in.)
Lip	0.688	0.015
Corners	0.257	0.168
$Corners_{Stiff}$	0.108	0.208
Bottom Flange	1.228	0.015
Web	1.275	0.755
Web_{Stiff}	0.564	0.200
Top Flange	0.895	1.495

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	0	0.000	0.015	0.000	0.000	--
Bottom Corner	12	3.079	0.168	0.516	0.086	0.006
Web	12	15.296	0.755	11.544	8.713	1.884
Top Corner	12	3.079	1.342	4.131	5.544	0.006
Top Flange	12	10.745	1.495	16.061	24.007	--
Bottom Flange	6	7.366	0.015	0.109	0.002	--
High $Corner_{Stiff}$	12	1.300	1.302	1.693	2.203	0.000
Web_{Stiff}	12	6.762	1.310	8.857	11.600	0.046
Low $Corner_{Stiff}$	12	1.300	0.902	1.173	1.058	0.000
	Σ	48.926		44.083	53.213	1.942

Solved ȳ =	ΣLy/ΣL =	0.901	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.901	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2 \Sigma L]t$ =	0.455	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.505	$in.^3$
Mn =	**Se*Fy =**	**16.68**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)			
Gage:	22	GA			
Strength:	40	ksi			
Thickness:	0.0295	in.			
Total Height:	1.510	in.	Comp Flange:	1.377	in
Radius:	0.20275	in.			
θ:	72.5	deg	θ_{Stiff}:	30.625	deg
θ:	1.265	rad	θ_{Stiff}:	0.535	rad
Curve I'_x:	0.000477	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000008	$in.^3$

Guess ȳ: 0.909 in.

Stress in Flange:	40.000	ksi
k:	4.000	
Fcr:	48.966	ksi
λ:	0.904	
ρ:	0.837	
Effective Width:	1.152	in.

Element	L (in.)	y from top (in.)
Lip	0.688	0.015
Corners	0.257	0.168
$Corners_{Stiff}$	0.108	0.208
Bottom Flange	1.152	0.015
Web	1.275	0.755
Web_{Stiff}	0.564	0.200
Top Flange	0.895	1.495

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	0	0.000	0.015	0.000	0.000	--
Bottom Corner	12	3.079	0.168	0.516	0.086	0.006
Web	12	15.296	0.755	11.544	8.713	1.884
Top Corner	12	3.079	1.342	4.131	5.544	0.006
Top Flange	12	10.745	1.495	16.061	24.007	--
Bottom Flange	6	6.915	0.015	0.102	0.002	--
High $Corner_{Stiff}$	12	1.300	1.302	1.693	2.203	0.000
Web_{Stiff}	12	6.762	1.310	8.857	11.600	0.046
Low $Corner_{Stiff}$	12	1.300	0.902	1.173	1.058	0.000
	Σ	48.475		44.076	53.213	1.942

Solved ȳ =	ΣLy/ΣL =	0.909	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.909	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2\Sigma L]t$ =	0.445	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.489	$in.^3$
Mn =	**Se*Fy =**	**19.57**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)			
Gage:	22	GA			
Strength:	50	ksi			
Thickness:	0.0295	in.			
Total Height:	1.510	in.	Comp Flange:	1.377	in
Radius:	0.20275	in.			
θ:	72.5	deg	θ_{Stiff}:	30.625	deg
θ:	1.265	rad	θ_{Stiff}:	0.535	rad
Curve I'_x:	0.000477	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000008	$in.^3$

Guess $\bar{y}$: 0.919 in.

Stress in Flange:	50.000	ksi
k:	4.000	
Fcr:	48.966	ksi
λ:	1.011	
ρ:	0.774	
Effective Width:	1.066	in.

Element	L (in.)	y from top (in.)
Lip	0.688	0.015
Corners	0.257	0.168
$Corners_{Stiff}$	0.108	0.208
Bottom Flange	1.066	0.015
Web	1.275	0.755
Web_{Stiff}	0.564	0.200
Top Flange	0.895	1.495

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	ΣLy^2 ($in.^3$)	$\Sigma I'_x$ ($in.^3$)
Lip	0	0.000	0.015	0.000	0.000	--
Bottom Corner	12	3.079	0.168	0.516	0.086	0.006
Web	12	15.296	0.755	11.544	8.713	1.884
Top Corner	12	3.079	1.342	4.131	5.544	0.006
Top Flange	12	10.745	1.495	16.061	24.007	--
Bottom Flange	6	6.395	0.015	0.094	0.001	--
High $Corner_{Stiff}$	12	1.300	1.302	1.693	2.203	0.000
Web_{Stiff}	12	6.762	1.310	8.857	11.600	0.046
Low $Corner_{Stiff}$	12	1.300	0.902	1.173	1.058	0.000
	Σ	47.955		44.069	53.212	1.942

Solved $\bar{y}$ =	ΣLy/ΣL =	0.919	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max($\bar{y}$, h - $\bar{y}$) =	0.919	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2\Sigma L]t$ =	0.432	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.471	$in.^3$
Mn =	**Se*Fy =**	**23.53**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)			
Gage:	22	GA			
Strength:	60	ksi			
Thickness:	0.0295	in.			
Total Height:	1.510	in.	Comp Flange:	1.377	in
Radius:	0.20275	in.			
θ:	72.5	deg	θ_{Stiff}:	30.625	deg
θ:	1.265	rad	θ_{Stiff}:	0.535	rad
Curve I'_x:	0.000477	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000008	$in.^3$

Guess $\bar{y}$: 0.927 in.

Stress in Flange:	60.000	ksi
k:	4.000	
Fcr:	48.966	ksi
λ:	1.107	
ρ:	0.724	
Effective Width:	0.997	in.

Element	L (in.)	y from top (in.)
Lip	0.688	0.015
Corners	0.257	0.168
$Corners_{Stiff}$	0.108	0.208
Bottom Flange	0.997	0.015
Web	1.275	0.755
Web_{Stiff}	0.564	0.200
Top Flange	0.895	1.495

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	ΣLy^2 ($in.^3$)	$\Sigma I'_x$ ($in.^3$)
Lip	0	0.000	0.015	0.000	0.000	--
Bottom Corner	12	3.079	0.168	0.516	0.086	0.006
Web	12	15.296	0.755	11.544	8.713	1.884
Top Corner	12	3.079	1.342	4.131	5.544	0.006
Top Flange	12	10.745	1.495	16.061	24.007	--
Bottom Flange	6	5.979	0.015	0.088	0.001	--
High $Corner_{Stiff}$	12	1.300	1.302	1.693	2.203	0.000
Web_{Stiff}	12	6.762	1.310	8.857	11.600	0.046
Low $Corner_{Stiff}$	12	1.300	0.902	1.173	1.058	0.000
	Σ	47.540		44.063	53.212	1.942

Solved $\bar{y}$ =	ΣLy/ΣL =	0.927	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max($\bar{y}$, h - $\bar{y}$) =	0.927	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2 \Sigma L]t$ =	0.422	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.456	$in.^3$
Mn =	**Se*Fy =**	**27.34**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)			
Gage:	24	GA			
Strength:	33	ksi			
Thickness:	0.0238	in.			
Total Height:	1.504	in.	Comp Flange:	1.377	in
Radius:	0.1999	in.			
θ:	72.5	deg	θ_{Stiff}:	30.625	deg
θ:	1.265	rad	θ_{Stiff}:	0.535	rad
Curve I'_x:	0.000457	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000007	$in.^3$

Guess ȳ: 0.917 in.

Stress in Flange:	33.000	ksi
k:	4.000	
Fcr:	31.872	ksi
λ:	1.018	
ρ:	0.770	
Effective Width:	1.060	in.

Element	L (in.)	y from top (in.)
Lip	0.688	0.012
Corners	0.253	0.163
$Corners_{Stiff}$	0.107	0.202
Bottom Flange	1.060	0.012
Web	1.275	0.752
Web_{Stiff}	0.564	0.197
Top Flange	0.895	1.492

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	0	0.000	0.012	0.000	0.000	--
Bottom Corner	12	3.035	0.163	0.493	0.080	0.005
Web	12	15.296	0.752	11.501	8.647	1.884
Top Corner	12	3.035	1.341	4.071	5.460	0.005
Top Flange	12	10.745	1.492	16.030	23.915	--
Bottom Flange	6	6.363	0.012	0.076	0.001	--
High $Corner_{Stiff}$	12	1.282	1.301	1.669	2.171	0.000
Web_{Stiff}	12	6.762	1.307	8.837	11.549	0.046
Low $Corner_{Stiff}$	12	1.282	0.908	1.164	1.056	0.000
	Σ	47.800		43.841	52.881	1.941

Solved ȳ =	ΣLy/ΣL =	0.917	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.917	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2 \Sigma L]t$ =	0.348	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.379	$in.^3$
Mn =	**Se*Fy =**	**12.51**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)			
Gage:	24	GA			
Strength:	40	ksi			
Thickness:	0.0238	in.			
Total Height:	1.504	in.	Comp Flange:	1.377	in
Radius:	0.1999	in.			
θ:	72.5	deg	θ_{Stiff}:	30.625	deg
θ:	1.265	rad	θ_{Stiff}:	0.535	rad
Curve I'_x:	0.000457	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000007	$in.^3$

Guess ȳ: 0.926 in.

Stress in Flange:	40.000	ksi
k:	4.000	
Fcr:	31.872	ksi
λ:	1.120	
ρ:	0.717	
Effective Width:	0.988	in.

Element	L (in.)	y from top (in.)
Lip	0.688	0.012
Corners	0.253	0.163
$Corners_{Stiff}$	0.107	0.202
Bottom Flange	0.988	0.012
Web	1.275	0.752
Web_{Stiff}	0.564	0.197
Top Flange	0.895	1.492

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	0	0.000	0.012	0.000	0.000	--
Bottom Corner	12	3.035	0.163	0.493	0.080	0.005
Web	12	15.296	0.752	11.501	8.647	1.884
Top Corner	12	3.035	1.341	4.071	5.460	0.005
Top Flange	12	10.745	1.492	16.030	23.915	--
Bottom Flange	6	5.926	0.012	0.071	0.001	--
High $Corner_{Stiff}$	12	1.282	1.301	1.669	2.171	0.000
Web_{Stiff}	12	6.762	1.307	8.837	11.549	0.046
Low $Corner_{Stiff}$	12	1.282	0.908	1.164	1.056	0.000
	Σ	47.363		43.835	52.881	1.941

Solved ȳ =	ΣLy/ΣL =	0.926	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.926	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2 \Sigma L]t$ =	0.339	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.366	$in.^3$
Mn =	**Se*Fy =**	**14.66**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)			
Gage:	24	GA			
Strength:	50	ksi			
Thickness:	0.0238	in.			
Total Height:	1.504	in.	Comp Flange:	1.377	in
Radius:	0.1999	in.			
θ:	72.5	deg	θ_{Stiff}:	30.625	deg
θ:	1.265	rad	θ_{Stiff}:	0.535	rad
Curve I'_x:	0.000457	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000007	$in.^3$

Guess ȳ: 0.935 in.

Stress in Flange:	50.000	ksi
k:	4.000	
Fcr:	31.872	ksi
λ:	1.253	
ρ:	0.658	
Effective Width:	0.906	in.

Element	L (in.)	y from top (in.)
Lip	0.688	0.012
Corners	0.253	0.163
$Corners_{Stiff}$	0.107	0.202
Bottom Flange	0.906	0.012
Web	1.275	0.752
Web_{Stiff}	0.564	0.197
Top Flange	0.895	1.492

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	ΣLy^2 ($in.^3$)	$\Sigma I'_x$ ($in.^3$)
Lip	0	0.000	0.012	0.000	0.000	--
Bottom Corner	12	3.035	0.163	0.493	0.080	0.005
Web	12	15.296	0.752	11.501	8.647	1.884
Top Corner	12	3.035	1.341	4.071	5.460	0.005
Top Flange	12	10.745	1.492	16.030	23.915	--
Bottom Flange	6	5.437	0.012	0.065	0.001	--
High $Corner_{Stiff}$	12	1.282	1.301	1.669	2.171	0.000
Web_{Stiff}	12	6.762	1.307	8.837	11.549	0.046
Low $Corner_{Stiff}$	12	1.282	0.908	1.164	1.056	0.000
	Σ	46.874		43.830	52.881	1.941

Solved ȳ =	ΣLy/ΣL =	0.935	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.935	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2\Sigma L]t$ =	0.329	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.352	$in.^3$
Mn =	**Se*Fy =**	**17.61**	**k-in.**

EFFECTIVE WIDTH METHOD
NEGATIVE BENDING
CFS Floor & Roof Deck Sections

date: 1/12/2015
calc by: RKD
check by: TS

Deck:	1.5B	(stiffeners)
Gage:	24	GA
Strength:	60	ksi
Thickness:	0.0238	in.
Total Height:	1.504	in.
Radius:	0.1999	in.
θ:	72.5	deg
θ:	1.265	rad
Curve I'_x:	0.000457	$in.^3$

Comp Flange:	1.377	in
$θ_{Stiff}$:	30.625	deg
$θ_{Stiff}$:	0.535	rad
$Curve_{Stiff}$ I'_x:	0.000007	$in.^3$

Guess ȳ: 0.943 in.

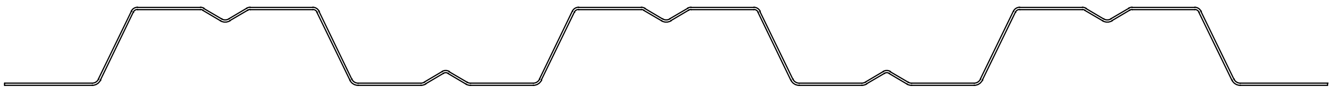
Stress in Flange:	60.000	ksi
k:	4.000	
Fcr:	31.872	ksi
λ:	1.372	
ρ:	0.612	
Effective Width:	0.843	in.

Element	L (in.)	y from top (in.)
Lip	0.688	0.012
Corners	0.253	0.163
$Corners_{Stiff}$	0.107	0.202
Bottom Flange	0.843	0.012
Web	1.275	0.752
Web_{Stiff}	0.564	0.197
Top Flange	0.895	1.492

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	0	0.000	0.012	0.000	0.000	--
Bottom Corner	12	3.035	0.163	0.493	0.080	0.005
Web	12	15.296	0.752	11.501	8.647	1.884
Top Corner	12	3.035	1.341	4.071	5.460	0.005
Top Flange	12	10.745	1.492	16.030	23.915	--
Bottom Flange	6	5.055	0.012	0.060	0.001	--
High $Corner_{Stiff}$	12	1.282	1.301	1.669	2.171	0.000
Web_{Stiff}	12	6.762	1.307	8.837	11.549	0.046
Low $Corner_{Stiff}$	12	1.282	0.908	1.164	1.056	0.000
	Σ	46.493		43.825	52.881	1.941

Solved ȳ =	ΣLy/ΣL =	0.943	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	0.943	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	0.322	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.341	$in.^3$
Mn =	**Se*Fy =**	**20.47**	**k-in.**

CHAPTER 8: ANALYSIS | 2 DECK (STIFFENED) | ± BENDING



8.0 Executive Summary

The Direct Strength Method predicted higher strengths for each of the 2C Deck sections undergoing positive and negative flexure. Similar to the stiffened 1.5B Deck, the 2C Deck is able to take advantage of the DSM with the compression flange stiffener breaking up and decreasing the flat width the compression element. The nominal moment capacity ratio (Mn_{DSM}/Mn_{EWM}) ranged between 1.108 and 1.295.

8.1 Mn_{DSM} / Mn_{EWM} vs. Thickness Plot

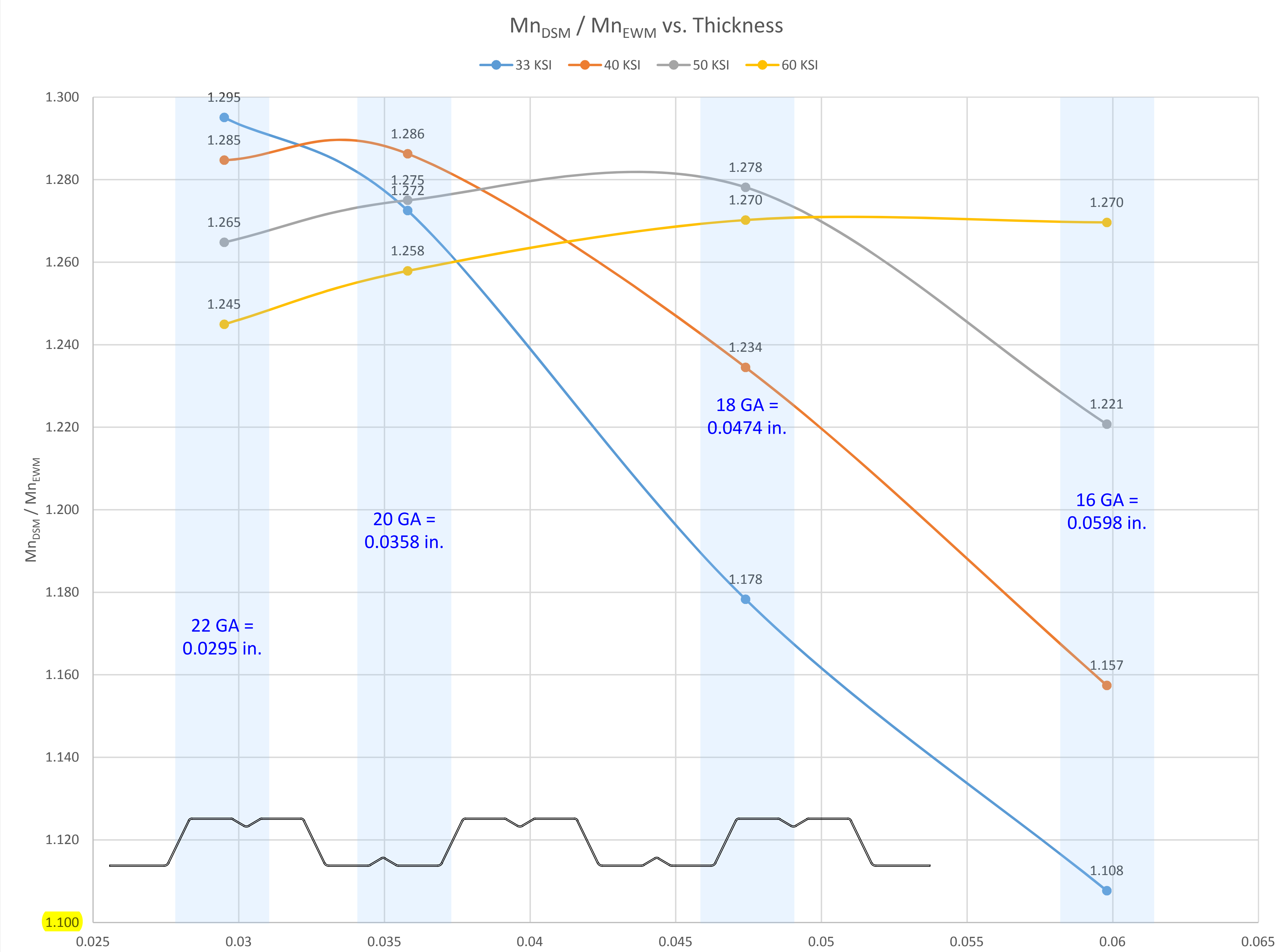


Figure 14 - 2C | MnDSM / MnEWM vs. Thickness | +/- Flexure

8.2 Analysis Results Summary

Table 8 - 2C | Analysis Results Summary | +/- Flexure

2 DECK - 33 KSI				
Gage	**22**	**20**	**18**	**16**
Thickness	0.0295	0.0358	0.0474	0.0598
Curve Radius	0.2028	0.2059	0.2117	0.2179
I_G (CUFSM)	1.030	1.249	1.654	2.087
y-bar (CUFSM)	1.04876	1.05239	1.05907	1.06621
Sxx	0.982	1.187	1.562	1.958
My	32.40	39.18	51.55	64.61
Mn_{DSM}	31.25	39.18	51.55	64.61
Mn_{EWM}	24.13	30.79	43.75	58.33
% ERROR	22.784%	21.414%	15.131%	9.720%

2 DECK - 33 KSI				
Thickness		Mn_{DSM}	Mn_{EWM}	Mn_{DSM} / Mn_{EWM}
22	0.0295	31.25	24.13	1.295
20	0.0358	39.18	30.79	1.272
18	0.0474	51.55	43.75	1.178
16	0.0598	64.61	58.33	1.108

2 DECK - 40 KSI				
Gage	**22**	**20**	**18**	**16**
Thickness	0.0295	0.0358	0.0474	0.0598
Curve Radius	0.2028	0.2059	0.2117	0.2179
I_G (CUFSM)	1.030	1.249	1.654	2.087
y-bar (CUFSM)	1.04876	1.05239	1.05907	1.06621
Sxx	0.982	1.187	1.562	1.958
My	39.27	47.49	62.49	78.31
Mn_{DSM}	35.78	45.74	62.49	78.31
Mn_{EWM}	27.85	35.56	50.62	67.66
% ERROR	22.163%	22.256%	18.995%	13.600%

2 DECK - 40 KSI				
Thickness		Mn_{DSM}	Mn_{EWM}	Mn_{DSM} / Mn_{EWM}
22	0.0295	35.78	27.85	1.285
20	0.0358	45.74	35.56	1.286
18	0.0474	62.49	50.62	1.234
16	0.0598	78.31	67.66	1.157

2 DECK - 50 KSI				
Gage	**22**	**20**	**18**	**16**
Thickness	0.0295	0.0358	0.0474	0.0598
Curve Radius	0.2028	0.2059	0.2117	0.2179
I_G (CUFSM)	1.030	1.249	1.654	2.087
y-bar (CUFSM)	1.04876	1.05239	1.05907	1.06621
Sxx	0.982	1.187	1.562	1.958
My	49.08	59.36	78.11	97.89
Mn_{DSM}	41.60	53.55	76.51	97.89
Mn_{EWM}	32.89	42	59.86	80.19
% ERROR	20.938%	21.569%	21.762%	18.082%

2 DECK - 50 KSI				
Thickness		Mn_{DSM}	Mn_{EWM}	Mn_{DSM} / Mn_{EWM}
22	0.0295	41.60	32.89	1.265
20	0.0358	53.55	42.00	1.275
18	0.0474	76.51	59.86	1.278
16	0.0598	97.89	80.19	1.221

2 DECK - 60 KSI				
Gage	**22**	**20**	**18**	**16**
Thickness	0.0295	0.0358	0.0474	0.0598
Curve Radius	0.2028	0.2059	0.2117	0.2179
I_G (CUFSM)	1.030	1.249	1.654	2.087
y-bar (CUFSM)	1.04876	1.05239	1.05907	1.06621
Sxx	0.982	1.187	1.562	1.958
My	58.90	71.23	93.73	117.47
Mn_{DSM}	46.91	60.53	87.15	116.82
Mn_{EWM}	37.68	48.12	68.61	92.01
% ERROR	19.676%	20.502%	21.274%	21.238%

2 DECK - 60 KSI				
Thickness		Mn_{DSM}	Mn_{EWM}	Mn_{DSM} / Mn_{EWM}
22	0.0295	46.91	37.68	1.245
20	0.0358	60.53	48.12	1.258
18	0.0474	87.15	68.61	1.270
16	0.0598	116.82	92.01	1.270

8.3 Direct Strength Method Calculations

Date:	11/26/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	2					
	Gage:	16 GA					
	Strength:	33 KSI					
	M_y =	64.61	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	5.86000	$M_{cr\ell}$ =	378.6146	kip-in	2	in
dist.	M_{crd}/M_y =	3.93000	M_{crd} =	253.9173	kip-in	12	in
global	M_{cre}/M_y =	12.06000	M_{cre} =	779.1966	kip-in	120	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1-\frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 64.61 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell}=M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell}=\left(1-0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.41 (local-global slenderness)

$M_{n\ell}$ = 64.61 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1-0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5}M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.50 (distortional slenderness)

M_{nd} = 64.61 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 64.61 kip-in (local-global controls)

Date:	11/26/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	2					
	Gage:	16 GA					
	Strength:	40 KSI					
	M_y =	78.31	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	4.84000	$M_{cr\ell}$ =	379.0204	kip-in	2	in
dist.	M_{crd}/M_y =	3.24000	M_{crd} =	253.7244	kip-in	12	in
global	M_{cre}/M_y =	9.95000	M_{cre} =	779.1845	kip-in	120	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 78.31 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.45 (local-global slenderness)

$M_{n\ell}$ = 78.31 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.56 (distortional slenderness)

M_{nd} = 78.31 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 78.31 kip-in (local-global controls)

Date: 11/26/14 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	2					
	Gage:	16	GA				
	Strength:	50	KSI				
	M_y =	97.89	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	3.87000	$M_{cr\ell}$ =	378.8343	kip-in	2	in
dist.	M_{crd}/M_y =	2.60000	M_{crd} =	254.514	kip-in	12	in
global	M_{cre}/M_y =	7.96000	M_{cre} =	779.2044	kip-in	120	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 97.89 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.51 (local-global slenderness)

$M_{n\ell}$ = 97.89 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.62 (distortional slenderness)

M_{nd} = 97.89 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 97.89 kip-in (local-global controls)

Date:	11/26/14		calc by:	RKD
			check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	2				
	Gage:	16 GA				
	Strength:	50 KSI				
	M_y =	117.47	kip-in		Length:	
local	$M_{cr\ell}/M_y$ =	3.23000	$M_{cr\ell}$ =	379.4281 kip-in	2	in
dist.	M_{crd}/M_y =	2.16000	M_{crd} =	253.7352 kip-in	12	in
global	M_{cre}/M_y =	6.63000	M_{cre} =	778.8261 kip-in	120	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 117.47 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (*Eq.* 1.2.2-7)

λ_ℓ = 0.56 (local-global slenderness)

$M_{n\ell}$ = 117.47 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (*Eq.* 1.2.2-10)

λ_d = 0.68 (distortional slenderness)

M_{nd} = 116.82 kip-in (distortional reduction)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 116.82 kip-in (distortional controls)

Date:	11/26/14		calc by:	RKD
			check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	2						
	Gage:	18	GA					
	Strength:	33	KSI					
	M_y =	51.55	kip-in				Length:	
local	$M_{cr\ell}/M_y$ =	3.80940		$M_{cr\ell}$ =	196.37457	kip-in	1	in
dist.	M_{crd}/M_y =	3.08000		M_{crd} =	158.774	kip-in	12	in
global	M_{cre}/M_y =	9.60000		M_{cre} =	494.88	kip-in	240	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 51.55 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.51 (local-global slenderness)

$M_{n\ell}$ = 51.55 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.57 (distortional slenderness)

M_{nd} = 51.55 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 51.55 kip-in (local-global controls)

Date:	11/26/14		calc by:	RKD
			check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	2				
	Gage:	18 GA				
	Strength:	40 KSI				
	M_y =	62.49	kip-in		Length:	
local	$M_{cr\ell}/M_y$ =	3.14270	$M_{cr\ell}$ =	196.38732 kip-in	1	in
dist.	M_{crd}/M_y =	2.54000	M_{crd} =	158.7246 kip-in	12	in
global	M_{cre}/M_y =	7.93000	M_{cre} =	495.5457 kip-in	240	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 62.49 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.56 (local-global slenderness)

$M_{n\ell}$ = 62.49 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.63 (distortional slenderness)

M_{nd} = 62.49 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 62.49 kip-in (local-global controls)

Date:	11/26/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	2				Length:	
	Gage:	18 GA					
	Strength:	50 KSI					
	$M_y =$	78.11	kip-in				
local	$M_{cr\ell}/M_y =$	2.51420	$M_{cr\ell} =$	196.38416	kip-in	1	in
dist.	$M_{crd}/M_y =$	2.04000	$M_{crd} =$	159.3444	kip-in	12	in
global	$M_{cre}/M_y =$	6.34000	$M_{cre} =$	495.2174	kip-in	240	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 78.11 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.63 (local-global slenderness)

$M_{n\ell}$ = 78.11 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.70 (distortional slenderness)

M_{nd} = 76.51 kip-in (distortional reduction)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 76.51 kip-in (distortional controls)

Date:	11/26/14		calc by:	RKD
			check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:							
	Deck:	2					
	Gage:	18 GA					
	Strength:	60 KSI					
	M_y =	93.73	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	2.09520	$M_{cr\ell}$ =	196.3831	kip-in	1	in
dist.	M_{crd}/M_y =	1.70000	M_{crd} =	159.341	kip-in	12	in
global	M_{cre}/M_y =	5.28000	M_{cre} =	494.8944	kip-in	240	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 93.73 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.69 (local-global slenderness)

$M_{n\ell}$ = 93.73 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.77 (distortional slenderness)

M_{nd} = 87.15 kip-in (distortional reduction)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 87.15 kip-in (distortional controls)

Date:	11/26/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:							
	Deck:	2					
	Gage:	20	GA				
	Strength:	33	KSI				
	M_y =	39.18	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	2.21990	$M_{cr\ell}$ =	86.975682	kip-in	1	in
dist.	M_{crd}/M_y =	2.33000	M_{crd} =	91.2894	kip-in	12	in
global	M_{cre}/M_y =	6.83000	M_{cre} =	267.5994	kip-in	240	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 39.18 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.67 (local-global slenderness)

$M_{n\ell}$ = 39.18 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.66 (distortional slenderness)

M_{nd} = 39.18 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 39.18 kip-in (local-global controls)

Date:	11/26/14		calc by:	RKD
			check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	2				
	Gage:	20 GA				
	Strength:	40 KSI				
	M_y =	47.49	kip-in		Length:	
local	$M_{cr\ell}/M_y$ =	1.83150	$M_{cr\ell}$ =	86.977935 kip-in	1	in
dist.	M_{crd}/M_y =	1.92000	M_{crd} =	91.1808 kip-in	12	in
global	M_{cre}/M_y =	5.64000	M_{cre} =	267.8436 kip-in	240	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 47.49 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.74 (local-global slenderness)

$M_{n\ell}$ = 47.49 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.72 (distortional slenderness)

M_{nd} = 45.74 kip-in (distortional reduction)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 45.74 kip-in (distortional controls)

Date:	11/26/14			calc by:	RKD
				check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	2					
	Gage:	20	GA				
	Strength:	50	KSI				
	M_y =	59.36	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	1.46520	$M_{cr\ell}$ =	86.974272	kip-in	1	in
dist.	M_{crd}/M_y =	1.54000	M_{crd} =	91.4144	kip-in	12	in
global	M_{cre}/M_y =	4.51000	M_{cre} =	267.7136	kip-in	240	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 59.36 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.83 (local-global slenderness)

$M_{n\ell}$ = 57.07 kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.81 (distortional slenderness)

M_{nd} = 53.55 kip-in (distortional reduction)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 53.55 kip-in (distortional controls)

Date:	11/26/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:							
	Deck:	2					
	Gage:	20	GA				
	Strength:	60	KSI				
	M_y =	71.23	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	1.22100	$M_{cr\ell}$ =	86.97183	kip-in	1	in
dist.	M_{crd}/M_y =	1.28000	M_{crd} =	91.1744	kip-in	12	in
global	M_{cre}/M_y =	3.76000	M_{cre} =	267.8248	kip-in	240	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **71.23** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.90 (local-global slenderness)

$M_{n\ell}$ = **64.62** kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.88 (distortional slenderness)

M_{nd} = **60.53** kip-in (distortional reduction)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 60.53 kip-in (distortional controls)

Date: 11/26/14 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:								
	Deck:	2						
	Gage:	22	GA					
	Strength:	33	KSI					
	M_y =	32.40	kip-in				Length:	
local	$M_{cr\ell}/M_y$ =	1.52840		$M_{cr\ell}$ =	49.52016	kip-in	1	in
dist.	M_{crd}/M_y =	1.93000		M_{crd} =	62.532	kip-in	12	in
global	M_{cre}/M_y =	5.57000		M_{cre} =	180.468	kip-in	240	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **32.40** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ $\qquad (Eq.\ 1.2.2\text{-}7)$

λ_ℓ = 0.81 (local-global slenderness)

$M_{n\ell}$ = **31.57** kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ $\qquad (Eq.\ 1.2.2\text{-}10)$

λ_d = 0.72 (distortional slenderness)

M_{nd} = **31.25** kip-in (distortional reduction)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 31.25 kip-in (distortional controls)

Date:	11/26/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	2				
	Gage:	22 GA				
	Strength:	40 KSI				
	M_y =	39.27	kip-in		Length:	
local	$M_{cr\ell}/M_y$ =	1.26090	$M_{cr\ell}$ =	49.515543 kip-in	1	in
dist.	M_{crd}/M_y =	1.59000	M_{crd} =	62.4393 kip-in	12	in
global	M_{cre}/M_y =	4.60000	M_{cre} =	180.642 kip-in	240	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq. 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq. 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq. 1.2.2\text{-}3)$$

M_{ne} = **39.27** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq. 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq. 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.89 (local-global slenderness)

$M_{n\ell}$ = **35.99** kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq. 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq. 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.79 (distortional slenderness)

M_{nd} = **35.78** kip-in (distortional reduction)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 35.78 kip-in (distortional controls)

Date:	11/26/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	2					
	Gage:	22	GA				
	Strength:	50	KSI				
	M_y =	49.08	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	1.00870	$M_{cr\ell}$ =	49.506996	kip-in	1	in
dist.	M_{crd}/M_y =	1.27000	M_{crd} =	62.3316	kip-in	12	in
global	M_{cre}/M_y =	3.68000	M_{cre} =	180.6144	kip-in	240	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 49.08 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (*Eq.* 1.2.2-7)

λ_ℓ = 1.00 (local-global slenderness)

$M_{n\ell}$ = 41.84 kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (*Eq.* 1.2.2-10)

λ_d = 0.89 (distortional slenderness)

M_{nd} = 41.60 kip-in (distortional reduction)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 41.60 kip-in (distortional controls)

Date:	11/26/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	2			Length:	
	Gage:	22 GA				
	Strength:	60 KSI				
	M_y =	58.90 kip-in				
local	$M_{cr\ell}/M_y$ =	0.84062	$M_{cr\ell}$ =	49.512518 kip-in	1	in
dist.	M_{crd}/M_y =	1.06000	M_{crd} =	62.434 kip-in	12	in
global	M_{cre}/M_y =	3.06000	M_{cre} =	180.234 kip-in	240	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1-\frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 58.90 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell}=M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell}=\left(1-0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 1.09 (local-global slenderness)

$M_{n\ell}$ = 47.26 kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1-0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5}M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.97 (distortional slenderness)

M_{nd} = 46.91 kip-in (distortional reduction)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 46.91 kip-in (distortional controls)

8.4 Effective Width Method Calculations

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	2		bo:	4.765	in.
Gage:	16	GA	h:	2.050	in.
Strength:	33	ksi	bp:	1.706	in.
Thickness:	0.0598	in.	Ag:	0.295	$in.^2$
Total Height:	2.120	in.	n:	1	
Radius:	0.2179	in.			
θ:	64.106	deg	θ_{Stiff}:	30.625	deg
θ:	1.119	rad	θ_{Stiff}:	0.535	rad
Curve I'_x:	0.000337	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000010	$in.^3$

Guess ȳ: 1.096 in.

Stress in Flange:	33.000	ksi	δ:	1.037	
k:	11.461		Isp:	0.004	$in.^4$
Fcr:	48.128	ksi	γ:	41.366	
λ:	0.828		β:	3.025	
ρ:	0.887		kd:	6.605	
Effective Width:	4.381	in.	kloc:	31.205	
			R:	1.735	

Element	L (in.)	y from top (in.)
Lip	2.382	2.090
Corners	0.244	0.205
$Corners_{Stiff}$	0.116	0.238
Bottom Flange	1.706	2.090
Web	2.050	1.060
Web_{Stiff}	0.564	0.215
Top Flange	4.381	0.083

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	ΣLy^2 ($in.^3$)	$\Sigma I'_x$ ($in.^3$)
Lip	2	4.764	2.090	9.956	20.808	--
Bottom Corner	6	1.463	1.915	2.801	5.363	0.002
Web	6	12.300	1.060	13.037	13.818	3.486
Top Corner	6	1.463	0.205	0.300	0.062	0.002
Top Flange	3	13.142	0.083	1.097	0.092	--
Bottom Flange	4	6.824	2.090	14.261	29.805	--
Low $Corner_{Stiff}$	4	0.466	1.882	0.877	1.650	0.000
Web_{Stiff}	4	2.254	1.905	4.294	8.179	0.015
High $Corner_{Stiff}$	4	0.466	1.452	0.677	0.983	0.000
	Σ	43.142		47.299	80.758	3.506

Solved ȳ =	ΣLy/ΣL =	1.096	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	1.096	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2 \Sigma L]t$ =	1.938	$in.^4$
S_e =	$I_x/\bar{y}$ =	1.768	$in.^3$
Mn =	**Se*Fy =**	**58.33**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	2		bo:	4.765	in.
Gage:	16	GA	h:	2.050	in.
Strength:	40	ksi	bp:	1.706	in.
Thickness:	0.0598	in.	Ag:	0.295	$in.^2$
Total Height:	2.120	in.	n:	1	
Radius:	0.2179	in.			
θ:	64.106	deg	θ_{Stiff}:	30.625	deg
θ:	1.119	rad	θ_{Stiff}:	0.535	rad
Curve I'_x:	0.000337	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000010	$in.^3$

Guess ȳ: 1.116 in.

Stress in Flange:	40.000	ksi	δ:	1.037	
k:	11.461		Isp:	0.004	$in.^4$
Fcr:	48.128	ksi	γ:	41.366	
λ:	0.912		β:	3.025	
ρ:	0.832		kd:	6.605	
Effective Width:	4.111	in.	kloc:	31.205	
			R:	1.735	

Element	L (in.)	y from top (in.)
Lip	2.382	2.090
Corners	0.244	0.205
$Corners_{Stiff}$	0.116	0.238
Bottom Flange	1.706	2.090
Web	2.050	1.060
Web_{Stiff}	0.564	0.215
Top Flange	4.111	0.083

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	ΣLy^2 ($in.^3$)	$\Sigma I'_x$ ($in.^3$)
Lip	2	4.764	2.090	9.956	20.808	--
Bottom Corner	6	1.463	1.915	2.801	5.363	0.002
Web	6	12.300	1.060	13.037	13.818	3.486
Top Corner	6	1.463	0.205	0.300	0.062	0.002
Top Flange	3	12.333	0.083	1.029	0.086	--
Bottom Flange	4	6.824	2.090	14.261	29.805	--
Low $Corner_{Stiff}$	4	0.466	1.882	0.877	1.650	0.000
Web_{Stiff}	4	2.254	1.905	4.294	8.179	0.015
High $Corner_{Stiff}$	4	0.466	1.452	0.677	0.983	0.000
	Σ	42.333		47.232	80.753	3.506

Solved ȳ =	ΣLy/ΣL =	1.116	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	1.116	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2 \Sigma L]t$ =	1.887	$in.^4$
S_e =	$I_x/\bar{y}$ =	1.692	$in.^3$
Mn =	**Se*Fy =**	**67.66**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	2		bo:	4.765	in.
Gage:	16	GA	h:	2.050	in.
Strength:	50	ksi	bp:	1.706	in.
Thickness:	0.0598	in.	Ag:	0.295	$in.^2$
Total Height:	2.120	in.	n:	1	
Radius:	0.2179	in.			
θ:	64.106	deg	$θ_{Stiff}$:	30.625	deg
θ:	1.119	rad	$θ_{Stiff}$:	0.535	rad
Curve I'_x:	0.000337	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000010	$in.^3$

Guess ȳ: 1.139 in.

Stress in Flange:	50.000	ksi	δ:	1.037	
k:	11.461		Isp:	0.004	$in.^4$
Fcr:	48.128	ksi	γ:	41.366	
λ:	1.019		β:	3.025	
ρ:	0.769		kd:	6.605	
Effective Width:	3.801	in.	kloc:	31.205	
			R:	1.735	

Element	L (in.)	y from top (in.)
Lip	2.382	2.090
Corners	0.244	0.205
$Corners_{Stiff}$	0.116	0.238
Bottom Flange	1.706	2.090
Web	2.050	1.060
Web_{Stiff}	0.564	0.215
Top Flange	3.801	0.083

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	4.764	2.090	9.956	20.808	--
Bottom Corner	6	1.463	1.915	2.801	5.363	0.002
Web	6	12.300	1.060	13.037	13.818	3.486
Top Corner	6	1.463	0.205	0.300	0.062	0.002
Top Flange	3	11.402	0.083	0.952	0.079	--
Bottom Flange	4	6.824	2.090	14.261	29.805	--
Low $Corner_{Stiff}$	4	0.466	1.882	0.877	1.650	0.000
Web_{Stiff}	4	2.254	1.905	4.294	8.179	0.015
High $Corner_{Stiff}$	4	0.466	1.452	0.677	0.983	0.000
	Σ	41.401		47.154	80.746	3.506

Solved ȳ =	ΣLy/ΣL =	1.139	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	1.139	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	1.827	$in.^4$
S_e =	$I_x/ȳ$ =	1.604	$in.^3$
Mn =	**Se*Fy =**	**80.19**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	2		bo:	4.765	in.
Gage:	16	GA	h:	2.050	in.
Strength:	60	ksi	bp:	1.706	in.
Thickness:	0.0598	in.	Ag:	0.295	$in.^2$
Total Height:	2.120	in.	n:	1	
Radius:	0.2179	in.			
θ:	64.106	deg	$θ_{Stiff}$:	30.625	deg
θ:	1.119	rad	$θ_{Stiff}$:	0.535	rad
Curve I'_x:	0.000337	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000010	$in.^3$

Guess $\bar{y}$: 1.158 in.

Stress in Flange:	60.000	ksi	δ:	1.037	
k:	11.461		Isp:	0.004	$in.^4$
Fcr:	48.128	ksi	γ:	41.366	
λ:	1.117		β:	3.025	
ρ:	0.719		kd:	6.605	
Effective Width:	3.553	in.	kloc:	31.205	
			R:	1.735	

Element	L (in.)	y from top (in.)
Lip	2.382	2.090
Corners	0.244	0.205
$Corners_{Stiff}$	0.116	0.238
Bottom Flange	1.706	2.090
Web	2.050	1.060
Web_{Stiff}	0.564	0.215
Top Flange	3.553	0.083

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	4.764	2.090	9.956	20.808	--
Bottom Corner	6	1.463	1.915	2.801	5.363	0.002
Web	6	12.300	1.060	13.037	13.818	3.486
Top Corner	6	1.463	0.205	0.300	0.062	0.002
Top Flange	3	10.658	0.083	0.889	0.074	--
Bottom Flange	4	6.824	2.090	14.261	29.805	--
Low $Corner_{Stiff}$	4	0.466	1.882	0.877	1.650	0.000
Web_{Stiff}	4	2.254	1.905	4.294	8.179	0.015
High $Corner_{Stiff}$	4	0.466	1.452	0.677	0.983	0.000
	Σ	40.657		47.092	80.741	3.506

Solved $\bar{y}$ =	$ΣLy/ΣL$ =	1.158	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max($\bar{y}$, h - $\bar{y}$) =	1.158	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	1.776	$in.^4$
S_e =	$I_x/\bar{y}$ =	1.533	$in.^3$
Mn =	**Se*Fy =**	**92.01**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	2		bo:	4.765	in.
Gage:	18	GA	h:	2.050	in.
Strength:	33	ksi	bp:	1.706	in.
Thickness:	0.0474	in.	Ag:	0.234	$in.^2$
Total Height:	2.107	in.	n:	1	
Radius:	0.2117	in.			
θ:	64.106	deg	$θ_{Stiff}$:	30.625	deg
θ:	1.119	rad	$θ_{Stiff}$:	0.535	rad
Curve I'_x:	0.000309	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000009	$in.^3$

Guess ȳ: 1.116 in.

Stress in Flange:	33.000	ksi	δ:	1.037	
k:	14.072		Isp:	0.003	$in.^4$
Fcr:	37.127	ksi	γ:	65.203	
λ:	0.943		β:	3.386	
ρ:	0.813		kd:	8.110	
Effective Width:	4.017	in.	kloc:	31.205	
			R:	1.735	

Element	L (in.)	y from top (in.)
Lip	2.382	2.084
Corners	0.237	0.194
$Corners_{Stiff}$	0.113	0.225
Bottom Flange	1.706	2.084
Web	2.050	1.054
Web_{Stiff}	0.564	0.209
Top Flange	4.017	0.077

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	4.764	2.084	9.927	20.684	--
Bottom Corner	6	1.421	1.913	2.719	5.204	0.002
Web	6	12.300	1.054	12.961	13.656	3.486
Top Corner	6	1.421	0.194	0.276	0.053	0.002
Top Flange	3	12.051	0.077	0.928	0.072	--
Bottom Flange	4	6.824	2.084	14.219	29.628	--
Low $Corner_{Stiff}$	4	0.453	1.882	0.852	1.603	0.000
Web_{Stiff}	4	2.254	1.899	4.280	8.126	0.015
High $Corner_{Stiff}$	4	0.453	1.465	0.663	0.971	0.000
	Σ	41.941		46.824	79.997	3.505

Solved ȳ =	ΣLy/ΣL =	1.116	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	1.116	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	1.480	$in.^4$
S_e =	$I_x/ȳ$ =	1.326	$in.^3$
Mn =	**Se*Fy =**	**43.75**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	2		bo:	4.765	in.
Gage:	18	GA	h:	2.050	in.
Strength:	40	ksi	bp:	1.706	in.
Thickness:	0.0474	in.	Ag:	0.234	$in.^2$
Total Height:	2.107	in.	n:	1	
Radius:	0.2117	in.			
θ:	64.106	deg	$θ_{Stiff}$:	30.625	deg
θ:	1.119	rad	$θ_{Stiff}$:	0.535	rad
Curve I'_x:	0.000309	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000009	$in.^3$

Guess ȳ: 1.137 in.

Stress in Flange:	40.000	ksi	δ:	1.037	
k:	14.072		Isp:	0.003	$in.^4$
Fcr:	37.127	ksi	γ:	65.203	
λ:	1.038		β:	3.386	
ρ:	0.759		kd:	8.110	
Effective Width:	3.751	in.	kloc:	31.205	
			R:	1.735	

Element	L (in.)	y from top (in.)
Lip	2.382	2.084
Corners	0.237	0.194
$Corners_{Stiff}$	0.113	0.225
Bottom Flange	1.706	2.084
Web	2.050	1.054
Web_{Stiff}	0.564	0.209
Top Flange	3.751	0.077

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	4.764	2.084	9.927	20.684	--
Bottom Corner	6	1.421	1.913	2.719	5.204	0.002
Web	6	12.300	1.054	12.961	13.656	3.486
Top Corner	6	1.421	0.194	0.276	0.053	0.002
Top Flange	3	11.252	0.077	0.867	0.067	--
Bottom Flange	4	6.824	2.084	14.219	29.628	--
Low $Corner_{Stiff}$	4	0.453	1.882	0.852	1.603	0.000
Web_{Stiff}	4	2.254	1.899	4.280	8.126	0.015
High $Corner_{Stiff}$	4	0.453	1.465	0.663	0.971	0.000
	Σ	41.141		46.763	79.993	3.505

Solved ȳ =	ΣLy/ΣL =	1.137	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	1.137	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	1.438	$in.^4$
S_e =	$I_x/ȳ$ =	1.265	$in.^3$
Mn =	**Se*Fy =**	**50.62**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	2		bo:	4.765	in.
Gage:	18	GA	h:	2.050	in.
Strength:	50	ksi	bp:	1.706	in.
Thickness:	0.0474	in.	Ag:	0.234	$in.^2$
Total Height:	2.107	in.	n:	1	
Radius:	0.2117	in.			
θ:	64.106	deg	$θ_{Stiff}$:	30.625	deg
θ:	1.119	rad	$θ_{Stiff}$:	0.535	rad
Curve I'_x:	0.000309	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000009	$in.^3$

Guess ȳ: 1.160 in.

Stress in Flange:	50.000	ksi	δ:	1.037	
k:	14.072		Isp:	0.003	$in.^4$
Fcr:	37.127	ksi	γ:	65.203	
λ:	1.160		β:	3.386	
ρ:	0.698		kd:	8.110	
Effective Width:	3.450	in.	kloc:	31.205	
			R:	1.735	

Element	L (in.)	y from top (in.)
Lip	2.382	2.084
Corners	0.237	0.194
$Corners_{Stiff}$	0.113	0.225
Bottom Flange	1.706	2.084
Web	2.050	1.054
Web_{Stiff}	0.564	0.209
Top Flange	3.450	0.077

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	4.764	2.084	9.927	20.684	--
Bottom Corner	6	1.421	1.913	2.719	5.204	0.002
Web	6	12.300	1.054	12.961	13.656	3.486
Top Corner	6	1.421	0.194	0.276	0.053	0.002
Top Flange	3	10.350	0.077	0.797	0.061	--
Bottom Flange	4	6.824	2.084	14.219	29.628	--
Low $Corner_{Stiff}$	4	0.453	1.882	0.852	1.603	0.000
Web_{Stiff}	4	2.254	1.899	4.280	8.126	0.015
High $Corner_{Stiff}$	4	0.453	1.465	0.663	0.971	0.000
	Σ	40.239		46.693	79.987	3.505

Solved ȳ =	ΣLy/ΣL =	1.160	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	1.160	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	1.389	$in.^4$
S_e =	$I_x/ȳ$ =	1.197	$in.^3$
Mn =	**Se*Fy =**	**59.86**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	2		bo:	4.765	in.
Gage:	18	GA	h:	2.050	in.
Strength:	60	ksi	bp:	1.706	in.
Thickness:	0.0474	in.	Ag:	0.234	$in.^2$
Total Height:	2.107	in.	n:	1	
Radius:	0.2117	in.			
θ:	64.106	deg	$θ_{Stiff}$:	30.625	deg
θ:	1.119	rad	$θ_{Stiff}$:	0.535	rad
Curve I'_x:	0.000309	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000009	$in.^3$

Guess ȳ: 1.180 in.

Stress in Flange:	60.000	ksi	δ:	1.037	
k:	14.072		Isp:	0.003	$in.^4$
Fcr:	37.127	ksi	γ:	65.203	
λ:	1.271		β:	3.386	
ρ:	0.650		kd:	8.110	
Effective Width:	3.213	in.	kloc:	31.205	
			R:	1.735	

Element	L (in.)	y from top (in.)
Lip	2.382	2.084
Corners	0.237	0.194
$Corners_{Stiff}$	0.113	0.225
Bottom Flange	1.706	2.084
Web	2.050	1.054
Web_{Stiff}	0.564	0.209
Top Flange	3.213	0.077

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	4.764	2.084	9.927	20.684	--
Bottom Corner	6	1.421	1.913	2.719	5.204	0.002
Web	6	12.300	1.054	12.961	13.656	3.486
Top Corner	6	1.421	0.194	0.276	0.053	0.002
Top Flange	3	9.640	0.077	0.743	0.057	--
Bottom Flange	4	6.824	2.084	14.219	29.628	--
Low $Corner_{Stiff}$	4	0.453	1.882	0.852	1.603	0.000
Web_{Stiff}	4	2.254	1.899	4.280	8.126	0.015
High $Corner_{Stiff}$	4	0.453	1.465	0.663	0.971	0.000
	Σ	39.530		46.638	79.983	3.505

Solved ȳ =	ΣLy/ΣL =	1.180	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	1.180	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	1.349	$in.^4$
S_e =	$I_x/ȳ$ =	1.144	$in.^3$
Mn =	**Se*Fy =**	**68.61**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	2		bo:	4.765	in.
Gage:	20	GA	h:	2.050	in.
Strength:	33	ksi	bp:	1.706	in.
Thickness:	0.0358	in.	Ag:	0.177	$in.^2$
Total Height:	2.096	in.	n:	1	
Radius:	0.2059	in.			
θ:	64.106	deg	θ_{Stiff}:	30.625	deg
θ:	1.119	rad	θ_{Stiff}:	0.535	rad
Curve I'_x:	0.000284	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000008	$in.^3$

Guess ȳ: 1.143 in.

Stress in Flange:	33.000	ksi	δ:	1.037	
k:	18.169		Isp:	0.002	$in.^4$
Fcr:	27.345	ksi	γ:	113.380	
λ:	1.099		β:	3.885	
ρ:	0.728		kd:	10.471	
Effective Width:	3.596	in.	kloc:	31.205	
			R:	1.735	

Element	L (in.)	y from top (in.)
Lip	2.382	2.078
Corners	0.230	0.183
$Corners_{Stiff}$	0.110	0.214
Bottom Flange	1.706	2.078
Web	2.050	1.048
Web_{Stiff}	0.564	0.203
Top Flange	3.596	0.071

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	ΣLy^2 ($in.^3$)	$\Sigma I'_x$ ($in.^3$)
Lip	2	4.764	2.078	9.899	20.569	--
Bottom Corner	6	1.382	1.912	2.643	5.055	0.002
Web	6	12.300	1.048	12.889	13.507	3.486
Top Corner	6	1.382	0.183	0.254	0.047	0.002
Top Flange	3	10.789	0.071	0.767	0.054	--
Bottom Flange	4	6.824	2.078	14.180	29.464	--
Low $Corner_{Stiff}$	4	0.440	1.882	0.828	1.559	0.000
Web_{Stiff}	4	2.254	1.893	4.267	8.076	0.015
High $Corner_{Stiff}$	4	0.440	1.476	0.650	0.959	0.000
	Σ	40.576		46.376	79.289	3.505

Solved ȳ =	ΣLy/ΣL =	1.143	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	1.143	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2 \Sigma L]t$ =	1.066	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.933	$in.^3$
Mn =	**Se*Fy =**	**30.79**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	2		bo:	4.765	in.
Gage:	20	GA	h:	2.050	in.
Strength:	40	ksi	bp:	1.706	in.
Thickness:	0.0358	in.	Ag:	0.177	$in.^2$
Total Height:	2.096	in.	n:	1	
Radius:	0.2059	in.			
θ:	64.106	deg	$θ_{Stiff}$:	30.625	deg
θ:	1.119	rad	$θ_{Stiff}$:	0.535	rad
Curve I'_x:	0.000284	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000008	$in.^3$

Guess ȳ: 1.164 in.

Stress in Flange:	40.000	ksi	δ:	1.037	
k:	18.169		Isp:	0.002	$in.^4$
Fcr:	27.345	ksi	γ:	113.380	
λ:	1.209		β:	3.885	
ρ:	0.676		kd:	10.471	
Effective Width:	3.342	in.	kloc:	31.205	
			R:	1.735	

Element	L (in.)	y from top (in.)
Lip	2.382	2.078
Corners	0.230	0.183
$Corners_{Stiff}$	0.110	0.214
Bottom Flange	1.706	2.078
Web	2.050	1.048
Web_{Stiff}	0.564	0.203
Top Flange	3.342	0.071

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	4.764	2.078	9.899	20.569	--
Bottom Corner	6	1.382	1.912	2.643	5.055	0.002
Web	6	12.300	1.048	12.889	13.507	3.486
Top Corner	6	1.382	0.183	0.254	0.047	0.002
Top Flange	3	10.025	0.071	0.712	0.051	--
Bottom Flange	4	6.824	2.078	14.180	29.464	--
Low $Corner_{Stiff}$	4	0.440	1.882	0.828	1.559	0.000
Web_{Stiff}	4	2.254	1.893	4.267	8.076	0.015
High $Corner_{Stiff}$	4	0.440	1.476	0.650	0.959	0.000
	Σ	39.811		46.322	79.286	3.505

Solved ȳ =	ΣLy/ΣL =	1.164	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	1.164	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	1.034	$in.^4$
S_e =	$I_x/ȳ$ =	0.889	$in.^3$
Mn =	**Se*Fy =**	**35.56**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	2		bo:	4.765	in.
Gage:	20	GA	h:	2.050	in.
Strength:	50	ksi	bp:	1.706	in.
Thickness:	0.0358	in.	Ag:	0.177	$in.^2$
Total Height:	2.096	in.	n:	1	
Radius:	0.2059	in.			
θ:	64.106	deg	$θ_{Stiff}$:	30.625	deg
θ:	1.119	rad	$θ_{Stiff}$:	0.535	rad
Curve I'_x:	0.000284	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000008	$in.^3$

Guess ȳ: 1.187 in.

Stress in Flange:	50.000	ksi	δ:	1.037	
k:	18.169		Isp:	0.002	$in.^4$
Fcr:	27.345	ksi	γ:	113.380	
λ:	1.352		β:	3.885	
ρ:	0.619		kd:	10.471	
Effective Width:	3.059	in.	kloc:	31.205	
			R:	1.735	

Element	L (in.)	y from top (in.)
Lip	2.382	2.078
Corners	0.230	0.183
$Corners_{Stiff}$	0.110	0.214
Bottom Flange	1.706	2.078
Web	2.050	1.048
Web_{Stiff}	0.564	0.203
Top Flange	3.059	0.071

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	4.764	2.078	9.899	20.569	--
Bottom Corner	6	1.382	1.912	2.643	5.055	0.002
Web	6	12.300	1.048	12.889	13.507	3.486
Top Corner	6	1.382	0.183	0.254	0.047	0.002
Top Flange	3	9.177	0.071	0.652	0.046	--
Bottom Flange	4	6.824	2.078	14.180	29.464	--
Low $Corner_{Stiff}$	4	0.440	1.882	0.828	1.559	0.000
Web_{Stiff}	4	2.254	1.893	4.267	8.076	0.015
High $Corner_{Stiff}$	4	0.440	1.476	0.650	0.959	0.000
	Σ	38.964		46.261	79.281	3.505

Solved ȳ =	ΣLy/ΣL =	1.187	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	1.187	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	0.997	$in.^4$
S_e =	$I_x/ȳ$ =	0.840	$in.^3$
Mn =	**Se*Fy =**	**42.00**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	2		bo:	4.765	in.
Gage:	20	GA	h:	2.050	in.
Strength:	60	ksi	bp:	1.706	in.
Thickness:	0.0358	in.	Ag:	0.177	$in.^2$
Total Height:	2.096	in.	n:	1	
Radius:	0.2059	in.			
θ:	64.106	deg	$θ_{Stiff}$:	30.625	deg
θ:	1.119	rad	$θ_{Stiff}$:	0.535	rad
Curve I'_x:	0.000284	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000008	$in.^3$

Guess ȳ: 1.206 in.

Stress in Flange:	60.000	ksi	δ:	1.037	
k:	18.169		Isp:	0.002	$in.^4$
Fcr:	27.345	ksi	γ:	113.380	
λ:	1.481		β:	3.885	
ρ:	0.575		kd:	10.471	
Effective Width:	2.840	in.	kloc:	31.205	
			R:	1.735	

Element	L (in.)	y from top (in.)
Lip	2.382	2.078
Corners	0.230	0.183
$Corners_{Stiff}$	0.110	0.214
Bottom Flange	1.706	2.078
Web	2.050	1.048
Web_{Stiff}	0.564	0.203
Top Flange	2.840	0.071

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	4.764	2.078	9.899	20.569	--
Bottom Corner	6	1.382	1.912	2.643	5.055	0.002
Web	6	12.300	1.048	12.889	13.507	3.486
Top Corner	6	1.382	0.183	0.254	0.047	0.002
Top Flange	3	8.519	0.071	0.605	0.043	--
Bottom Flange	4	6.824	2.078	14.180	29.464	--
Low $Corner_{Stiff}$	4	0.440	1.882	0.828	1.559	0.000
Web_{Stiff}	4	2.254	1.893	4.267	8.076	0.015
High $Corner_{Stiff}$	4	0.440	1.476	0.650	0.959	0.000
	Σ	38.306		46.215	79.278	3.505

Solved ȳ =	ΣLy/ΣL =	1.206	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	1.206	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	0.968	$in.^4$
S_e =	$I_x/ȳ$ =	0.802	$in.^3$
Mn =	**Se*Fy =**	**48.12**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	2		bo:	4.765	in.
Gage:	22	GA	h:	2.050	in.
Strength:	33	ksi	bp:	1.706	in.
Thickness:	0.0295	in.	Ag:	0.146	$in.^2$
Total Height:	2.090	in.	n:	1	
Radius:	0.20275	in.			
θ:	64.106	deg	$θ_{Stiff}$:	30.625	deg
θ:	1.119	rad	$θ_{Stiff}$:	0.535	rad
Curve I'_x:	0.000271	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000008	$in.^3$

Guess ȳ: 1.162 in.

Stress in Flange:	33.000	ksi	δ:	1.037	
k:	21.736		Isp:	0.002	$in.^4$
Fcr:	22.212	ksi	γ:	166.038	
λ:	1.219		β:	4.272	
ρ:	0.672		kd:	12.527	
Effective Width:	3.321	in.	kloc:	31.205	
			R:	1.735	

Element	L (in.)	y from top (in.)
Lip	2.382	2.075
Corners	0.227	0.178
$Corners_{Stiff}$	0.108	0.208
Bottom Flange	1.706	2.075
Web	2.050	1.045
Web_{Stiff}	0.564	0.200
Top Flange	3.321	0.068

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	4.764	2.075	9.884	20.507	--
Bottom Corner	6	1.361	1.912	2.602	4.974	0.002
Web	6	12.300	1.045	12.850	13.425	3.486
Top Corner	6	1.361	0.178	0.242	0.043	0.002
Top Flange	3	9.964	0.068	0.675	0.046	--
Bottom Flange	4	6.824	2.075	14.158	29.375	--
Low $Corner_{Stiff}$	4	0.433	1.882	0.816	1.535	0.000
Web_{Stiff}	4	2.254	1.890	4.259	8.049	0.015
High $Corner_{Stiff}$	4	0.433	1.482	0.642	0.952	0.000
	Σ	39.695		46.130	78.906	3.505

Solved ȳ =	ΣLy/ΣL =	1.162	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	1.162	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	0.850	$in.^4$
S_e =	$I_x/ȳ$ =	0.731	$in.^3$
Mn =	**Se*Fy =**	**24.13**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	2		bo:	4.765	in.
Gage:	22	GA	h:	2.050	in.
Strength:	40	ksi	bp:	1.706	in.
Thickness:	0.0295	in.	Ag:	0.146	$in.^2$
Total Height:	2.090	in.	n:	1	
Radius:	0.20275	in.			
θ:	64.106	deg	θ_{Stiff}:	30.625	deg
θ:	1.119	rad	θ_{Stiff}:	0.535	rad
Curve I'_x:	0.000271	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000008	$in.^3$

Guess ȳ: 1.183 in.

Stress in Flange:	40.000	ksi	δ:	1.037	
k:	21.736		Isp:	0.002	$in.^4$
Fcr:	22.212	ksi	γ:	166.038	
λ:	1.342		β:	4.272	
ρ:	0.623		kd:	12.527	
Effective Width:	3.078	in.	kloc:	31.205	
			R:	1.735	

Element	L (in.)	y from top (in.)
Lip	2.382	2.075
Corners	0.227	0.178
$Corners_{Stiff}$	0.108	0.208
Bottom Flange	1.706	2.075
Web	2.050	1.045
Web_{Stiff}	0.564	0.200
Top Flange	3.078	0.068

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	ΣLy^2 ($in.^3$)	$\Sigma I'_x$ ($in.^3$)
Lip	2	4.764	2.075	9.884	20.507	--
Bottom Corner	6	1.361	1.912	2.602	4.974	0.002
Web	6	12.300	1.045	12.850	13.425	3.486
Top Corner	6	1.361	0.178	0.242	0.043	0.002
Top Flange	3	9.233	0.068	0.626	0.042	--
Bottom Flange	4	6.824	2.075	14.158	29.375	--
Low $Corner_{Stiff}$	4	0.433	1.882	0.816	1.535	0.000
Web_{Stiff}	4	2.254	1.890	4.259	8.049	0.015
High $Corner_{Stiff}$	4	0.433	1.482	0.642	0.952	0.000
	Σ	38.964		46.080	78.903	3.505

Solved ȳ =	ΣLy/ΣL =	1.183	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	1.183	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2 \Sigma L]t$ =	0.823	$in.^4$
S_e =	$I_x/\bar{y}$ =	0.696	$in.^3$
Mn =	**Se*Fy =**	**27.85**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	2		bo:	4.765	in.
Gage:	22	GA	h:	2.050	in.
Strength:	50	ksi	bp:	1.706	in.
Thickness:	0.0295	in.	Ag:	0.146	$in.^2$
Total Height:	2.090	in.	n:	1	
Radius:	0.20275	in.			
θ:	64.106	deg	$θ_{Stiff}$:	30.625	deg
θ:	1.119	rad	$θ_{Stiff}$:	0.535	rad
Curve I'_x:	0.000271	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000008	$in.^3$

Guess ȳ: 1.206 in.

Stress in Flange:	50.000	ksi	δ:	1.037	
k:	21.736		Isp:	0.002	$in.^4$
Fcr:	22.212	ksi	γ:	166.038	
λ:	1.500		β:	4.272	
ρ:	0.569		kd:	12.527	
Effective Width:	2.810	in.	kloc:	31.205	
			R:	1.735	

Element	L (in.)	y from top (in.)
Lip	2.382	2.075
Corners	0.227	0.178
$Corners_{Stiff}$	0.108	0.208
Bottom Flange	1.706	2.075
Web	2.050	1.045
Web_{Stiff}	0.564	0.200
Top Flange	2.810	0.068

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	4.764	2.075	9.884	20.507	--
Bottom Corner	6	1.361	1.912	2.602	4.974	0.002
Web	6	12.300	1.045	12.850	13.425	3.486
Top Corner	6	1.361	0.178	0.242	0.043	0.002
Top Flange	3	8.429	0.068	0.571	0.039	--
Bottom Flange	4	6.824	2.075	14.158	29.375	--
$Low Corner_{Stiff}$	4	0.433	1.882	0.816	1.535	0.000
Web_{Stiff}	4	2.254	1.890	4.259	8.049	0.015
$High Corner_{Stiff}$	4	0.433	1.482	0.642	0.952	0.000
	Σ	38.161		46.026	78.899	3.505

Solved ȳ =	ΣLy/ΣL =	1.206	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	1.206	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	0.793	$in.^4$
S_e =	I_x/ȳ =	0.658	$in.^3$
Mn =	**Se*Fy =**	**32.89**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	2		bo:	4.765	in.
Gage:	22	GA	h:	2.050	in.
Strength:	60	ksi	bp:	1.706	in.
Thickness:	0.0295	in.	Ag:	0.146	$in.^2$
Total Height:	2.090	in.	n:	1	
Radius:	0.20275	in.			
θ:	64.106	deg	$θ_{Stiff}$:	30.625	deg
θ:	1.119	rad	$θ_{Stiff}$:	0.535	rad
Curve I'_x:	0.000271	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000008	$in.^3$

Guess ȳ: 1.225 in.

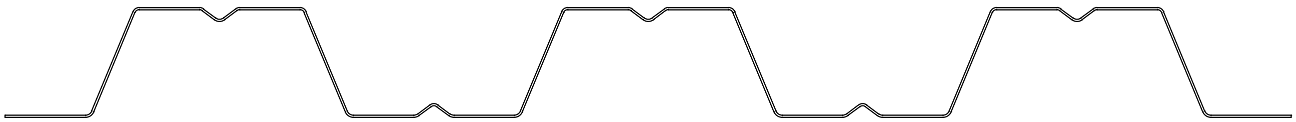
Stress in Flange:	60.000	ksi	δ:	1.037	
k:	21.736		Isp:	0.002	$in.^4$
Fcr:	22.212	ksi	γ:	166.038	
λ:	1.644		β:	4.272	
ρ:	0.527		kd:	12.527	
Effective Width:	2.603	in.	kloc:	31.205	
			R:	1.735	

Element	L (in.)	y from top (in.)
Lip	2.382	2.075
Corners	0.227	0.178
$Corners_{Stiff}$	0.108	0.208
Bottom Flange	1.706	2.075
Web	2.050	1.045
Web_{Stiff}	0.564	0.200
Top Flange	2.603	0.068

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	4.764	2.075	9.884	20.507	--
Bottom Corner	6	1.361	1.912	2.602	4.974	0.002
Web	6	12.300	1.045	12.850	13.425	3.486
Top Corner	6	1.361	0.178	0.242	0.043	0.002
Top Flange	3	7.810	0.068	0.529	0.036	--
Bottom Flange	4	6.824	2.075	14.158	29.375	--
Low $Corner_{Stiff}$	4	0.433	1.882	0.816	1.535	0.000
Web_{Stiff}	4	2.254	1.890	4.259	8.049	0.015
High $Corner_{Stiff}$	4	0.433	1.482	0.642	0.952	0.000
	Σ	37.541		45.984	78.896	3.505

Solved ȳ =	ΣLy/ΣL =	1.225	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	1.225	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	0.769	$in.^4$
S_e =	$I_x/ȳ$ =	0.628	$in.^3$
Mn =	**Se*Fy =**	**37.68**	**k-in.**

CHAPTER 9: ANALYSIS | 3 DECK (STIFFENED) | ± BENDING



9.0 Executive Summary

The Direct Strength Method predicted higher strengths for each of the 3C Deck sections undergoing positive and negative flexure. Similar to the stiffened 1.5B Deck, the 3C Deck is able to take advantage of the DSM with the compression flange stiffener breaking up and decreasing the flat width the compression element. The nominal moment capacity ratio (Mn_{DSM}/Mn_{EWM}) ranged between 1.056 and 1.192.

9.1 Mn_{DSM} / Mn_{EWM} vs. Thickness Plot

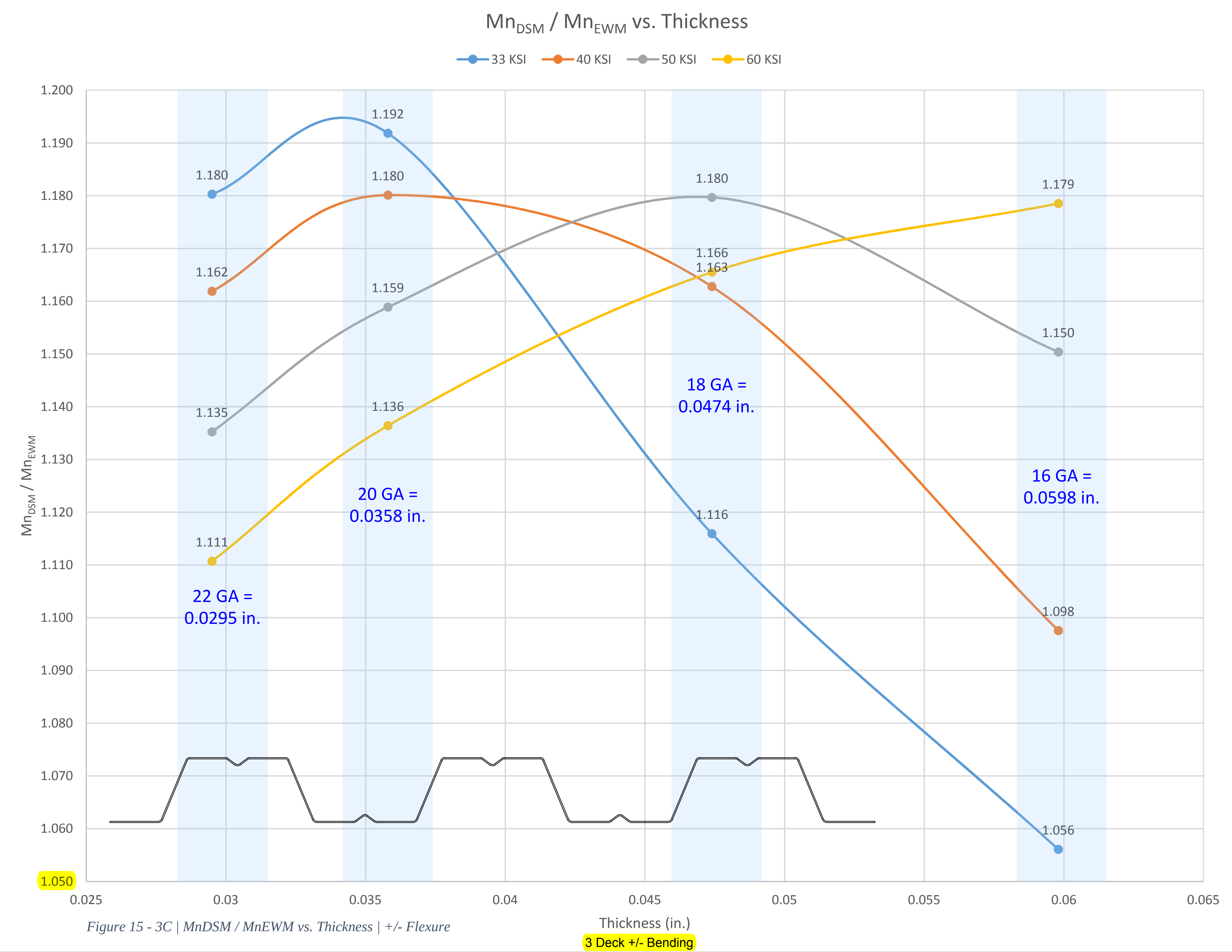


Figure 15 - 3C | MnDSM / MnEWM vs. Thickness | +/- Flexure

9.2 Analysis Results Summary

Table 9 - 3C | Analysis Results Summary | +/- Flexure

3 DECK - 33 KSI				
Gage	**22**	**20**	**18**	**16**
Thickness	0.0295	0.0358	0.0474	0.0598
Curve Radius	0.2028	0.2059	0.2117	0.2179
I_G (CUFSM)	2.276	2.762	3.657	4.613
y-bar (CUFSM)	1.51607	1.51986	1.52684	1.53430
Sxx	1.501	1.817	2.395	3.007
My	49.53	59.96	79.03	99.23
Mn_{DSM}	46.61	59.77	79.03	99.23
Mn_{EWM}	39.49	50.15	70.82	93.96
% ERROR	15.276%	16.095%	10.388%	5.311%

3 DECK - 33 KSI				
Thickness		Mn_{DSM}	Mn_{EWM}	Mn_{DSM} / Mn_{EWM}
22	0.0295	46.61	39.49	1.180
20	0.0358	59.77	50.15	1.192
18	0.0474	79.03	70.82	1.116
16	0.0598	99.23	93.96	1.056

3 DECK - 40 KSI				
Gage	**22**	**20**	**18**	**16**
Thickness	0.0295	0.0358	0.0474	0.0598
Curve Radius	0.2028	0.2059	0.2117	0.2179
I_G (CUFSM)	2.276	2.762	3.657	4.613
y-bar (CUFSM)	1.51607	1.51986	1.52684	1.53430
Sxx	1.501	1.817	2.395	3.007
My	60.04	72.68	95.80	120.27
Mn_{DSM}	53.26	68.73	95.80	120.27
Mn_{EWM}	45.84	58.24	82.39	109.58
% ERROR	13.932%	15.263%	13.998%	8.888%

3 DECK - 40 KSI				
Thickness		Mn_{DSM}	Mn_{EWM}	Mn_{DSM} / Mn_{EWM}
22	0.0295	53.26	45.84	1.162
20	0.0358	68.73	58.24	1.180
18	0.0474	95.80	82.39	1.163
16	0.0598	120.27	109.58	1.098

3 DECK - 50 KSI				
Gage	**22**	**20**	**18**	**16**
Thickness	0.0295	0.0358	0.0474	0.0598
Curve Radius	0.2028	0.2059	0.2117	0.2179
I_G (CUFSM)	2.276	2.762	3.657	4.613
y-bar (CUFSM)	1.51607	1.51986	1.52684	1.53430
Sxx	1.501	1.817	2.395	3.007
My	75.05	90.85	119.75	150.34
Mn_{DSM}	61.86	80.24	115.68	150.34
Mn_{EWM}	54.49	69.24	98.06	130.69
% ERROR	11.914%	13.709%	15.232%	13.070%

3 DECK - 50 KSI				
Thickness		Mn_{DSM}	Mn_{EWM}	Mn_{DSM} / Mn_{EWM}
22	0.0295	61.86	54.49	1.135
20	0.0358	80.24	69.24	1.159
18	0.0474	115.68	98.06	1.180
16	0.0598	150.34	130.69	1.150

3 DECK - 60 KSI				
Gage	**22**	**20**	**18**	**16**
Thickness	0.0295	0.0358	0.0474	0.0598
Curve Radius	0.2028	0.2059	0.2117	0.2179
I_G (CUFSM)	2.276	2.762	3.657	4.613
y-bar (CUFSM)	1.51607	1.51986	1.52684	1.53430
Sxx	1.501	1.817	2.395	3.007
My	90.06	109.02	143.69	180.41
Mn_{DSM}	69.74	90.64	131.68	177.64
Mn_{EWM}	62.79	79.76	112.98	150.73
% ERROR	9.966%	12.004%	14.201%	15.149%

3 DECK - 60 KSI				
Thickness		Mn_{DSM}	Mn_{EWM}	Mn_{DSM} / Mn_{EWM}
22	0.0295	69.74	62.79	1.111
20	0.0358	90.64	79.76	1.136
18	0.0474	131.68	112.98	1.166
16	0.0598	177.64	150.73	1.179

9.3 Direct Strength Method Calculations

Date: 11/26/14 calc by: RKD
check by: TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	3					
	Gage:	16	GA				
	Strength:	33	KSI				
	M_y =	99.23	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	5.58000	$M_{cr\ell}$ =	553.7034	kip-in	2	in
dist.	M_{crd}/M_y =	3.79000	M_{crd} =	376.0817	kip-in	10	in
global	M_{cre}/M_y =	4.00000	M_{cre} =	396.92	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 99.23 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.42 (local-global slenderness)

$M_{n\ell}$ = 99.23 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.51 (distortional slenderness)

M_{nd} = 99.23 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 99.23 kip-in (local-global controls)

Date:	11/26/14		calc by:	RKD
			check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	3					
	Gage:	16 GA					
	Strength:	40 KSI					
	M_y =	120.27	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	4.61000	$M_{cr\ell}$ =	554.4447	kip-in	2	in
dist.	M_{crd}/M_y =	3.12000	M_{crd} =	375.2424	kip-in	10	in
global	M_{cre}/M_y =	4.00000	M_{cre} =	481.08	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 120.27 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.47 (local-global slenderness)

$M_{n\ell}$ = 120.27 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.57 (distortional slenderness)

M_{nd} = 120.27 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 120.27 kip-in (local-global controls)

Date:	11/26/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	3					
	Gage:	16	GA				
	Strength:	50	KSI				
	M_y =	150.34	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	3.68000	$M_{cr\ell}$ =	553.2512	kip-in	2	in
dist.	M_{crd}/M_y =	2.50000	M_{crd} =	375.85	kip-in	10	in
global	M_{cre}/M_y =	4.00000	M_{cre} =	601.36	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1-\frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 150.34 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell}=M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell}=\left(1-0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ $\qquad (Eq.\ 1.2.2\text{-}7)$

λ_ℓ = 0.52 (local-global slenderness)

$M_{n\ell}$ = 150.34 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1-0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5}M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ $\qquad (Eq.\ 1.2.2\text{-}10)$

λ_d = 0.63 (distortional slenderness)

M_{nd} = 150.34 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 150.34 kip-in (local-global controls)

Date:	11/26/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	3					
	Gage:	16 GA					
	Strength:	60 KSI					
	M_y =	180.41	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	3.07000	$M_{cr\ell}$ =	553.8587	kip-in	2	in
dist.	M_{crd}/M_y =	2.08000	M_{crd} =	375.2528	kip-in	10	in
global	M_{cre}/M_y =	4.00000	M_{cre} =	721.64	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 180.41 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.57 (local-global slenderness)

$M_{n\ell}$ = 180.41 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.69 (distortional slenderness)

M_{nd} = 177.64 kip-in (distortional reduction)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 177.64 kip-in (distortional controls)

Date:	11/26/14			calc by:	RKD
				check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:						
	Deck:	3				
	Gage:	18 GA				
	Strength:	33 KSI				
	M_y =	79.03	kip-in		Length:	
local	$M_{cr\ell}/M_y$ =	3.62000	$M_{cr\ell}$ =	286.0886 kip-in	2	in
dist.	M_{crd}/M_y =	2.95000	M_{crd} =	233.1385 kip-in	10	in
global	M_{cre}/M_y =	4.00000	M_{cre} =	316.12 kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 79.03 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ $\qquad (Eq.\ 1.2.2\text{-}7)$

λ_ℓ = 0.53 (local-global slenderness)

$M_{n\ell}$ = 79.03 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ $\qquad (Eq.\ 1.2.2\text{-}10)$

λ_d = 0.58 (distortional slenderness)

M_{nd} = 79.03 kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 79.03 kip-in (local-global controls)

Date:	11/26/14		calc by:	RKD
			check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	3					
	Gage:	18	GA				
	Strength:	40	KSI				
	M_y =	95.80	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	2.99000	$M_{cr\ell}$ =	286.442	kip-in	2	in
dist.	M_{crd}/M_y =	2.43000	M_{crd} =	232.794	kip-in	10	in
global	M_{cre}/M_y =	4.00000	M_{cre} =	383.2	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **95.80** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.58 (local-global slenderness)

$M_{n\ell}$ = **95.80** kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.64 (distortional slenderness)

M_{nd} = **95.80** kip-in (fully effective section for distortional buckling)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 95.80 kip-in (local-global controls)

Date:	11/26/14			calc by:	RKD
				check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	3					
	Gage:	18 GA					
	Strength:	50 KSI					
	$M_y =$	119.75	kip-in			Length:	
local	$M_{cr\ell}/M_y =$	2.39000	$M_{cr\ell} =$	286.2025	kip-in	2	in
dist.	$M_{crd}/M_y =$	1.94000	$M_{crd} =$	232.315	kip-in	10	in
global	$M_{cre}/M_y =$	4.00000	$M_{cre} =$	479	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 119.75 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.65 (local-global slenderness)

$M_{n\ell}$ = 119.75 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.72 (distortional slenderness)

M_{nd} = 115.68 kip-in (distortional reduction)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 115.68 kip-in (distortional controls)

Date:	11/26/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	3						
	Gage:	18	GA					
	Strength:	60	KSI					
	M_y =	143.69	kip-in				Length:	
local	$M_{cr\ell}/M_y$ =	1.99000		$M_{cr\ell}$ =	285.9431	kip-in	2	in
dist.	M_{crd}/M_y =	1.62000		M_{crd} =	232.7778	kip-in	10	in
global	M_{cre}/M_y =	4.00000		M_{cre} =	574.76	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1-\frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **143.69** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1-0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.71 (local-global slenderness)

$M_{n\ell}$ = **143.69** kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1-0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5}M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.79 (distortional slenderness)

M_{nd} = **131.68** kip-in (distortional reduction)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 131.68 kip-in (distortional controls)

Date:	11/26/14			calc by:	RKD	
				check by:	TS	

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	3					
	Gage:	20	GA				
	Strength:	33	KSI				
	M_y =	59.96	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	2.13000	$M_{cr\ell}$ =	127.7148	kip-in	2	in
dist.	M_{crd}/M_y =	2.18000	M_{crd} =	130.7128	kip-in	12	in
global	M_{cre}/M_y =	4.00000	M_{cre} =	239.84	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 59.96 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.69 (local-global slenderness)

$M_{n\ell}$ = 59.96 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.68 (distortional slenderness)

M_{nd} = 59.77 kip-in (distortional reduction)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 59.77 kip-in (distortional controls)

Date:	11/26/14		calc by:	RKD
			check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	3						
	Gage:	20	GA					
	Strength:	40	KSI					
	M_y =	72.68	kip-in				Length:	
local	$M_{cr\ell}/M_y$ =	1.76000	$M_{cr\ell}$ =	127.9168	kip-in		2	in
dist.	M_{crd}/M_y =	1.80000	M_{crd} =	130.824	kip-in		12	in
global	M_{cre}/M_y =	4.00000	M_{cre} =	290.72	kip-in		-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 72.68 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.75 (local-global slenderness)

$M_{n\ell}$ = 72.68 kip-in (fully effective section for local buckling)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.75 (distortional slenderness)

M_{nd} = 68.73 kip-in (distortional reduction)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 68.73 kip-in (distortional controls)

Date:	11/26/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	3				Length:	
	Gage:	20	GA				
	Strength:	50	KSI				
	M_y =	90.85	kip-in				
local	$M_{cr\ell}/M_y$ =	1.40000	$M_{cr\ell}$ =	127.19	kip-in	2	in
dist.	M_{crd}/M_y =	1.44000	M_{crd} =	130.824	kip-in	12	in
global	M_{cre}/M_y =	4.00000	M_{cre} =	363.4	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 90.85 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ $\qquad (Eq.\ 1.2.2\text{-}7)$

λ_ℓ = 0.85 (local-global slenderness)

$M_{n\ell}$ = 86.10 kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ $\qquad (Eq.\ 1.2.2\text{-}10)$

λ_d = 0.83 (distortional slenderness)

M_{nd} = 80.24 kip-in (distortional reduction)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 80.24 kip-in (distortional controls)

Date:	11/26/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	3					
	Gage:	20 GA					
	Strength:	60 KSI					
	M_y =	109.02	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	1.17000	$M_{cr\ell}$ =	127.5534	kip-in	2	in
dist.	M_{crd}/M_y =	1.20000	M_{crd} =	130.824	kip-in	12	in
global	M_{cre}/M_y =	4.00000	M_{cre} =	436.08	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1-\frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 109.02 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell}=M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell}=\left(1-0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ $\qquad (Eq.\ 1.2.2\text{-}7)$

λ_ℓ = 0.92 (local-global slenderness)

$M_{n\ell}$ = 97.54 kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1-0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5}M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ $\qquad (Eq.\ 1.2.2\text{-}10)$

λ_d = 0.91 (distortional slenderness)

M_{nd} = 90.64 kip-in (distortional reduction)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 90.64 kip-in (distortional controls)

Date:	11/26/14					calc by:	RKD	
						check by:	TS	

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	3						
	Gage:	22	GA					
	Strength:	33	KSI					
	M_y =	49.53	kip-in				Length:	
local	$M_{cr\ell}/M_y$ =	1.47000	$M_{cr\ell}$ =	72.8091	kip-in		2	in
dist.	M_{crd}/M_y =	1.77000	M_{crd} =	87.6681	kip-in		12	in
global	M_{cre}/M_y =	4.00000	M_{cre} =	198.12	kip-in		-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1-\frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 49.53 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell}=M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell}=\left(1-0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ $\qquad (Eq.\ 1.2.2\text{-}7)$

λ_ℓ = 0.82 (local-global slenderness)

$M_{n\ell}$ = 47.67 kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1-0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5}M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ $\qquad (Eq.\ 1.2.2\text{-}10)$

λ_d = 0.75 (distortional slenderness)

M_{nd} = 46.61 kip-in (distortional reduction)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 46.61 kip-in (distortional controls)

Date:	11/26/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	3					
	Gage:	22	GA				
	Strength:	40	KSI				
	M_y =	60.04	kip-in			Length:	
local	$M_{cr\ell}/M_y$ =	1.22000	$M_{cr\ell}$ =	73.2488	kip-in	2	in
dist.	M_{crd}/M_y =	1.46000	M_{crd} =	87.6584	kip-in	12	in
global	M_{cre}/M_y =	4.00000	M_{cre} =	240.16	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 60.04 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 0.91 (local-global slenderness)

$M_{n\ell}$ = 54.45 kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.83 (distortional slenderness)

M_{nd} = 53.26 kip-in (distortional reduction)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 53.26 kip-in (distortional controls)

Date:	11/26/14		calc by:	RKD
			check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:								
	Deck:	3						
	Gage:	22	GA					
	Strength:	50	KSI					
	M_y =	75.05	kip-in				Length:	
local	$M_{cr\ell}/M_y$ =	0.97344	$M_{cr\ell}$ =	73.056672	kip-in		2	in
dist.	M_{crd}/M_y =	1.17000	M_{crd} =	87.8085	kip-in		12	in
global	M_{cre}/M_y =	4.00000	M_{cre} =	300.2	kip-in		-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \quad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \quad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \quad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = 75.05 kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \quad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \quad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (Eq. 1.2.2-7)

λ_ℓ = 1.01 (local-global slenderness)

$M_{n\ell}$ = 63.23 kip-in (local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \quad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \quad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (Eq. 1.2.2-10)

λ_d = 0.92 (distortional slenderness)

M_{nd} = 61.86 kip-in (distortional reduction)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 61.86 kip-in (distortional controls)

Date:	11/26/14	calc by:	RKD
		check by:	TS

Deck Strength Calculations using the Direct Strength Method of Appendix 1

Given:	Deck:	3				Length:	
	Gage:	22 GA					
	Strength:	60 KSI					
	M_y =	90.06	kip-in				
local	$M_{cr\ell}/M_y$ =	0.81120	$M_{cr\ell}$ =	73.056672	kip-in	2	in
dist.	M_{crd}/M_y =	0.98000	M_{crd} =	88.2588	kip-in	12	in
global	M_{cre}/M_y =	4.00000	M_{cre} =	360.24	kip-in	-	in

Lateral-torsional buckling nominal flexural strength per DSM 1.2.2.1

for $M_{cre} < 0.56M_y$

$$M_{ne} = M_{cre} \qquad (Eq.\ 1.2.2\text{-}1)$$

for $2.78M_y \geq M_{cre} \geq 0.56M_y$

$$M_{ne} = \frac{10}{9}M_y\left(1 - \frac{10M_y}{36M_{cre}}\right) \qquad (Eq.\ 1.2.2\text{-}2)$$

for $M_{cre} > 2.78M_y$

$$M_{ne} = M_y \qquad (Eq.\ 1.2.2\text{-}3)$$

M_{ne} = **90.06** kip-in

Local buckling nominal flexural strength per DSM 1.2.2.2

for $\lambda_\ell \leq 0.776$

$$M_{n\ell} = M_{ne} \qquad (Eq.\ 1.2.2\text{-}5)$$

for $\lambda_\ell > 0.776$

$$M_{n\ell} = \left(1 - 0.15\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4}\right)\left(\frac{M_{cr\ell}}{M_{ne}}\right)^{0.4} M_{ne} \qquad (Eq.\ 1.2.2\text{-}6)$$

where $\lambda_\ell = \sqrt{M_{ne}/M_{cr\ell}}$ (*Eq.* 1.2.2-7)

λ_ℓ =	1.11	(local-global slenderness)
$M_{n\ell}$ =	**71.40** kip-in	(local-global interaction reduction)

Distortional buckling nominal flexural strength per DSM 1.2.2.3

for $\lambda_d \leq 0.673$

$$M_{nd} = M_y \qquad (Eq.\ 1.2.2\text{-}8)$$

for $\lambda_d > 0.673$

$$M_{nd} = \left(1 - 0.22\left(\frac{M_{crd}}{M_y}\right)^{0.5}\right)\left(\frac{M_{crd}}{M_y}\right)^{0.5} M_y \qquad (Eq.\ 1.2.2\text{-}9)$$

where $\lambda_d = \sqrt{M_y/M_{crd}}$ (*Eq.* 1.2.2-10)

λ_d =	1.01	(distortional slenderness)
M_{nd} =	**69.74** kip-in	(distortional reduction)

Nominal flexural strength of the beam per DSM 1.2.2

M_n = 69.74 kip-in (distortional controls)

9.4 Effective Width Method Calculations

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	3		bo:	4.499	in.
Gage:	16	GA	h:	3.000	in.
Strength:	33	ksi	bp:	1.687	in.
Thickness:	0.0598	in.	Ag:	0.281	$in.^2$
Total Height:	3.060	in.	n:	1	
Radius:	0.2179	in.			
θ:	67.38	deg	$θ_{Stiff}$:	36.501	deg
θ:	1.176	rad	$θ_{Stiff}$:	0.637	rad
Curve I'_x:	0.000424	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000023	$in.^3$

Guess ȳ: 1.573 in.

Stress in Flange:	33.000	ksi	δ:	1.044	
k:	11.662		Isp:	0.003	$in.^4$
Fcr:	54.932	ksi	γ:	35.413	
λ:	0.775		β:	2.911	
ρ:	0.924		kd:	6.137	
Effective Width:	4.339	in.	kloc:	28.449	
			R:	1.900	

Element	L (in.)	y from top (in.)
Lip	2.250	3.030
Corners	0.256	0.201
$Corners_{Stiff}$	0.139	0.233
Bottom Flange	1.687	3.030
Web	3.000	1.530
Web_{Stiff}	0.564	0.215
Top Flange	4.339	0.077

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	4.500	3.030	13.635	41.311	--
Bottom Corner	6	1.538	2.859	4.396	12.566	0.003
Web	6	18.000	1.530	27.538	42.131	11.503
Top Corner	6	1.538	0.201	0.309	0.062	0.003
Top Flange	3	13.017	0.077	0.998	0.076	--
Bottom Flange	4	6.748	3.030	20.446	61.949	--
Low $Corner_{Stiff}$	4	0.555	2.826	1.569	4.436	0.000
Web_{Stiff}	4	2.254	2.845	6.412	18.243	0.021
High $Corner_{Stiff}$	4	0.555	2.397	1.331	3.189	0.000
	Σ	48.705		76.633	183.963	11.529

Solved ȳ =	ΣLy/ΣL =	1.573	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	1.573	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	4.480	$in.^4$
S_e =	$I_x/ȳ$ =	2.847	$in.^3$
Mn =	**Se*Fy =**	**93.96**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	3		bo:	4.499	in.
Gage:	16	GA	h:	3.000	in.
Strength:	40	ksi	bp:	1.687	in.
Thickness:	0.0598	in.	Ag:	0.281	$in.^2$
Total Height:	3.060	in.	n:	1	
Radius:	0.2179	in.			
θ:	67.38	deg	θ_{Stiff}:	36.501	deg
θ:	1.176	rad	θ_{Stiff}:	0.637	rad
Curve I'_x:	0.000424	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000023	$in.^3$

Guess ȳ: 1.597 in.

Stress in Flange:	40.000	ksi	δ:	1.044	
k:	11.662		Isp:	0.003	$in.^4$
Fcr:	54.932	ksi	γ:	35.413	
λ:	0.853		β:	2.911	
ρ:	0.870		kd:	6.137	
Effective Width:	4.084	in.	kloc:	28.449	
			R:	1.900	

Element	L (in.)	y from top (in.)
Lip	2.250	3.030
Corners	0.256	0.201
$Corners_{Stiff}$	0.139	0.233
Bottom Flange	1.687	3.030
Web	3.000	1.530
Web_{Stiff}	0.564	0.215
Top Flange	4.084	0.077

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	ΣLy^2 ($in.^3$)	$\Sigma I'_x$ ($in.^3$)
Lip	2	4.500	3.030	13.635	41.311	--
Bottom Corner	6	1.538	2.859	4.396	12.566	0.003
Web	6	18.000	1.530	27.538	42.131	11.503
Top Corner	6	1.538	0.201	0.309	0.062	0.003
Top Flange	3	12.253	0.077	0.939	0.072	--
Bottom Flange	4	6.748	3.030	20.446	61.949	--
Low $Corner_{Stiff}$	4	0.555	2.826	1.569	4.436	0.000
Web_{Stiff}	4	2.254	2.845	6.412	18.243	0.021
High $Corner_{Stiff}$	4	0.555	2.397	1.331	3.189	0.000
	Σ	47.941		76.575	183.959	11.529

Solved ȳ =	ΣLy/ΣL =	1.597	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	1.597	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2 \Sigma L]t$ =	4.376	$in.^4$
S_e =	$I_x/\bar{y}$ =	2.740	$in.^3$
Mn =	**Se*Fy =**	**109.58**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	3		bo:	4.499	in.
Gage:	16	GA	h:	3.000	in.
Strength:	50	ksi	bp:	1.687	in.
Thickness:	0.0598	in.	Ag:	0.281	$in.^2$
Total Height:	3.060	in.	n:	1	
Radius:	0.2179	in.			
θ:	67.38	deg	$θ_{Stiff}$:	36.501	deg
θ:	1.176	rad	$θ_{Stiff}$:	0.637	rad
Curve I'_x:	0.000424	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000023	$in.^3$

Guess ȳ: 1.626 in.

Stress in Flange:	50.000	ksi	δ:	1.044	
k:	11.662		Isp:	0.003	$in.^4$
Fcr:	54.932	ksi	γ:	35.413	
λ:	0.954		β:	2.911	
ρ:	0.806		kd:	6.137	
Effective Width:	3.787	in.	kloc:	28.449	
			R:	1.900	

Element	L (in.)	y from top (in.)
Lip	2.250	3.030
Corners	0.256	0.201
$Corners_{Stiff}$	0.139	0.233
Bottom Flange	1.687	3.030
Web	3.000	1.530
Web_{Stiff}	0.564	0.215
Top Flange	3.787	0.077

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	4.500	3.030	13.635	41.311	--
Bottom Corner	6	1.538	2.859	4.396	12.566	0.003
Web	6	18.000	1.530	27.538	42.131	11.503
Top Corner	6	1.538	0.201	0.309	0.062	0.003
Top Flange	3	11.361	0.077	0.871	0.067	--
Bottom Flange	4	6.748	3.030	20.446	61.949	--
Low $Corner_{Stiff}$	4	0.555	2.826	1.569	4.436	0.000
Web_{Stiff}	4	2.254	2.845	6.412	18.243	0.021
High $Corner_{Stiff}$	4	0.555	2.397	1.331	3.189	0.000
	Σ	47.049		76.507	183.954	11.529

Solved ȳ =	ΣLy/ΣL =	1.626	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	1.626	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	4.250	$in.^4$
S_e =	$I_x/ȳ$ =	2.614	$in.^3$
Mn =	**Se*Fy =**	**130.69**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	3		bo:	4.499	in.
Gage:	16	GA	h:	3.000	in.
Strength:	60	ksi	bp:	1.687	in.
Thickness:	0.0598	in.	Ag:	0.281	$in.^2$
Total Height:	3.060	in.	n:	1	
Radius:	0.2179	in.			
θ:	67.38	deg	$θ_{Stiff}$:	36.501	deg
θ:	1.176	rad	$θ_{Stiff}$:	0.637	rad
Curve I'_x:	0.000424	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000023	$in.^3$

Guess ȳ: 1.650 in.

Stress in Flange:	60.000	ksi	δ:	1.044	
k:	11.662		Isp:	0.003	$in.^4$
Fcr:	54.932	ksi	γ:	35.413	
λ:	1.045		β:	2.911	
ρ:	0.755		kd:	6.137	
Effective Width:	3.547	in.	kloc:	28.449	
			R:	1.900	

Element	L (in.)	y from top (in.)
Lip	2.250	3.030
Corners	0.256	0.201
$Corners_{Stiff}$	0.139	0.233
Bottom Flange	1.687	3.030
Web	3.000	1.530
Web_{Stiff}	0.564	0.215
Top Flange	3.547	0.077

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	4.500	3.030	13.635	41.311	--
Bottom Corner	6	1.538	2.859	4.396	12.566	0.003
Web	6	18.000	1.530	27.538	42.131	11.503
Top Corner	6	1.538	0.201	0.309	0.062	0.003
Top Flange	3	10.642	0.077	0.816	0.063	--
Bottom Flange	4	6.748	3.030	20.446	61.949	--
Low $Corner_{Stiff}$	4	0.555	2.826	1.569	4.436	0.000
Web_{Stiff}	4	2.254	2.845	6.412	18.243	0.021
High $Corner_{Stiff}$	4	0.555	2.397	1.331	3.189	0.000
	Σ	46.330		76.451	183.949	11.529

Solved ȳ =	ΣLy/ΣL =	1.650	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	1.650	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	4.145	$in.^4$
S_e =	I_x/ȳ =	2.512	$in.^3$
Mn =	**Se*Fy =**	**150.73**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	3		bo:	4.499	in.
Gage:	18	GA	h:	3.000	in.
Strength:	33	ksi	bp:	1.687	in.
Thickness:	0.0474	in.	Ag:	0.223	$in.^2$
Total Height:	3.047	in.	n:	1	
Radius:	0.2117	in.			
θ:	67.38	deg	θ_{Stiff}:	36.501	deg
θ:	1.176	rad	θ_{Stiff}:	0.637	rad
Curve I'_x:	0.000389	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000021	$in.^3$

Guess ȳ: 1.600 in.

Stress in Flange:	33.000	ksi	δ:	1.044	
k:	14.296		Isp:	0.002	$in.^4$
Fcr:	42.308	ksi	γ:	55.839	
λ:	0.883		β:	3.258	
ρ:	0.850		kd:	7.524	
Effective Width:	3.993	in.	kloc:	28.449	
			R:	1.900	

Element	L (in.)	y from top (in.)
Lip	2.250	3.024
Corners	0.249	0.190
$Corners_{Stiff}$	0.135	0.221
Bottom Flange	1.687	3.024
Web	3.000	1.524
Web_{Stiff}	0.564	0.209
Top Flange	3.993	0.070

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	ΣLy^2 ($in.^3$)	$\Sigma I'_x$ ($in.^3$)
Lip	2	4.500	3.024	13.607	41.142	--
Bottom Corner	6	1.494	2.858	4.268	12.197	0.002
Web	6	18.000	1.524	27.427	41.790	11.503
Top Corner	6	1.494	0.190	0.284	0.054	0.002
Top Flange	3	11.978	0.070	0.841	0.059	--
Bottom Flange	4	6.748	3.024	20.404	61.695	--
Low $Corner_{Stiff}$	4	0.539	2.826	1.525	4.308	0.000
Web_{Stiff}	4	2.254	2.839	6.398	18.163	0.021
High $Corner_{Stiff}$	4	0.539	2.409	1.299	3.130	0.000
	Σ	47.546		76.053	182.539	11.529

Solved ȳ =	ΣLy/ΣL =	1.600	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	1.600	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2 \Sigma L]t$ =	3.433	$in.^4$
S_e =	$I_x/\bar{y}$ =	2.146	$in.^3$
Mn =	**Se*Fy =**	**70.82**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	3		bo:	4.499	in.
Gage:	18	GA	h:	3.000	in.
Strength:	40	ksi	bp:	1.687	in.
Thickness:	0.0474	in.	Ag:	0.223	$in.^2$
Total Height:	3.047	in.	n:	1	
Radius:	0.2117	in.			
θ:	67.38	deg	$θ_{Stiff}$:	36.501	deg
θ:	1.176	rad	$θ_{Stiff}$:	0.637	rad
Curve I'_x:	0.000389	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000021	$in.^3$

Guess ȳ: 1.625 in.

Stress in Flange:	40.000	ksi	δ:	1.044	
k:	14.296		Isp:	0.002	$in.^4$
Fcr:	42.308	ksi	γ:	55.839	
λ:	0.972		β:	3.258	
ρ:	0.796		kd:	7.524	
Effective Width:	3.737	in.	kloc:	28.449	
			R:	1.900	

Element	L (in.)	y from top (in.)
Lip	2.250	3.024
Corners	0.249	0.190
$Corners_{Stiff}$	0.135	0.221
Bottom Flange	1.687	3.024
Web	3.000	1.524
Web_{Stiff}	0.564	0.209
Top Flange	3.737	0.070

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	4.500	3.024	13.607	41.142	--
Bottom Corner	6	1.494	2.858	4.268	12.197	0.002
Web	6	18.000	1.524	27.427	41.790	11.503
Top Corner	6	1.494	0.190	0.284	0.054	0.002
Top Flange	3	11.211	0.070	0.787	0.055	--
Bottom Flange	4	6.748	3.024	20.404	61.695	--
Low $Corner_{Stiff}$	4	0.539	2.826	1.525	4.308	0.000
Web_{Stiff}	4	2.254	2.839	6.398	18.163	0.021
High $Corner_{Stiff}$	4	0.539	2.409	1.299	3.130	0.000
	Σ	46.779		75.999	182.535	11.529

Solved ȳ =	ΣLy/ΣL =	1.625	in.
$ȳ_{EXTREME FIBER}$ =	max(ȳ , h - ȳ) =	1.625	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	3.346	$in.^4$
S_e =	$I_x/ȳ$ =	2.060	$in.^3$
Mn =	**Se*Fy =**	**82.39**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	3		bo:	4.499	in.
Gage:	18	GA	h:	3.000	in.
Strength:	50	ksi	bp:	1.687	in.
Thickness:	0.0474	in.	Ag:	0.223	$in.^2$
Total Height:	3.047	in.	n:	1	
Radius:	0.2117	in.			
θ:	67.38	deg	θ_{Stiff}:	36.501	deg
θ:	1.176	rad	θ_{Stiff}:	0.637	rad
Curve I'_x:	0.000389	$in.^3$	$Curve_{Stiff}\ I'_x$:	0.000021	$in.^3$

Guess ȳ: 1.654 in.

Stress in Flange:	50.000	ksi	δ:	1.044	
k:	14.296		Isp:	0.002	$in.^4$
Fcr:	42.308	ksi	γ:	55.839	
λ:	1.087		β:	3.258	
ρ:	0.734		kd:	7.524	
Effective Width:	3.446	in.	kloc:	28.449	
			R:	1.900	

Element	L (in.)	y from top (in.)
Lip	2.250	3.024
Corners	0.249	0.190
$Corners_{Stiff}$	0.135	0.221
Bottom Flange	1.687	3.024
Web	3.000	1.524
Web_{Stiff}	0.564	0.209
Top Flange	3.446	0.070

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	ΣLy^2 ($in.^3$)	$\Sigma I'_x$ ($in.^3$)
Lip	2	4.500	3.024	13.607	41.142	--
Bottom Corner	6	1.494	2.858	4.268	12.197	0.002
Web	6	18.000	1.524	27.427	41.790	11.503
Top Corner	6	1.494	0.190	0.284	0.054	0.002
Top Flange	3	10.337	0.070	0.726	0.051	--
Bottom Flange	4	6.748	3.024	20.404	61.695	--
Low $Corner_{Stiff}$	4	0.539	2.826	1.525	4.308	0.000
Web_{Stiff}	4	2.254	2.839	6.398	18.163	0.021
High $Corner_{Stiff}$	4	0.539	2.409	1.299	3.130	0.000
	Σ	45.905		75.937	182.531	11.529

Solved ȳ =	ΣLy/ΣL =	1.654	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	1.654	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2 \Sigma L]t$ =	3.244	$in.^4$
S_e =	$I_x/\bar{y}$ =	1.961	$in.^3$
Mn =	**Se*Fy =**	**98.06**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	3		bo:	4.499	in.
Gage:	18	GA	h:	3.000	in.
Strength:	60	ksi	bp:	1.687	in.
Thickness:	0.0474	in.	Ag:	0.223	$in.^2$
Total Height:	3.047	in.	n:	1	
Radius:	0.2117	in.			
θ:	67.38	deg	$θ_{Stiff}$:	36.501	deg
θ:	1.176	rad	$θ_{Stiff}$:	0.637	rad
Curve I'_x:	0.000389	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000021	$in.^3$

Guess ȳ: 1.678 in.

Stress in Flange:	60.000	ksi	δ:	1.044	
k:	14.296		Isp:	0.002	$in.^4$
Fcr:	42.308	ksi	γ:	55.839	
λ:	1.191		β:	3.258	
ρ:	0.685		kd:	7.524	
Effective Width:	3.215	in.	kloc:	28.449	
			R:	1.900	

Element	L (in.)	y from top (in.)
Lip	2.250	3.024
Corners	0.249	0.190
$Corners_{Stiff}$	0.135	0.221
Bottom Flange	1.687	3.024
Web	3.000	1.524
Web_{Stiff}	0.564	0.209
Top Flange	3.215	0.070

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	4.500	3.024	13.607	41.142	--
Bottom Corner	6	1.494	2.858	4.268	12.197	0.002
Web	6	18.000	1.524	27.427	41.790	11.503
Top Corner	6	1.494	0.190	0.284	0.054	0.002
Top Flange	3	9.645	0.070	0.677	0.048	--
Bottom Flange	4	6.748	3.024	20.404	61.695	--
Low $Corner_{Stiff}$	4	0.539	2.826	1.525	4.308	0.000
Web_{Stiff}	4	2.254	2.839	6.398	18.163	0.021
High $Corner_{Stiff}$	4	0.539	2.409	1.299	3.130	0.000
	Σ	45.213		75.889	182.528	11.529

Solved ȳ =	ΣLy/ΣL =	1.678	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	1.678	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	3.161	$in.^4$
S_e =	$I_x/ȳ$ =	1.883	$in.^3$
Mn =	**Se*Fy =**	**112.98**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	3		bo:	4.499	in.
Gage:	20	GA	h:	3.000	in.
Strength:	33	ksi	bp:	1.687	in.
Thickness:	0.0358	in.	Ag:	0.168	$in.^2$
Total Height:	3.036	in.	n:	1	
Radius:	0.2059	in.			
θ:	67.38	deg	θ_{Stiff}:	36.501	deg
θ:	1.176	rad	θ_{Stiff}:	0.637	rad
Curve I'_x:	0.000358	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000019	$in.^3$

Guess ȳ: 1.634 in.

Stress in Flange:	33.000	ksi	δ:	1.044	
k:	18.401		Isp:	0.002	$in.^4$
Fcr:	31.065	ksi	γ:	96.807	
λ:	1.031		β:	3.735	
ρ:	0.763		kd:	9.684	
Effective Width:	3.584	in.	kloc:	28.449	
			R:	1.900	

Element	L (in.)	y from top (in.)
Lip	2.250	3.018
Corners	0.242	0.180
$Corners_{Stiff}$	0.131	0.210
Bottom Flange	1.687	3.018
Web	3.000	1.518
Web_{Stiff}	0.564	0.203
Top Flange	3.584	0.064

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	4.500	3.018	13.581	40.985	--
Bottom Corner	6	1.453	2.856	4.150	11.853	0.002
Web	6	18.000	1.518	27.322	41.472	11.503
Top Corner	6	1.453	0.180	0.261	0.047	0.002
Top Flange	3	10.751	0.064	0.690	0.044	--
Bottom Flange	4	6.748	3.018	20.365	61.459	--
$Low\ Corner_{Stiff}$	4	0.525	2.826	1.483	4.189	0.000
Web_{Stiff}	4	2.254	2.833	6.385	18.089	0.021
$High\ Corner_{Stiff}$	4	0.525	2.420	1.270	3.072	0.000
	Σ	46.208		75.506	181.211	11.528

Solved ȳ =	ΣLy/ΣL =	1.634	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	1.634	in.
I_x =	$[ΣI'_x + ΣLy^2 - \bar{y}^2ΣL]t$ =	2.483	$in.^4$
S_e =	$I_x/\bar{y}$ =	1.520	$in.^3$
Mn =	**Se*Fy =**	**50.15**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	3		bo:	4.499	in.
Gage:	20	GA	h:	3.000	in.
Strength:	40	ksi	bp:	1.687	in.
Thickness:	0.0358	in.	Ag:	0.168	$in.^2$
Total Height:	3.036	in.	n:	1	
Radius:	0.2059	in.			
θ:	67.38	deg	$θ_{Stiff}$:	36.501	deg
θ:	1.176	rad	$θ_{Stiff}$:	0.637	rad
Curve I'_x:	0.000358	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000019	$in.^3$

Guess ȳ: 1.660 in.

Stress in Flange:	40.000	ksi	δ:	1.044	
k:	18.401		Isp:	0.002	$in.^4$
Fcr:	31.065	ksi	γ:	96.807	
λ:	1.135		β:	3.735	
ρ:	0.710		kd:	9.684	
Effective Width:	3.336	in.	kloc:	28.449	
			R:	1.900	

Element	L (in.)	y from top (in.)
Lip	2.250	3.018
Corners	0.242	0.180
$Corners_{Stiff}$	0.131	0.210
Bottom Flange	1.687	3.018
Web	3.000	1.518
Web_{Stiff}	0.564	0.203
Top Flange	3.336	0.064

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	4.500	3.018	13.581	40.985	--
Bottom Corner	6	1.453	2.856	4.150	11.853	0.002
Web	6	18.000	1.518	27.322	41.472	11.503
Top Corner	6	1.453	0.180	0.261	0.047	0.002
Top Flange	3	10.008	0.064	0.642	0.041	--
Bottom Flange	4	6.748	3.018	20.365	61.459	--
Low $Corner_{Stiff}$	4	0.525	2.826	1.483	4.189	0.000
Web_{Stiff}	4	2.254	2.833	6.385	18.089	0.021
High $Corner_{Stiff}$	4	0.525	2.420	1.270	3.072	0.000
	Σ	45.465		75.458	181.207	11.528

Solved ȳ =	ΣLy/ΣL =	1.660	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	1.660	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	2.416	$in.^4$
S_e =	$I_x/ȳ$ =	1.456	$in.^3$
Mn =	**Se*Fy =**	**58.24**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	3		bo:	4.499	in.
Gage:	20	GA	h:	3.000	in.
Strength:	50	ksi	bp:	1.687	in.
Thickness:	0.0358	in.	Ag:	0.168	in.^{2}
Total Height:	3.036	in.	n:	1	
Radius:	0.2059	in.			
θ:	67.38	deg	θ_{Stiff}:	36.501	deg
θ:	1.176	rad	θ_{Stiff}:	0.637	rad
Curve I'_x:	0.000358	in.^{3}	Curve_{Stiff} I'_x:	0.000019	in.^{3}

Guess ȳ: 1.689 in.

Stress in Flange:	50.000	ksi	δ:	1.044	
k:	18.401		Isp:	0.002	in.^{4}
Fcr:	31.065	ksi	γ:	96.807	
λ:	1.269		β:	3.735	
ρ:	0.652		kd:	9.684	
Effective Width:	3.060	in.	kloc:	28.449	
			R:	1.900	

Element	L (in.)	y from top (in.)
Lip	2.250	3.018
Corners	0.242	0.180
Corners_{Stiff}	0.131	0.210
Bottom Flange	1.687	3.018
Web	3.000	1.518
Web_{Stiff}	0.564	0.203
Top Flange	3.060	0.064

Element	Quantity	ΣL	y from top fiber	ΣLy (in.^{2})	ΣLy^2 (in.^{3})	ΣI'$_x$ (in.^{3})
Lip	2	4.500	3.018	13.581	40.985	--
Bottom Corner	6	1.453	2.856	4.150	11.853	0.002
Web	6	18.000	1.518	27.322	41.472	11.503
Top Corner	6	1.453	0.180	0.261	0.047	0.002
Top Flange	3	9.179	0.064	0.589	0.038	--
Bottom Flange	4	6.748	3.018	20.365	61.459	--
Low Corner_{Stiff}	4	0.525	2.826	1.483	4.189	0.000
Web_{Stiff}	4	2.254	2.833	6.385	18.089	0.021
High Corner_{Stiff}	4	0.525	2.420	1.270	3.072	0.000
	Σ	44.636		75.405	181.204	11.528

Solved ȳ =	ΣLy/ΣL =	1.689	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	1.689	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2 \Sigma L]t$ =	2.339	in.^{4}
S_e =	$I_x/\bar{y}$ =	1.385	in.^{3}
Mn =	**Se*Fy =**	**69.24**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	3		bo:	4.499	in.
Gage:	20	GA	h:	3.000	in.
Strength:	60	ksi	bp:	1.687	in.
Thickness:	0.0358	in.	Ag:	0.168	$in.^2$
Total Height:	3.036	in.	n:	1	
Radius:	0.2059	in.			
θ:	67.38	deg	$θ_{Stiff}$:	36.501	deg
θ:	1.176	rad	$θ_{Stiff}$:	0.637	rad
Curve I'_x:	0.000358	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000019	$in.^3$

Guess ȳ: 1.713 in.

Stress in Flange:	60.000	ksi	δ:	1.044	
k:	18.401		Isp:	0.002	$in.^4$
Fcr:	31.065	ksi	γ:	96.807	
λ:	1.390		β:	3.735	
ρ:	0.606		kd:	9.684	
Effective Width:	2.844	in.	kloc:	28.449	
			R:	1.900	

Element	L (in.)	y from top (in.)
Lip	2.250	3.018
Corners	0.242	0.180
$Corners_{Stiff}$	0.131	0.210
Bottom Flange	1.687	3.018
Web	3.000	1.518
Web_{Stiff}	0.564	0.203
Top Flange	2.844	0.064

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	4.500	3.018	13.581	40.985	--
Bottom Corner	6	1.453	2.856	4.150	11.853	0.002
Web	6	18.000	1.518	27.322	41.472	11.503
Top Corner	6	1.453	0.180	0.261	0.047	0.002
Top Flange	3	8.532	0.064	0.548	0.035	--
Bottom Flange	4	6.748	3.018	20.365	61.459	--
Low $Corner_{Stiff}$	4	0.525	2.826	1.483	4.189	0.000
Web_{Stiff}	4	2.254	2.833	6.385	18.089	0.021
High $Corner_{Stiff}$	4	0.525	2.420	1.270	3.072	0.000
	Σ	43.989		75.363	181.201	11.528

Solved ȳ =	ΣLy/ΣL =	1.713	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	1.713	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	2.277	$in.^4$
S_e =	$I_x/ȳ$ =	1.329	$in.^3$
Mn =	**Se*Fy =**	**79.76**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	3		bo:	4.499	in.
Gage:	22	GA	h:	3.000	in.
Strength:	33	ksi	bp:	1.687	in.
Thickness:	0.0295	in.	Ag:	0.139	$in.^2$
Total Height:	3.030	in.	n:	1	
Radius:	0.20275	in.			
θ:	67.38	deg	$θ_{Stiff}$:	36.501	deg
θ:	1.176	rad	$θ_{Stiff}$:	0.637	rad
Curve I'_x:	0.000341	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000018	$in.^3$

Guess ȳ: 1.659 in.

Stress in Flange:	33.000	ksi	δ:	1.044	
k:	21.996		Isp:	0.002	$in.^4$
Fcr:	25.214	ksi	γ:	141.818	
λ:	1.144		β:	4.107	
ρ:	0.706		kd:	11.576	
Effective Width:	3.315	in.	kloc:	28.449	
			R:	1.900	

Element	L (in.)	y from top (in.)
Lip	2.250	3.015
Corners	0.238	0.174
$Corners_{Stiff}$	0.129	0.204
Bottom Flange	1.687	3.015
Web	3.000	1.515
Web_{Stiff}	0.564	0.200
Top Flange	3.315	0.061

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	4.500	3.015	13.566	40.899	--
Bottom Corner	6	1.431	2.856	4.085	11.666	0.002
Web	6	18.000	1.515	27.266	41.300	11.503
Top Corner	6	1.431	0.174	0.249	0.043	0.002
Top Flange	3	9.946	0.061	0.606	0.037	--
Bottom Flange	4	6.748	3.015	20.344	61.331	--
Low $Corner_{Stiff}$	4	0.517	2.825	1.460	4.125	0.000
Web_{Stiff}	4	2.254	2.830	6.378	18.049	0.021
High $Corner_{Stiff}$	4	0.517	2.426	1.253	3.041	0.000
	Σ	45.343		75.207	180.490	11.528

Solved ȳ =	ΣLy/ΣL =	1.659	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	1.659	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	1.985	$in.^4$
S_e =	$I_x/ȳ$ =	1.197	$in.^3$
Mn =	**Se*Fy =**	**39.49**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	3		bo:	4.499	in.
Gage:	22	GA	h:	3.000	in.
Strength:	40	ksi	bp:	1.687	in.
Thickness:	0.0295	in.	Ag:	0.139	$in.^2$
Total Height:	3.030	in.	n:	1	
Radius:	0.20275	in.			
θ:	67.38	deg	$θ_{Stiff}$:	36.501	deg
θ:	1.176	rad	$θ_{Stiff}$:	0.637	rad
Curve I'_x:	0.000341	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000018	$in.^3$

Guess ȳ: 1.684 in.

Stress in Flange:	40.000	ksi	δ:	1.044	
k:	21.996		Isp:	0.002	$in.^4$
Fcr:	25.214	ksi	γ:	141.818	
λ:	1.260		β:	4.107	
ρ:	0.655		kd:	11.576	
Effective Width:	3.077	in.	kloc:	28.449	
			R:	1.900	

Element	L (in.)	y from top (in.)
Lip	2.250	3.015
Corners	0.238	0.174
$Corners_{Stiff}$	0.129	0.204
Bottom Flange	1.687	3.015
Web	3.000	1.515
Web_{Stiff}	0.564	0.200
Top Flange	3.077	0.061

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	4.500	3.015	13.566	40.899	--
Bottom Corner	6	1.431	2.856	4.085	11.666	0.002
Web	6	18.000	1.515	27.266	41.300	11.503
Top Corner	6	1.431	0.174	0.249	0.043	0.002
Top Flange	3	9.231	0.061	0.562	0.034	--
Bottom Flange	4	6.748	3.015	20.344	61.331	--
Low $Corner_{Stiff}$	4	0.517	2.825	1.460	4.125	0.000
Web_{Stiff}	4	2.254	2.830	6.378	18.049	0.021
High $Corner_{Stiff}$	4	0.517	2.426	1.253	3.041	0.000
	Σ	44.628		75.163	180.488	11.528

Solved ȳ =	ΣLy/ΣL =	1.684	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	1.684	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	1.930	$in.^4$
S_e =	$I_x/ȳ$ =	1.146	$in.^3$
Mn =	**Se*Fy =**	**45.84**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	3		bo:	4.499	in.
Gage:	22	GA	h:	3.000	in.
Strength:	50	ksi	bp:	1.687	in.
Thickness:	0.0295	in.	Ag:	0.139	$in.^2$
Total Height:	3.030	in.	n:	1	
Radius:	0.20275	in.			
θ:	67.38	deg	$θ_{Stiff}$:	36.501	deg
θ:	1.176	rad	$θ_{Stiff}$:	0.637	rad
Curve I'_x:	0.000341	$in.^3$	$Curve_{Stiff}$ I'_x:	0.000018	$in.^3$

Guess ȳ: 1.713 in.

Stress in Flange:	50.000	ksi	δ:	1.044	
k:	21.996		Isp:	0.002	$in.^4$
Fcr:	25.214	ksi	γ:	141.818	
λ:	1.408		β:	4.107	
ρ:	0.599		kd:	11.576	
Effective Width:	2.814	in.	kloc:	28.449	
			R:	1.900	

Element	L (in.)	y from top (in.)
Lip	2.250	3.015
Corners	0.238	0.174
$Corners_{Stiff}$	0.129	0.204
Bottom Flange	1.687	3.015
Web	3.000	1.515
Web_{Stiff}	0.564	0.200
Top Flange	2.814	0.061

Element	Quantity	ΣL	y from top fiber	ΣLy ($in.^2$)	$ΣLy^2$ ($in.^3$)	$ΣI'_x$ ($in.^3$)
Lip	2	4.500	3.015	13.566	40.899	--
Bottom Corner	6	1.431	2.856	4.085	11.666	0.002
Web	6	18.000	1.515	27.266	41.300	11.503
Top Corner	6	1.431	0.174	0.249	0.043	0.002
Top Flange	3	8.441	0.061	0.514	0.031	--
Bottom Flange	4	6.748	3.015	20.344	61.331	--
$Low\ Corner_{Stiff}$	4	0.517	2.825	1.460	4.125	0.000
Web_{Stiff}	4	2.254	2.830	6.378	18.049	0.021
$High\ Corner_{Stiff}$	4	0.517	2.426	1.253	3.041	0.000
	Σ	43.838		75.115	180.485	11.528

Solved ȳ =	ΣLy/ΣL =	1.713	in.
$ȳ_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	1.713	in.
I_x =	$[ΣI'_x + ΣLy^2 - ȳ^2ΣL]t$ =	1.868	$in.^4$
S_e =	$I_x/ȳ$ =	1.090	$in.^3$
Mn =	**Se*Fy =**	**54.49**	**k-in.**

EFFECTIVE WIDTH METHOD
POSITIVE BENDING
CFS Floor & Roof Deck Sections

date: 12/11/2014
calc by: RKD
check by: TS

Deck:	3		bo:	4.499	in.
Gage:	22	GA	h:	3.000	in.
Strength:	60	ksi	bp:	1.687	in.
Thickness:	0.0295	in.	Ag:	0.139	in.^{2}
Total Height:	3.030	in.	n:	1	
Radius:	0.20275	in.			
θ:	67.38	deg	θ_{Stiff}:	36.501	deg
θ:	1.176	rad	θ_{Stiff}:	0.637	rad
Curve I'_x:	0.000341	in.^{3}	$Curve_{Stiff}$ I'_x:	0.000018	in.^{3}

Guess ȳ: 1.737 in.

Stress in Flange:	60.000	ksi	δ:	1.044	
k:	21.996		Isp:	0.002	in.^{4}
Fcr:	25.214	ksi	γ:	141.818	
λ:	1.543		β:	4.107	
ρ:	0.556		kd:	11.576	
Effective Width:	2.610	in.	kloc:	28.449	
			R:	1.900	

Element	L (in.)	y from top (in.)
Lip	2.250	3.015
Corners	0.238	0.174
$Corners_{Stiff}$	0.129	0.204
Bottom Flange	1.687	3.015
Web	3.000	1.515
Web_{Stiff}	0.564	0.200
Top Flange	2.610	0.061

Element	Quantity	ΣL	y from top fiber	ΣLy (in.^{2})	ΣLy^2 (in.^{3})	ΣI'$_x$ (in.^{3})
Lip	2	4.500	3.015	13.566	40.899	--
Bottom Corner	6	1.431	2.856	4.085	11.666	0.002
Web	6	18.000	1.515	27.266	41.300	11.503
Top Corner	6	1.431	0.174	0.249	0.043	0.002
Top Flange	3	7.830	0.061	0.477	0.029	--
Bottom Flange	4	6.748	3.015	20.344	61.331	--
Low $Corner_{Stiff}$	4	0.517	2.825	1.460	4.125	0.000
Web_{Stiff}	4	2.254	2.830	6.378	18.049	0.021
High $Corner_{Stiff}$	4	0.517	2.426	1.253	3.041	0.000
	Σ	43.227		75.078	180.483	11.528

Solved ȳ =	ΣLy/ΣL =	1.737	in.
$\bar{y}_{EXTREME\ FIBER}$ =	max(ȳ , h - ȳ) =	1.737	in.
I_x =	$[\Sigma I'_x + \Sigma Ly^2 - \bar{y}^2 \Sigma L]t$ =	1.818	in.^{4}
S_e =	$I_x/\bar{y}$ =	1.046	in.^{3}
Mn =	**Se*Fy =**	**62.79**	**k-in.**

CHAPTER 10: RESULTS

10.0 Comparison of Data

After running the DSM and EWM analyses, we have compared a couple of sets of data to observe trends between the various deck sections. The first data comparison plot shows the nominal moment capacity ratio of DSM to EWM, Mn_{DSM} / Mn_{EWM}, vs. the flat width of the compression flange over the thickness, b/t. The second data comparison plot shows the same relationship but now normalizing the nominal moment capacity ratio by the yield stress, (Mn_{DSM} / Mn_{EWM}) / Fy.

10.1 Comments on Results

From the first data comparison, we can see that DSM starts to predict lower strengths than EWM when b/t ratios exceed the 40-70 range for unstiffened deck sections. For the stiffened deck sections we see that the b/t ratio tops out around 55. DSM is able to take advantage of the lower range of b/t ratios and predict higher strengths than EWM. In the second data comparison with the normalized nominal moment capacity ratio, we can see the same decrease in DSM strength around the 40-70 b/t range. We can gather that DSM performs well for lower b/t ratios. We also saw that for certain cases, DSM predicted fully effective sections where the EWM did not.

10.2 Recommendations

To take advantage of the slight increase in strength with the DSM, consider using compression element stiffeners. By adding the stiffener to the compression element, the b/t ratio is reduced and as we have seen in this study, DSM predicts higher strengths than EWM for lower b/t ratios.

10.3 Future Work

The next step would be to conduct laboratory testing to verify the DSM strength results. Once our results are backed up with physical testing, potential enhancements to new deck profiles that may take advantage of DSM can be developed.

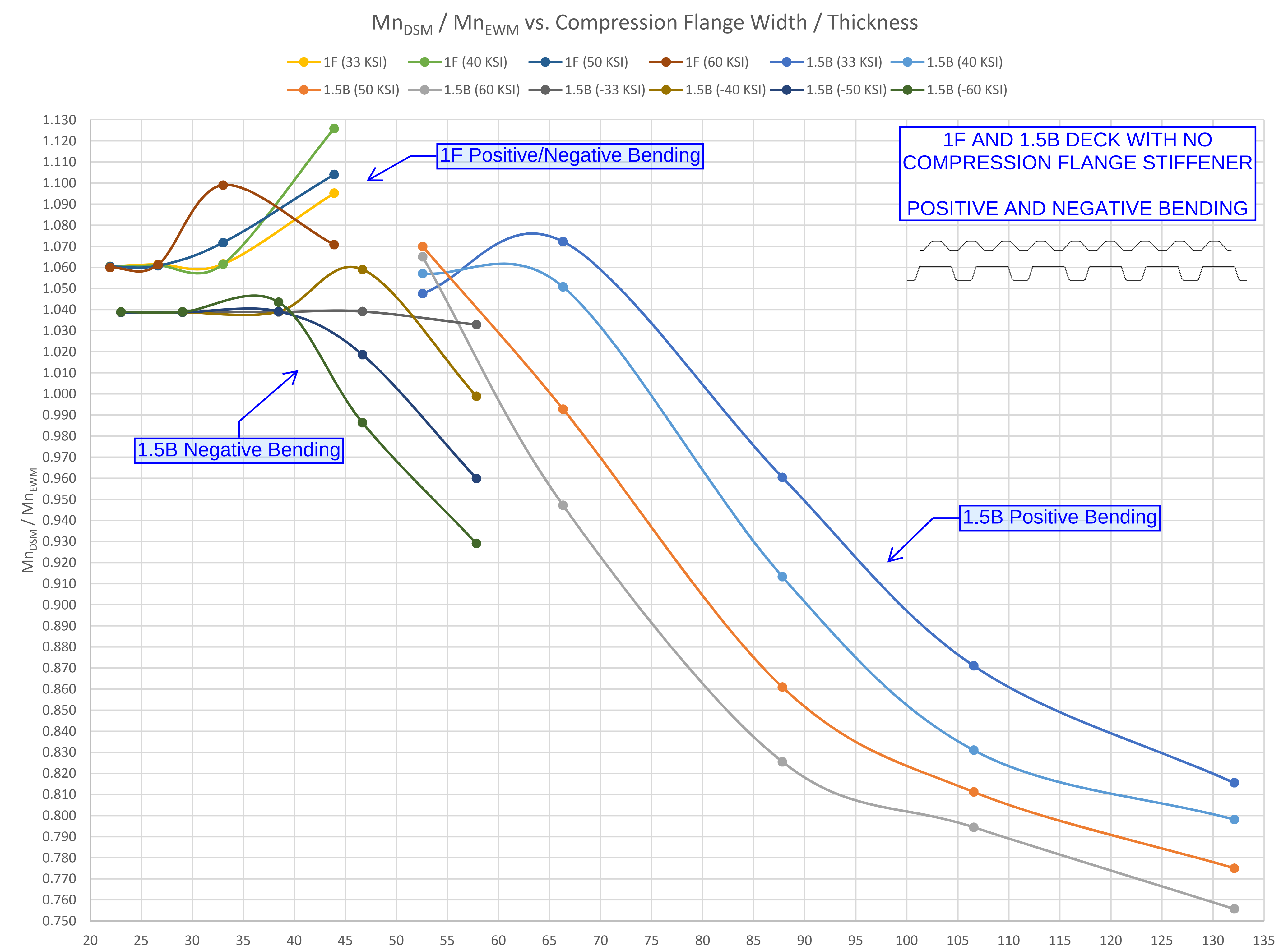


Figure 16 - Data Comparison (unstiffened sections) - MnDSM / MnEWM vs. b/t

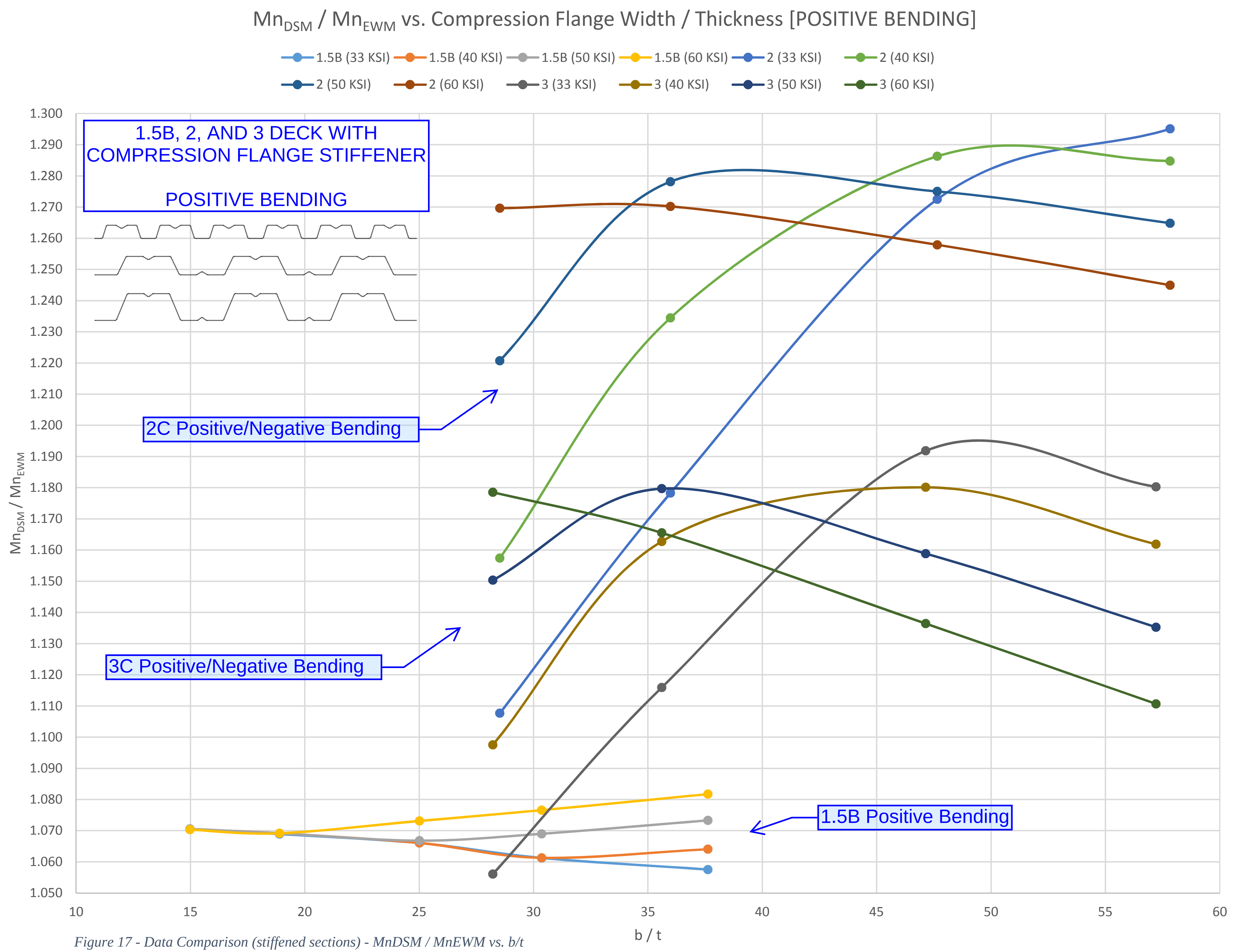


Figure 17 - Data Comparison (stiffened sections) - MnDSM / MnEWM vs. b/t

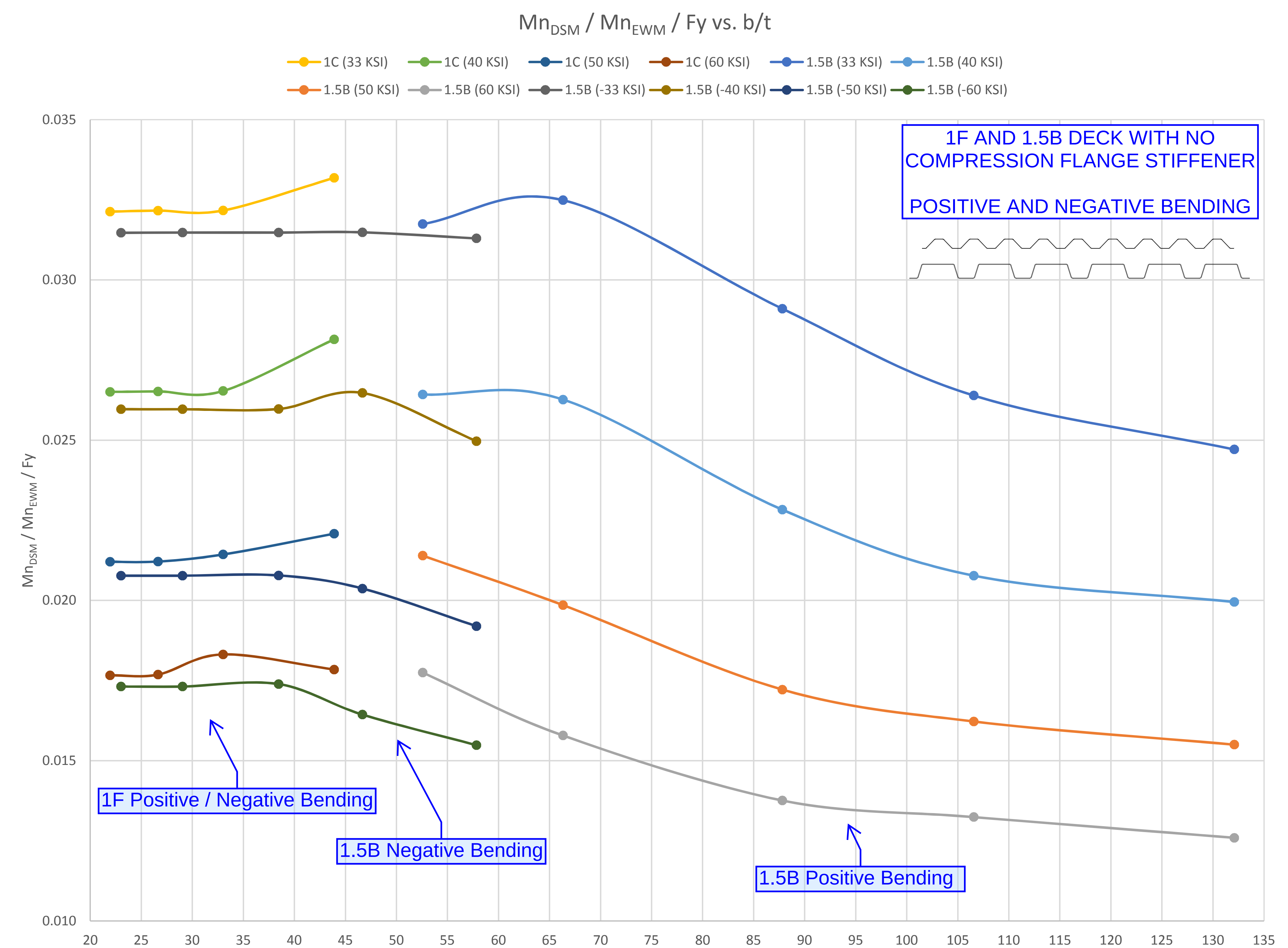


Figure 18 - Data Comparison (unstiffened sections) - (MnDSM / MnEWM) / Fy vs. b/t

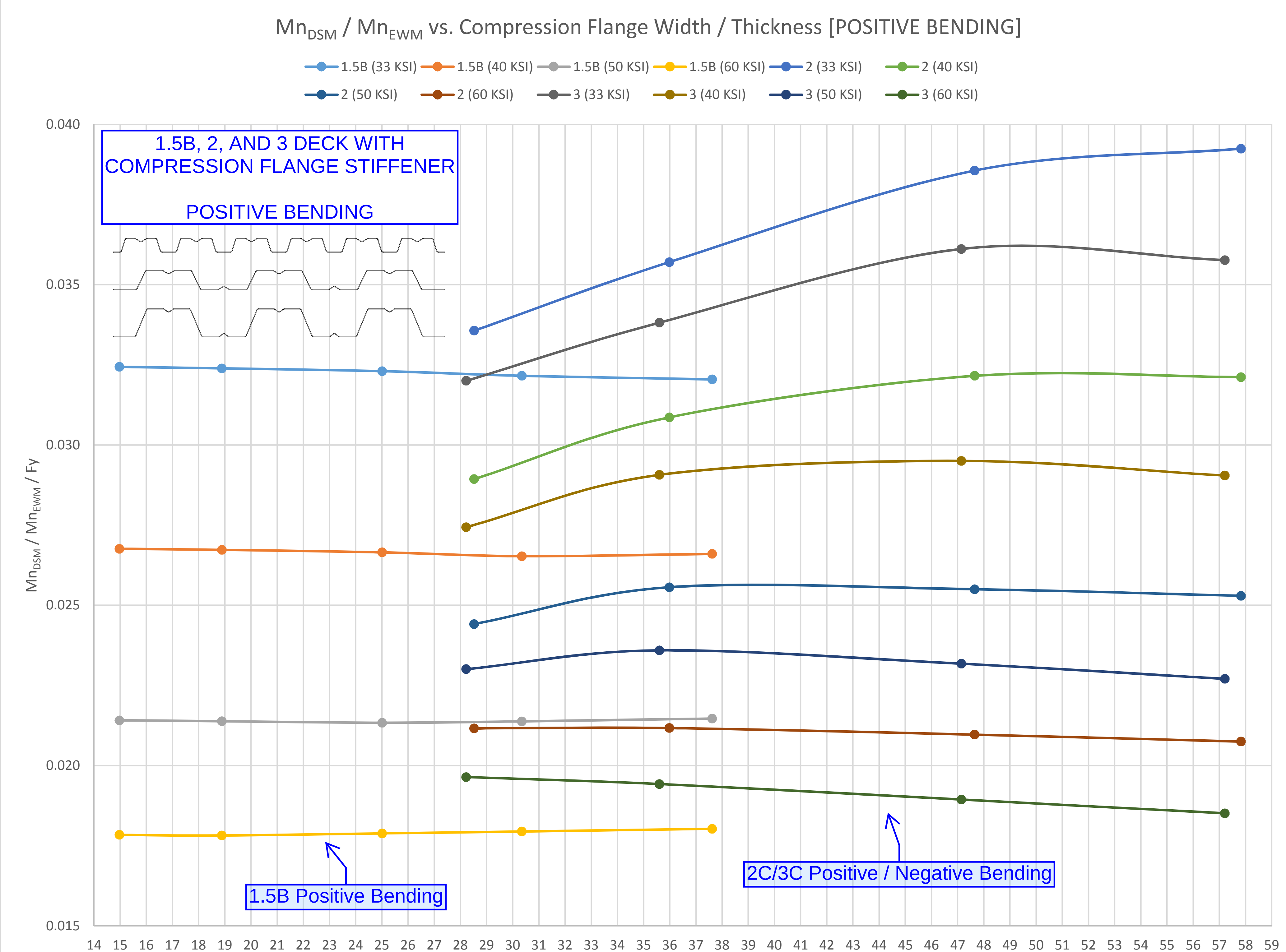


Figure 19 - Data Comparison (stiffened sections) - (MnDSM / MnEWM) / Fy vs. b/t

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