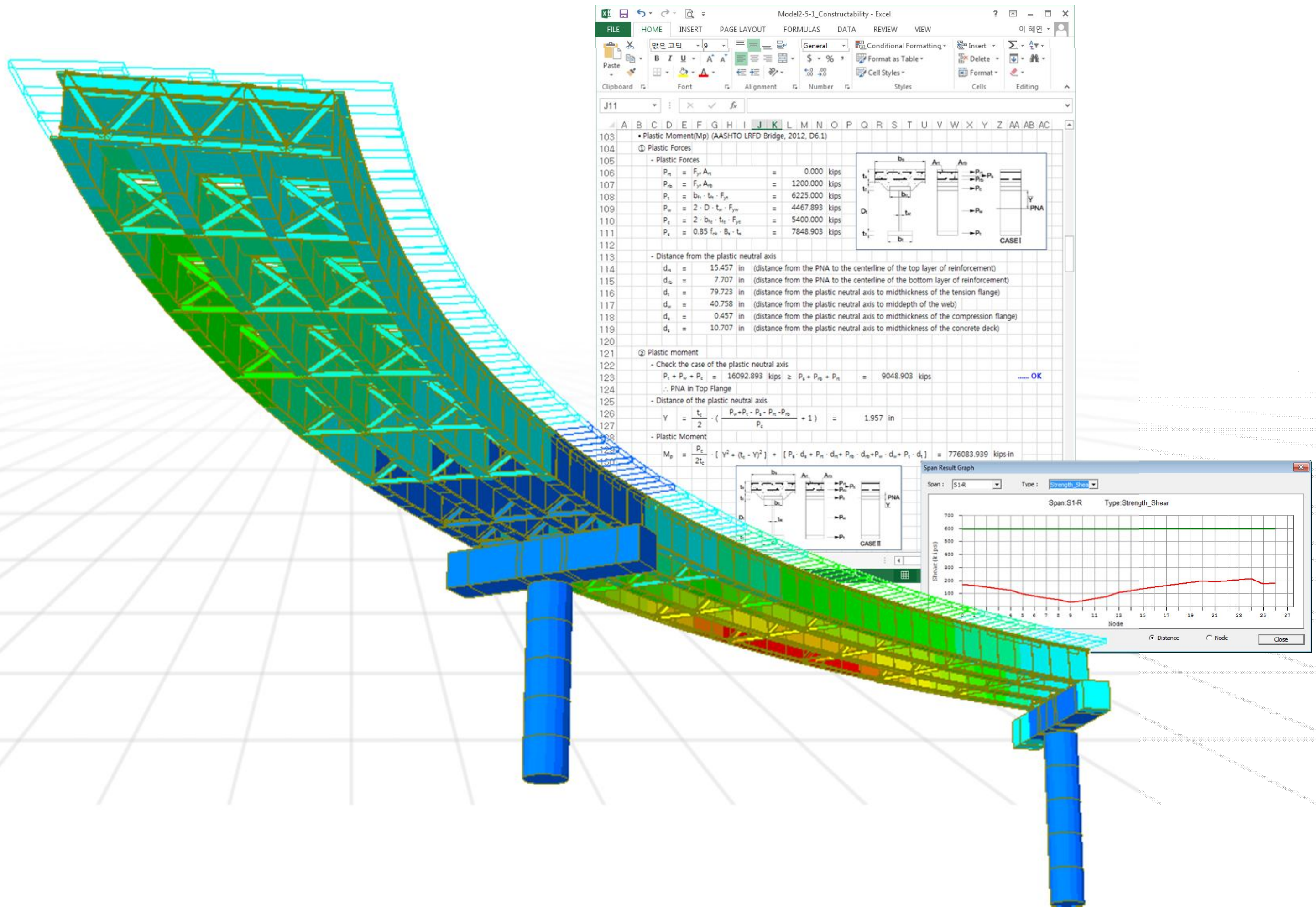
Curved Steel Composite I-Girder Bridge Design

Contents



Program Version	**2015 v1.1**
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http://en.midasuser.com

Midas Information Technology Co., Ltd.

Overview

This tutorial demonstrates the design capabilities of midas Civil for a steel composite I girder curved bridge.

Unless otherwise specified, the considerations comply with AASHTO LRFD 2012 Bridge Design Specification 6th Edition (US).

Bridge Specifications

Bridge Type :	2-Span Steel Composite I girder curved bridge
Number of main girder :	4, Steel Composite I girder
Curvature radius :	170’
Skew :	0^{o} (No skew)
Unbraced length :	223”
Longitudinal stiffener :	No
Shear connector :	Yes, 7/8” x 7”, Pitch = 5” (Section 2-2)
Transverse stiffener :	Yes, 1.5” x 5”, Fy = 36ksi, pitch = 90”
CS Analysis :	Yes
Time Dependent Material :	Long-term modular ratio of 3n considered through “Section Stiffness Scale Factor”.

Material Properties

Structural Steel

Web : ASTM09(S), A709, Grade HPS70W

Flange : ASTM09(S), A709, Grade 50W

Concrete

Pier & Pier Table : fc’ = 4.0ksi, ASTM(RC), Grade C4000

Deck : fc’ = 4.0ksi, ASTM(RC), Grade C4500

Reinforcing Steel

Main Rebar: ASTM(RC), Grade 60, Fy = 60ksi

Sub-Rebar : ASTM(RC), Grade 50, Fy = 50ksi

Bridge Specifications

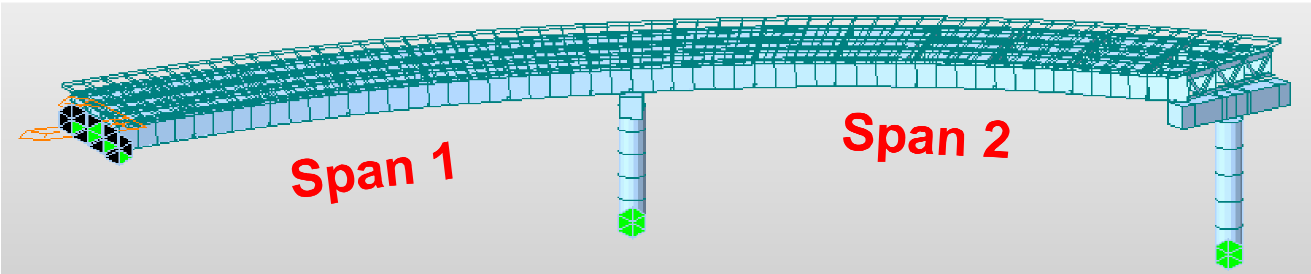


Image 1-1. 2 Span Ramp

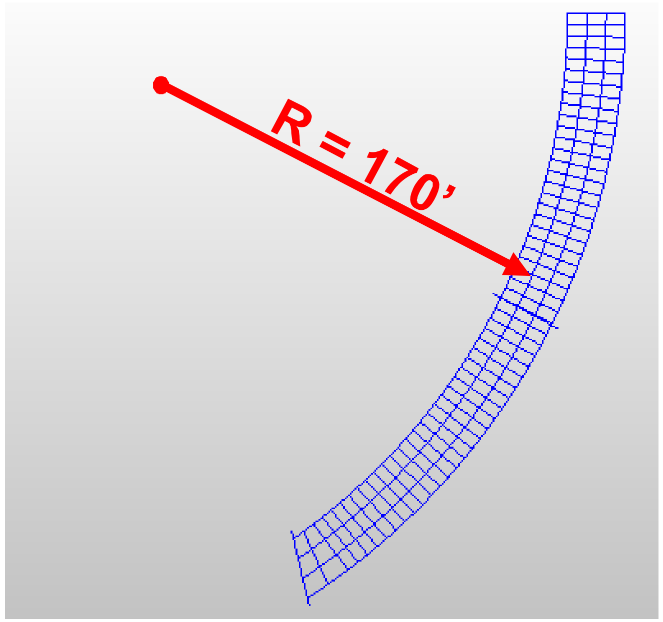


Image 1-3. Curvature Radius

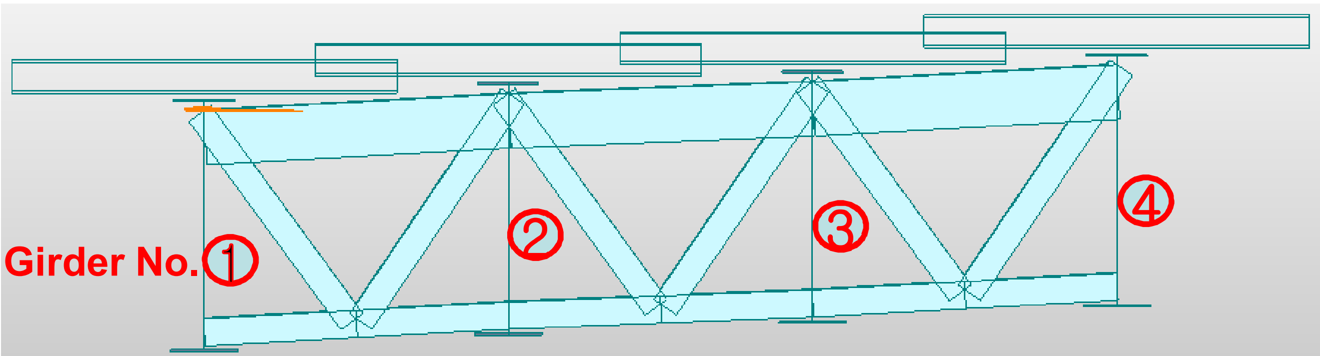


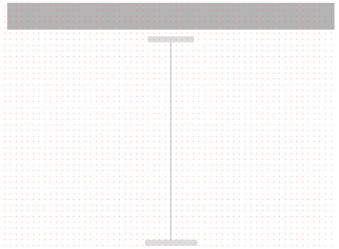
Image 1-2. 4 Steel Composite I Girders, Cross Frames & Bracings

Cross Section

Section 1-1 : Section in positive flexure

Section information (Section 1-1)

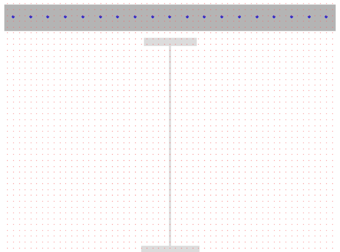
	d (in)	b (in)
Top flange	2	16
Web	69	0.5
Bottom flange	2	18
Concrete deck	9	114
Haunch	2.5	



Section 2-2 : Section in negative flexure

Section information (Section 2-2)

	d (in)	b (in)
Top flange	2.5	18
Web	69	0.5625
Bottom flange	2.5	20
Concrete deck	9	114
Haunch	2.5	



Note: Midas Civil provides an option to enter Girder number and CTC in the section data definition for composite section. This is only needed to consider the lateral stiffness of the bridge. **The number is kept as ‘1’ and CTS as ‘0’** if the cross beams have been modelled to consider the lateral stiffness, i.e. this option is not to be used for lateral stiffness consideration if the cross beams have been modelled.

Loads

DC1_1 : Self Weight acting on the non-composite section

DC1_2 : Wet concrete weight acting on the non-composite section

DC2 : Dead load of components and attachments acting on the long term composite section

DW : Wearing surface load acting on long term composite section

Moving Load :

Code: AASHTO LRFD
No. of lanes : 1, wheel spacing = 72”, eccentricity = 9”
Vehicle Load:2, HL-93TDM, HL-93TRK
Multiple presence factor: 1.2

2. Modeling Methodologies

MIDAS Civil provides three methods by which the initial modelling can be done. These methods are just to consider different types of analysis cases. They have no effect on the design methodology. Thus, irrespective of the method you choose, the design procedure followed by the software will be same.

- **A. Sequential Analysis + Accurate time dependent material**
- **B. Sequential Analysis + Long-term Modular Ratio of 3n**
- **C. Composite Action w/o Sequential Analysis**

A. Sequential Analysis + Accurate time dependent material

This modeling methodology is helpful when you want to have the Construction Stages along with accurate Time Dependent Material definition. The important steps for such modeling method are mentioned below.

❑Go to Properties > Time Dependent Material > Creep/Shrinkage

Define the time dependent material properties for considering creep and shrinkage through the construction stages.

❑Go to Load > Construction Stage > Define C.S. and Composite Section For C.S.

Define the Construction stages and composite section for construction stage.

❑Go to Analysis > Construction Stage Analysis Control

Check the box for inclusion of Time Dependent Effects in the CS Analysis.
Define Erection Loads. All the load cases which are to be distinguished from Dead Load for CS output can be specified here. Specifically for composite bridges, all the permanent loads after composite action which are to be distinguished from the permanent load before composite action are added here. Refer to the image 2-1.

Note: Two main load types to be used for composite bridges are:
DC: Component and Attachment Dead load acting on the long-term composite section.
DW: Wearing Surface Load acting on the long-term composite section.

This classification is necessary for Auto Generation of load combination. When you choose to Auto-generate the load combination, the software uses the load type from erection load definition to generate the load combinations.

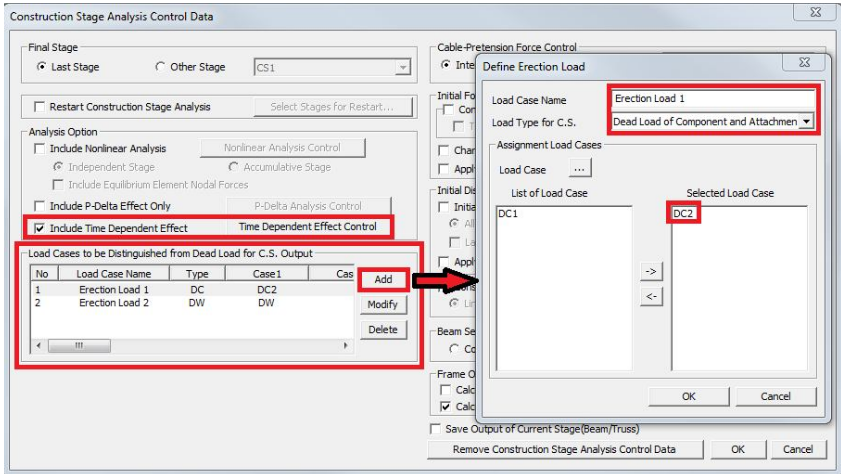


Image 2-1. Construction Stage Analysis Control

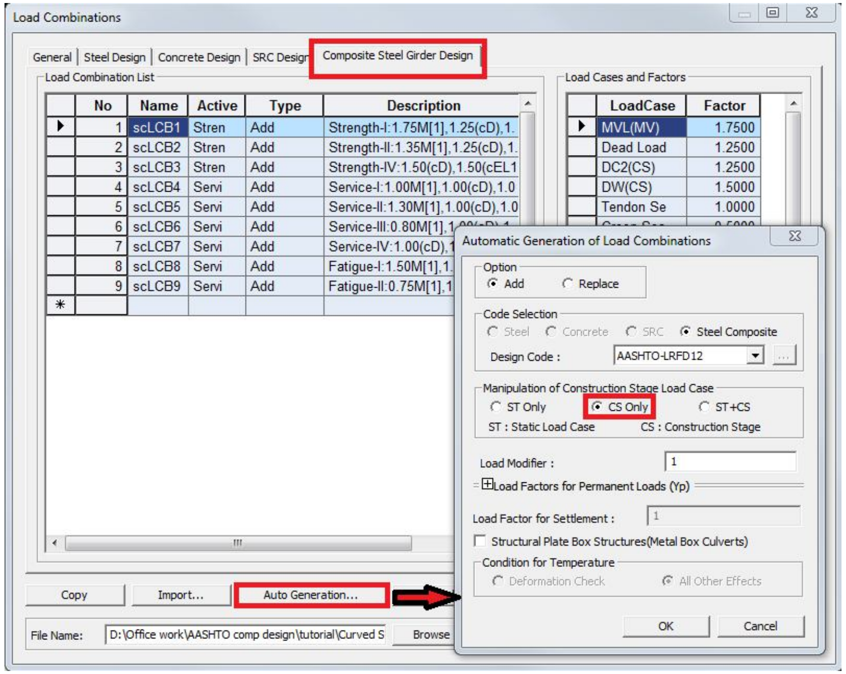


Image 2-2. Load Combinations

❑Go to Results > Load Combinations > Composite Steel Girder Design

You can manually add the load combinations or choose to Auto-Generate the load combinations. Auto Generation of load combinations for composite steel girder design in midas Civil is as per Table 3.4.1.1.

Note: Extreme Event Load Combinations are not considered in midas Civil for Composite Steel Girder Design.

Note: Midas Civil provides an option to manipulate with the load cases for auto generation of load combination. There you can choose to generate the combinations using “Static only”, “CS only” or “Static + CS” load cases. But for auto generation of load combinations in “Composite Steel Girder Design” tab, “CS Only” should be used with construction stage. Refer to Image 2-2.

Note: Software distinguishes the pre-composite, short-term and long-term loads from the construction stage definition.

B. Sequential Analysis + Long-term Modular Ratio of 3n

This modeling methodology is helpful when you want to have the Construction Stages analysis with time dependent effects considered only for composite section through the ratio of modulus of elasticity of steel and concrete. The important steps for such modeling method are mentioned below.

❑Go to Properties > Section Properties > Add > Composite Section

Define the composite section and check the box for Multiple Modulus of Elasticity.
Enter the long-term modular ratio (3n) i.e. Es/Ec for long term. Refer to the image 2-3.
After you click OK for the composite section definition, the software automatically generates Section Stiffness Scale Factors in discrete boundary groups. These factors take into account the varying section properties for the composite sections. Refer to the image 2-4.

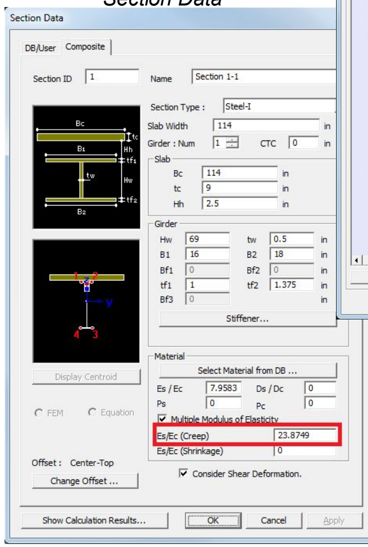
❑Go to Load > Construction Stage

Activate these boundary groups in their respective construction stages. **OR**
Go to Load > Construction Stage > Composite section for Construction stage
Click on Update Long Term. Doing so, all the effective width scale factor boundary groups automatically get activated in respective stages. Refer to the image 2-5.

Note: Construction Stage Analysis Control and Load Combination definition is same as the previous method.

Note: Software distinguishes the pre-composite, short-term and long-term loads from the construction stage definition. All the loads activated before the composite action in the CS are considered as Dead(Before) and use steel only section properties. Loads activated after the composite action in the CS are considered as Erection Load and use long-term composite section properties. All the loads acting in Post-CS are considered as temporary loads and use short-term composite section properties

Image 2-3. Composite Section Data



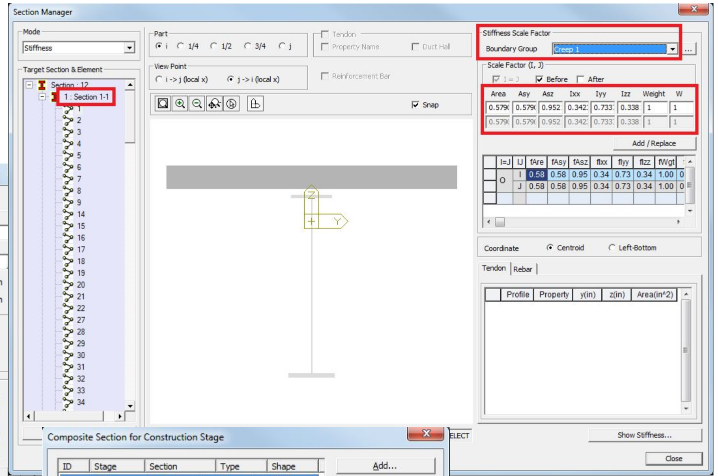


Image 2-4. Section Manager

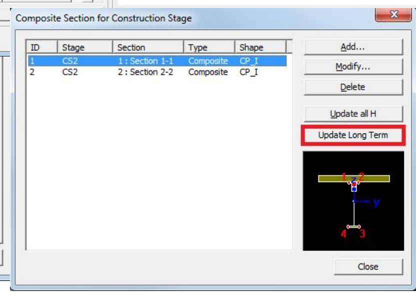


Image 2-5. Composite Section for CS

C. Composite Action w/o Sequential Analysis

This modeling methodology is helpful when you don't have Construction Stages defined. The long term effect consideration in the section data using modular ratio is same as that in methodology B. The important steps for such modeling method are mentioned below.

❑Go to Load > Settlement/Misc. > Pre-composite Section

Select the static load cases which are to be considered before composite action i.e. DC1. Refer to image 2-6.

❑Go to Analysis > Boundary Change Assignment

Assign the boundary groups representing the section stiffness scale factor, to the after composite static load cases i.e. DC2 and DW. Refer to image 2-7.

Note: All the static load cases selected in "Load Cases for Pre-Composite Section" are considered as Dead(Before) and use steel only section properties. Load cases assigned with section stiffness scale factor boundary groups through Boundary Change Assignment are considered as permanent loads and use long-term composite section properties. All the remaining static load cases are considered as temporary loads and use short-term composite section properties.

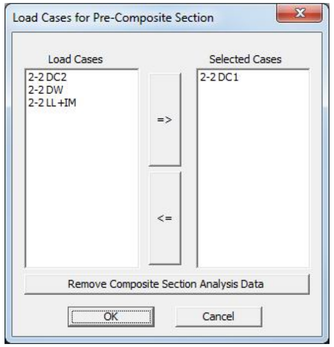


Image 2-6. Load Cases for Pre Composite Section

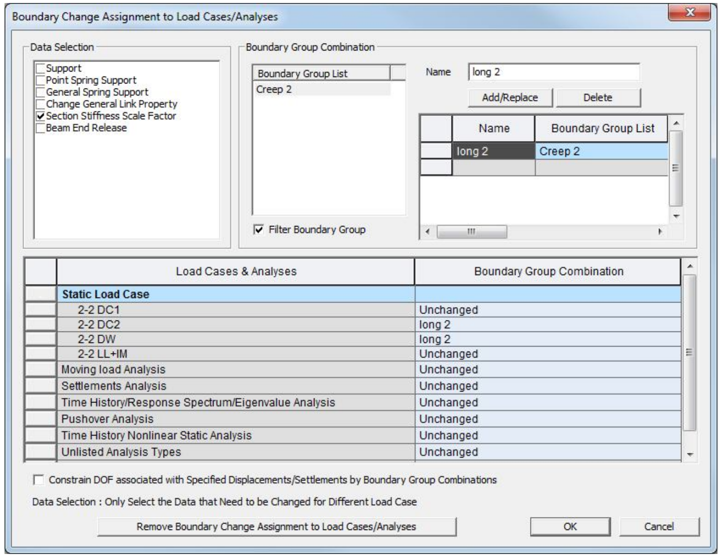


Image 2-7. Boundary Change Assignment

Note: Modeling method B is demonstrated through this tutorial and modeling method C is demonstrated thorough the box girder tutorial.

Composite Design

Composite bridge is one where a reinforced concrete deck slab sits on top of steel I-beams, and acts compositely with them in bending.

Preliminary sizing is part of the concept design, and is often based on crude estimations of load distribution, and resulting bending moments and shear forces. However, for steel composite highway bridges, preliminary design charts are available to facilitate far more accurate initial girder sizes.

Detailed design is effectively design verification to the AASHTO LRFD, which is more of a checking process than original creative design. Modelling and analysis is carried out for the selected structural arrangement for the various loading conditions (including fatigue) taking full account of any curvature and skew. The adequacy of the main members (composite beams, box girders etc.) is then checked in detail to ensure that they are adequate to carry the applied moments and forces. Details such as shear connector and stiffener sizes, are chosen at this stage to suit the global actions of the main members.

Design Steps:

A. Define Longitudinal Stiffeners
B. Define Effective Width Scale Factors
C. Input Span Information
D. Modify Construction Stage
E. Define Construction Stage Analysis Control
F. Generate Load Combinations
G. Input Design Information
H. View Design Results

A. Elastic Modulus Ratio

You can manually define the effective width scale factors to consider the long term effects like creep and shrinkage for the composite sections or use the **Section Data Dialog Box** to automatically define the Effective width Scale Factors to consider the same. In this tutorial we will use **Section Data Dialog Box**.

Material > Check ☑ Multiple Modulus of Elasticity
Material > Es/Ec (Creep) > 23.8749

Note: Long term modular ratio; 3n = 3*7.9583 = 23.8749

Click [OK] Refer to image 3-3.

Repeat steps A and B for Section 2-2.

Click [Close] Refer to image 3-4.

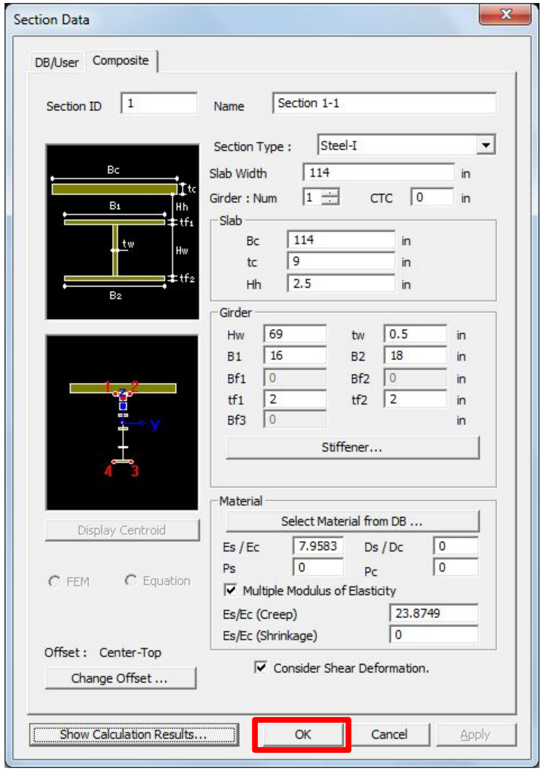


Image 3-1. Section Data Dialog Box

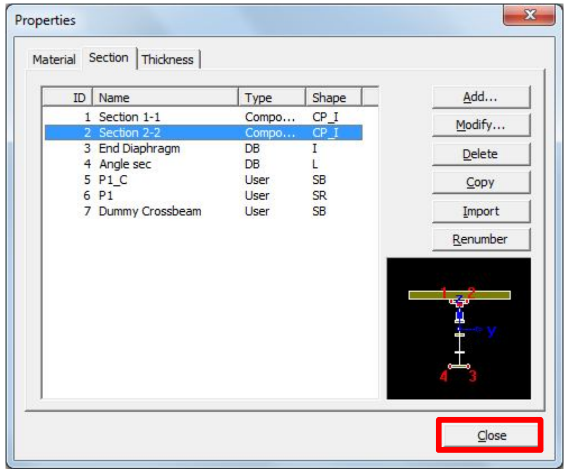


Image 3-2. Section Properties Dialog Box

B. Define Effective Width Scale Factors

To check the Auto Generated Section Stiffness Scale Factors,

❑Go to Properties > Section Manager > Stiffness

Target Section & Element > Double Click on 1 : Section 1-1
Target Section & Element > Click on element numbers to see the stiffness scale factors. Refer to image 3-5.

Repeat the same for Section 2-2 to see the stiffness scale factors for that section.

Note: These Stiffness Factors are automatically added into boundary groups which will be activated in post composite Construction Stage to take into account the long term effects.

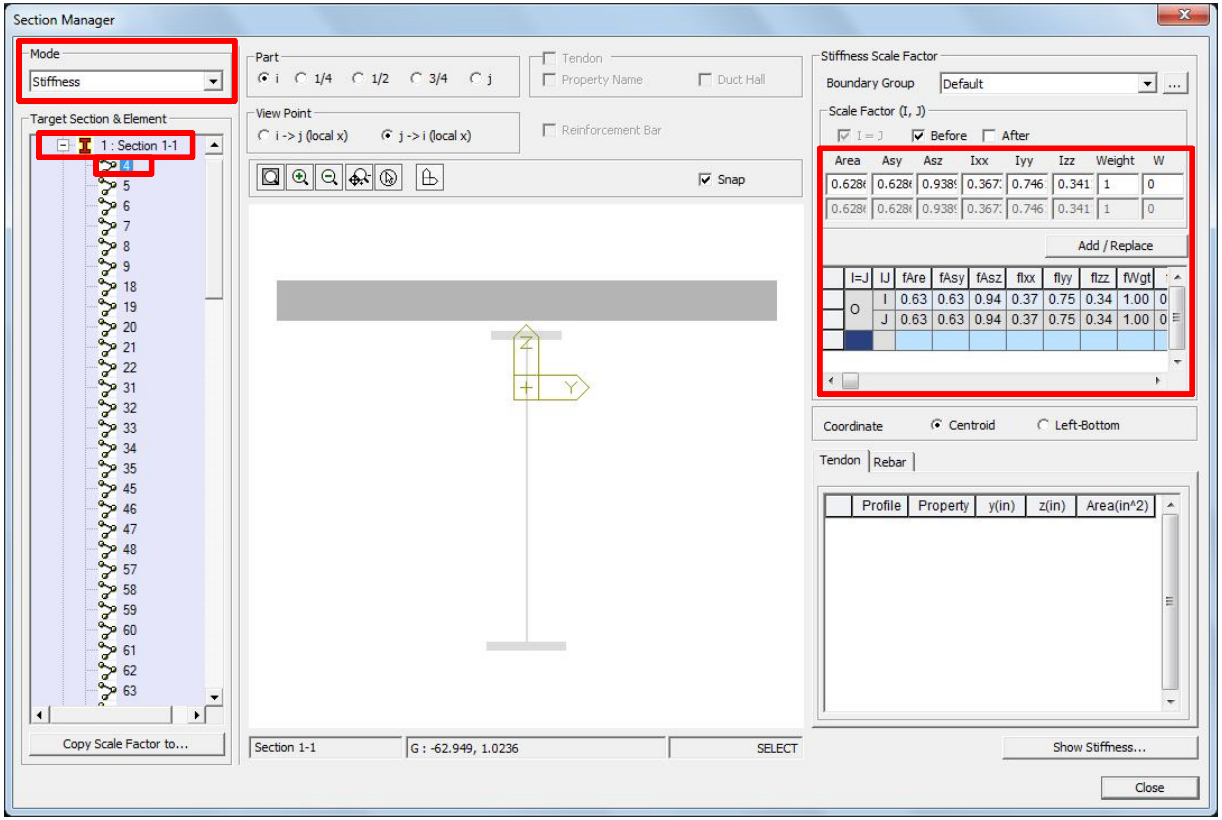


Image 3-3. Section Manager Dialog Box

C. Input Span Information

Span information is required for the program to distinguish the end and interior panes. Separate shear check formulae are needed for the panels depending upon their location. Span information is used for viewing the Composite Design Results and Design Result Diagram as per Span.

❑Go to Structure > Composite Bridge > Span Information

Girder Name > S1-L
Assign Elements > Check ◉ Number
Assign Elements > 98to111 436to449
Assign Elements > Click Add/Replace
Assign Elements > Support > Click on the box for support and change the support position from 'None' to 'I' and 'J' for Elements 98 (first element) and 449 (last element) respectively.
Refer to image 3-6.

Girder Information > Click Add

Repeat the above steps for other girders with the help of data below:

Girder Name > S1-R;	Assign Elements > 40to52 382to394
Girder Name > S2-L;	Assign Elements > 83to96 422to435
Girder Name > S2-R;	Assign Elements > 27to39 369to381
Girder Name > S3-L;	Assign Elements > 68to81 408to421
Girder Name > S3-R;	Assign Elements > 14to26 356to368
Girder Name > S4-L;	Assign Elements > 53to65 119 395to407 450
Girder Name > S4-R;	Assign Elements > 1to13 343to355

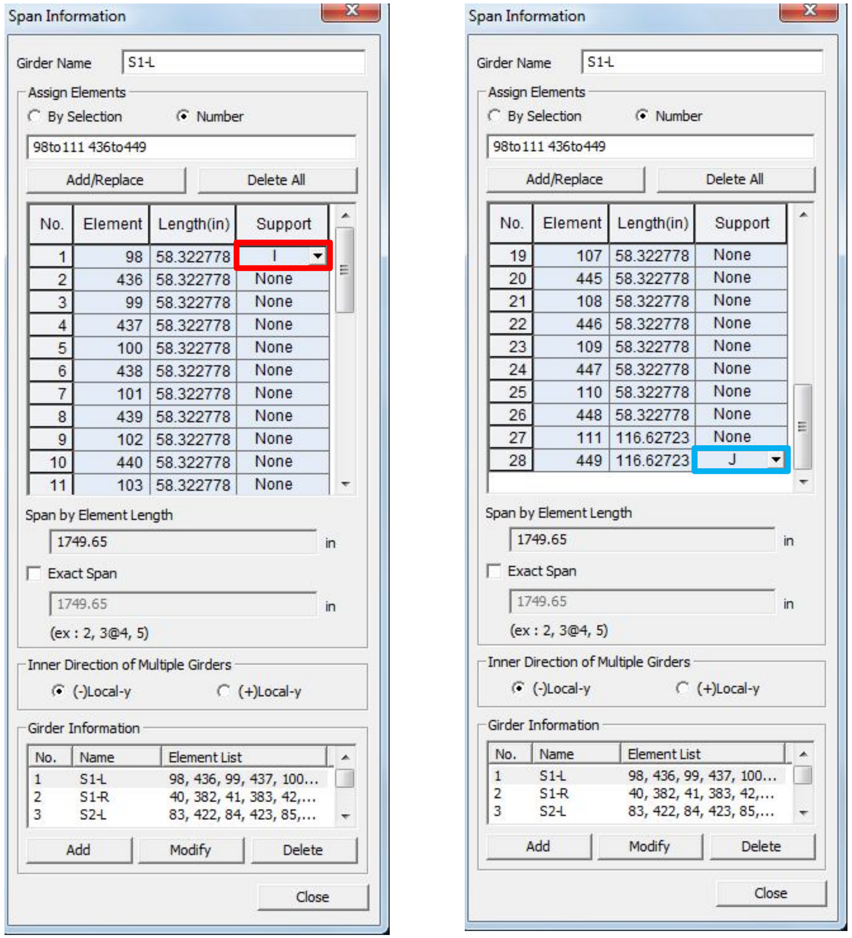


Image 3-4. Span Information Dialog Box

C. Model View of Span Information

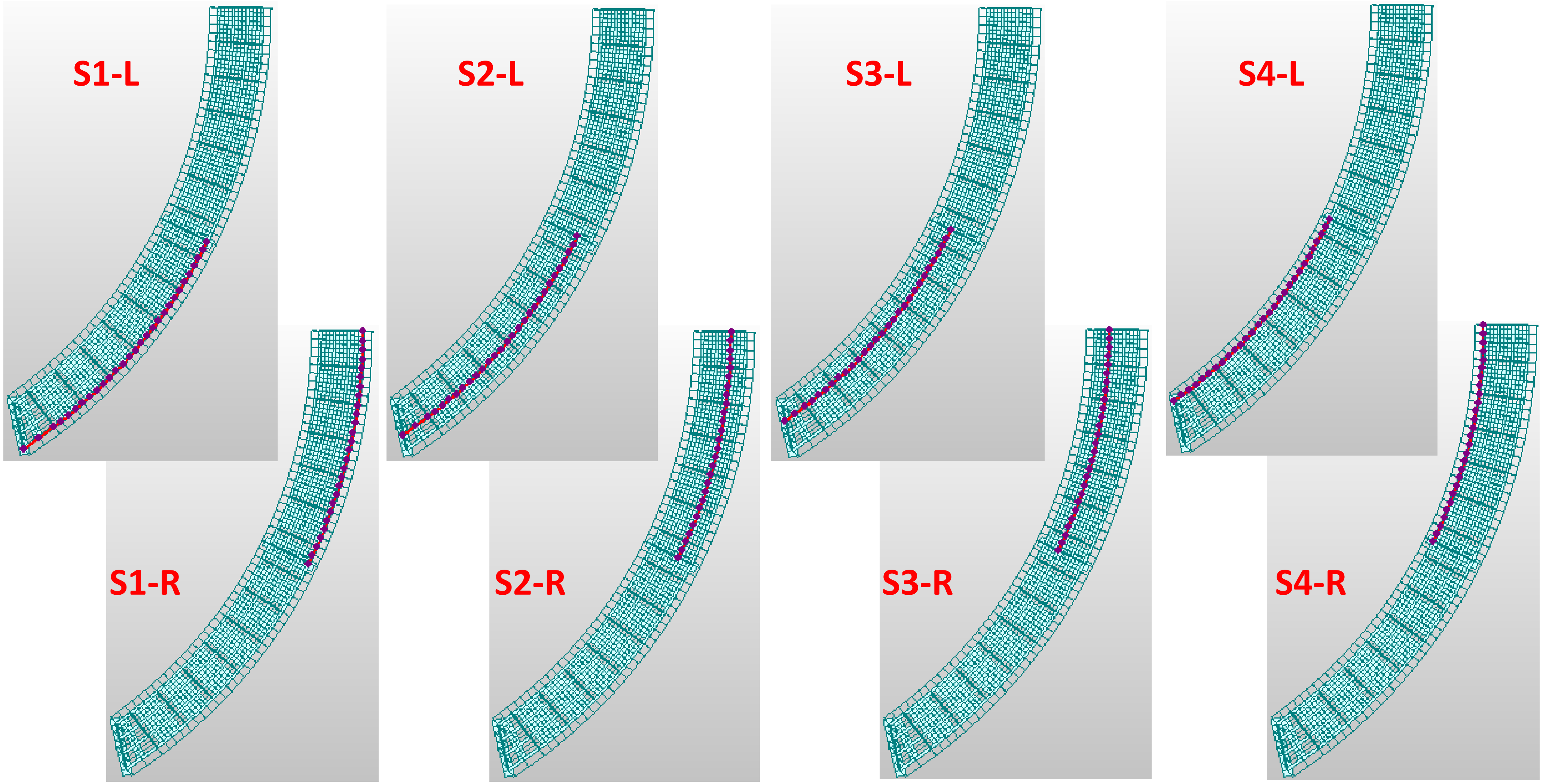


Image 3-5. Span Information

D. Modify Construction Stage

Section Stiffness Scale Factors need to be activated in the stage when the composite action begins. Composite action for both the composite sections starts in Construction Stage 3. Thus these stiffness scale factors should be activated in this stage.

❑Go to Load > Construction Stage > Define C.S.

Construction Stage Dialog Box > Select CS3
Click Modify/Show in Construction Stage Dialog Box
Compose Construction Stage > Click Boundary
Group List > Select Creep 1 and Creep 2
Activation > Click Add Refer to image 3-7
Click OK

Click Close

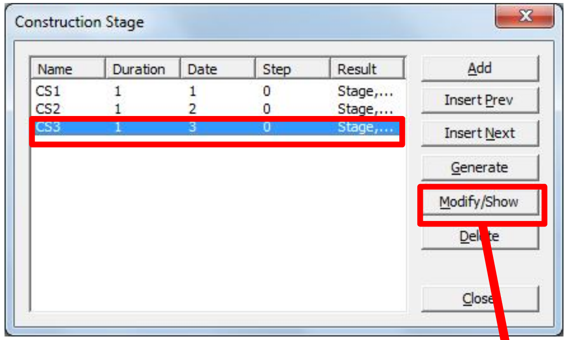


Image 3-6. Construction Stage Dialog Box

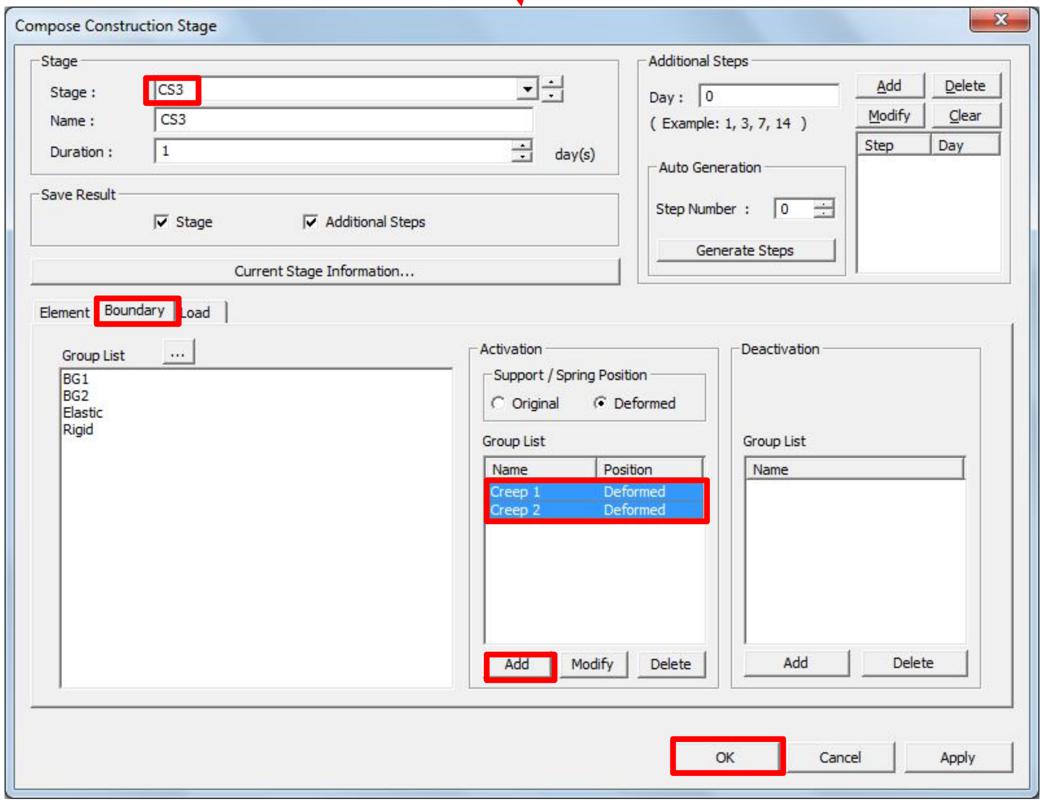


Image 3-7. Compose Construction Stage Dialog Box

E. Define Construction Stage Analysis Control

All the after composite loads have to be distinguished form the before composite dead loads, i.e. DC2 and DW have to be separated from DC1. This is done in midas Civil by defining DC2 and DW as erection loads in the construction stage analysis control.

❑Go to Analysis > Construction Stage

Load case to be distinguished from Dead Load for C.S. Output > Click Add
Define Erection Load > Load Case Name > DC2
Define Erection Load > Load Type for Post CS > Dead Load of Component & Attachments
Define Erection Load > Assignment Load Cases > Select DC2 > Click ->
Refer to image 3-8.
Click OK

Load case to be distinguished from Dead Load for C.S. Output > Click Add
Define Erection Load > Load Case Name > DW
Define Erection Load > Load Type for Post CS > Dead Load of Wearing Surface & Utilities
Define Erection Load > Assignment Load Cases > Select DW > Click ->
Refer to image 3-9.
Click OK

Click OK

❑Go to Analysis > Perform Analysis

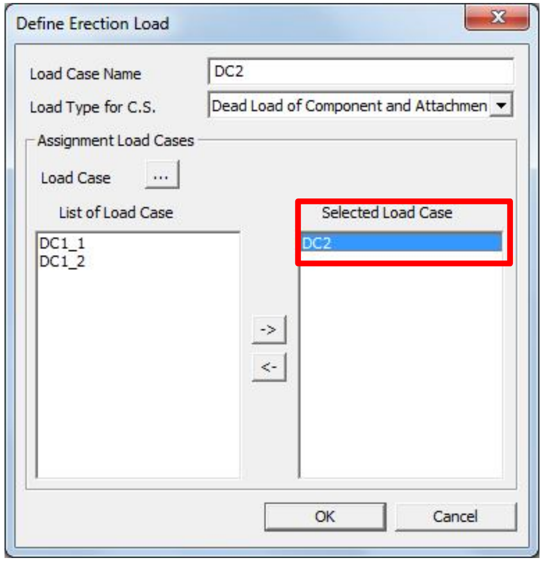


Image 3-8. Erection Load Definition for DC2

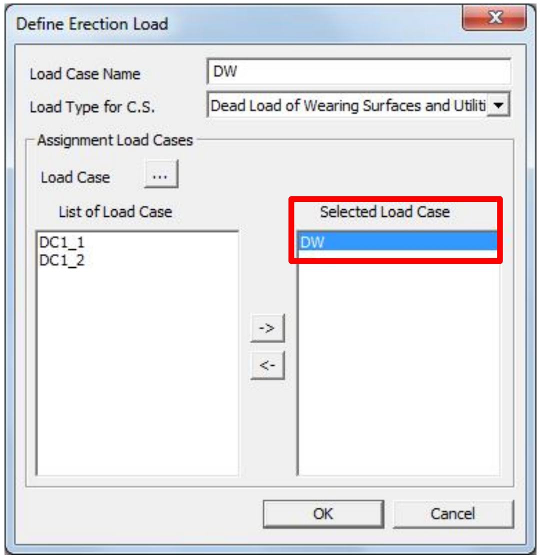


Image 3-9. Erection Load Definition for DW

F. Generate Load Combinations

In this tutorial we will Auto Generate Load Combinations for Composite Design as per AASHTO LRFD 2012.

❑Go to Results > Load Combinations

Click Composite Steel Girder Design

Click Auto Generation...

Automatic Generation of Load Combinations > Design Code > AASHTO-LRFD 12
Automatic Generation of Load Combinations > Manipulations of CS Load Cases > Select CS Only Refer to image 3-10

Click OK
You can view the Auto generated load combinations as in image 3-11
Click Close

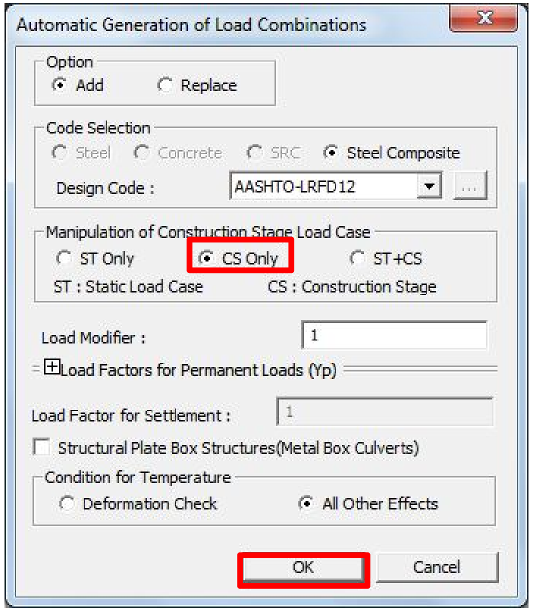
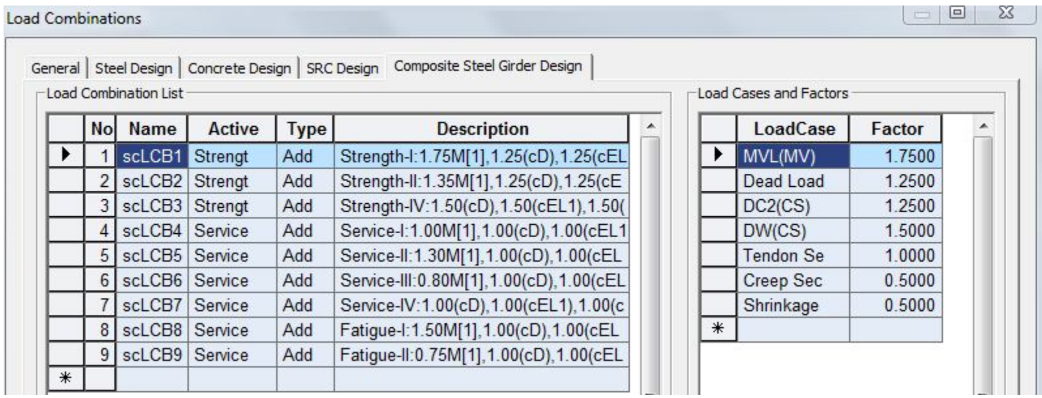


Image 3-10. Auto Generation Load Combinations Dialog Box



No	Name	Active	Type	Description
1	scLCB1	Strengt	Add	Strength-I:1.75M[1],1.25(cD),1.25(cEL
2	scLCB2	Strengt	Add	Strength-II:1.35M[1],1.25(cD),1.25(cE
3	scLCB3	Strengt	Add	Strength-IV:1.50(cD),1.50(cEL1),1.50(
4	scLCB4	Service	Add	Service-I:1.00M[1],1.00(cD),1.00(cEL1
5	scLCB5	Service	Add	Service-II:1.30M[1],1.00(cD),1.00(cEL
6	scLCB6	Service	Add	Service-III:0.80M[1],1.00(cD),1.00(cEL
7	scLCB7	Service	Add	Service-IV:1.00(cD),1.00(cEL1),1.00(c
8	scLCB8	Service	Add	Fatigue-I:1.50M[1],1.00(cD),1.00(cEL
9	scLCB9	Service	Add	Fatigue-II:0.75M[1],1.00(cD),1.00(cEL

LoadCase	Factor
MVL(MV)	1.7500
Dead Load	1.2500
DC2(CS)	1.2500
DW(CS)	1.5000
Tendon Se	1.0000
Creep Sec	0.5000
Shrinkage	0.5000

Image 3-11. Auto Generated Load Combinations

G. Input Design Information

❑Go to Design > Composite Design > Design Parameters

Composite Steel Girder Design Parameters > Code > AASHTO-LRFD12
Composite Steel Girder Design Parameters > Click [Update by Code]

Option For Strength Limit State > Check
- ☑ Appendix A6 for Negative Flexure Resistance in Web Compact / NonCompact Sections
- ☑ Mn<1.3RhMy in Positive Flexure and Compact Sections(6.10.7.1.2-3)
- ☑ Post-buckling Tension-field Action for Shear Resistance(6.10.9.3.2)

Click [OK] Refer to image 3-12.

❑Go to Design > Composite Design > Design Material

Select the SRC material in the 'Material List'.
Steel Material Selection > Code > ASTM09(S)
Steel Material Selection > Check ☑ Hybrid Factor
Steel Material Selection > Click [...]

Flange(Top) > Grade > A709-HPS70W
Flange(Bot) > Grade > A709-HPS70W
Web > Grade > A709-50W
Refer to image 3-13

G. Input Design Information

Click [OK] in Hybrid Factor Window

Concrete Material Selection > Code > ASTM(RC)
Concrete Material Selection > Grade C4500

Reinforcement Selection > Code > ASTM(RC)
Concrete Material Selection > Grade of main Rebar > Grade 60
Concrete Material Selection > Grade of sub Rebar > Grade 50

Click [Modify] in Modify Composite Material Window

Click [Close] Refer to image 3-14

Note: Hybrid Factor is used when the material of top flange, bottom flange and web are different. If the material for all the three components are same then single material can be defined without the use of hybrid factor.

❑Go to Design > Composite Design > Load Combination Type
Software automatically classifies the auto generated load combinations into Strength, Service and Fatigue categories. Here, you can choose the load combinations to be considered for Composite Design.

Click [OK] Refer to image 3-15

G. Input Design Information

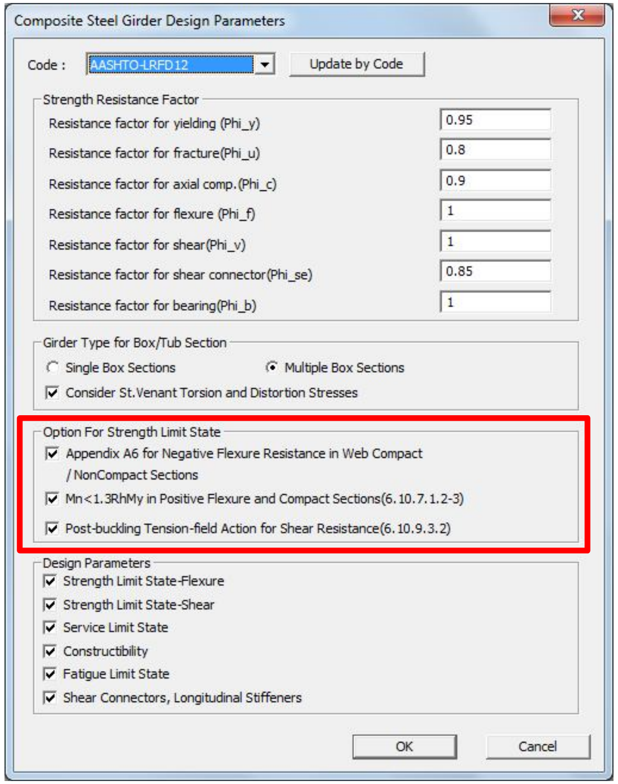


Image 3-12. Design Parameter Dialog Box

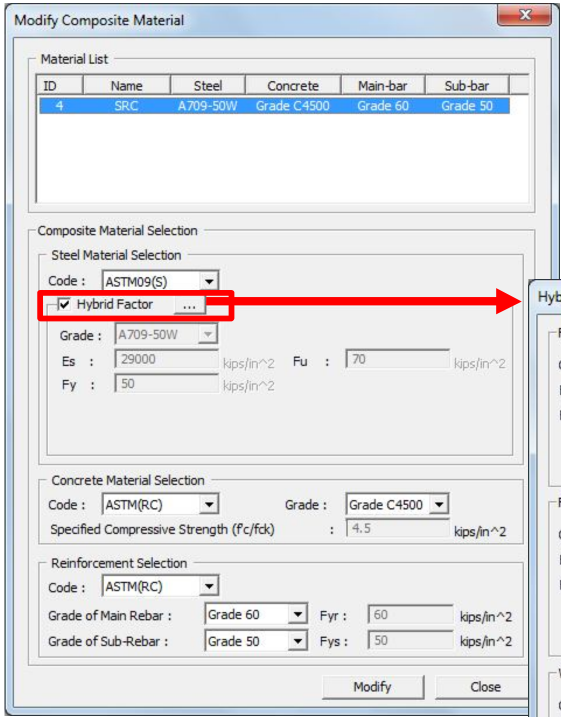


Image 3-14. Composite Material Dialog Box

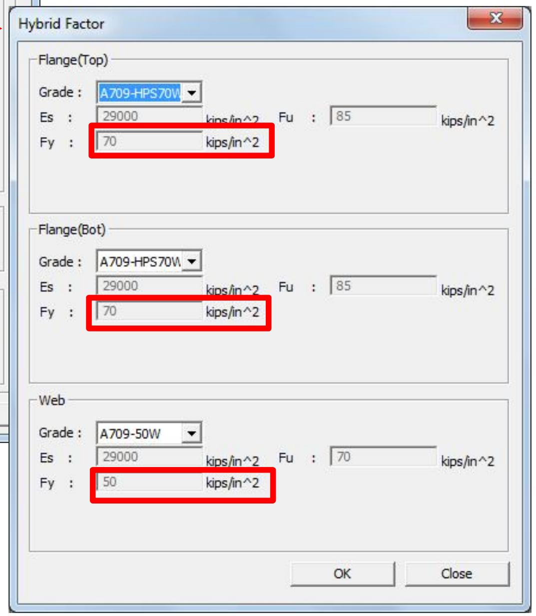


Image 3-13. Hybrid Factor Dialog Box

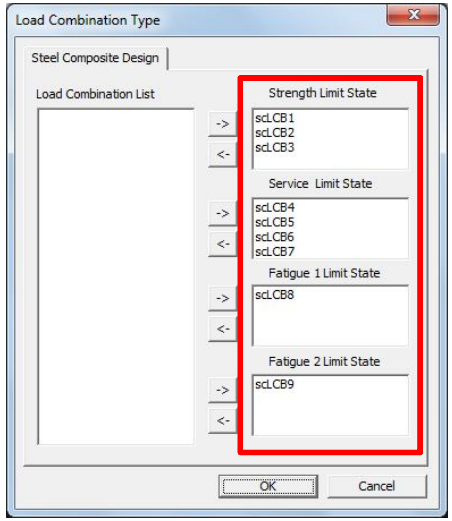


Image 3-15. Load Combination Type Dialog Box

G. Input Design Information

❑Go to Design > Composite Design > Longitudinal Reinforcement

In this tutorial, the longitudinal reinforcement will be provided in the negative flexure sections only i.e. Section 2-2.

Target Section & Element > Select '2 : Section 2-2'; Refer to image 3-16
Longitudinal Reinforcement > Select Input Method A
Ref. Y > Left
Y > 3in
Ref. Z > Top
Z > 4.37in
Num > 19; 'Num' stands for number of reinforcement bar
Spacing > 6in
Dia > #8
Part > Part 2; Part 2 is the concrete deck and Part 1 is the steel girder
Click Add
Click Apply
Click Close

Go to Design > Composite Design > Transverse Stiffener

Transverse stiffeners are required for considering the tension field action in interior stiffened panels for Strength Limit State check.

G. Input Design Information

Target Section & Element > Select '1 : Section 1-1'; Refer to image 3-17

Transverse Stiffener > Check Web
Transverse Stiffener > Click ...
Stiffener Type > Flat
Transverse Stiffener > Select One stiffener
Transverse Stiffener > Fy > 36ksi
Transverse Stiffener > Pitch > 90in
Transverse Stiffener > H > 5in
Transverse Stiffener > B > 1.5in;

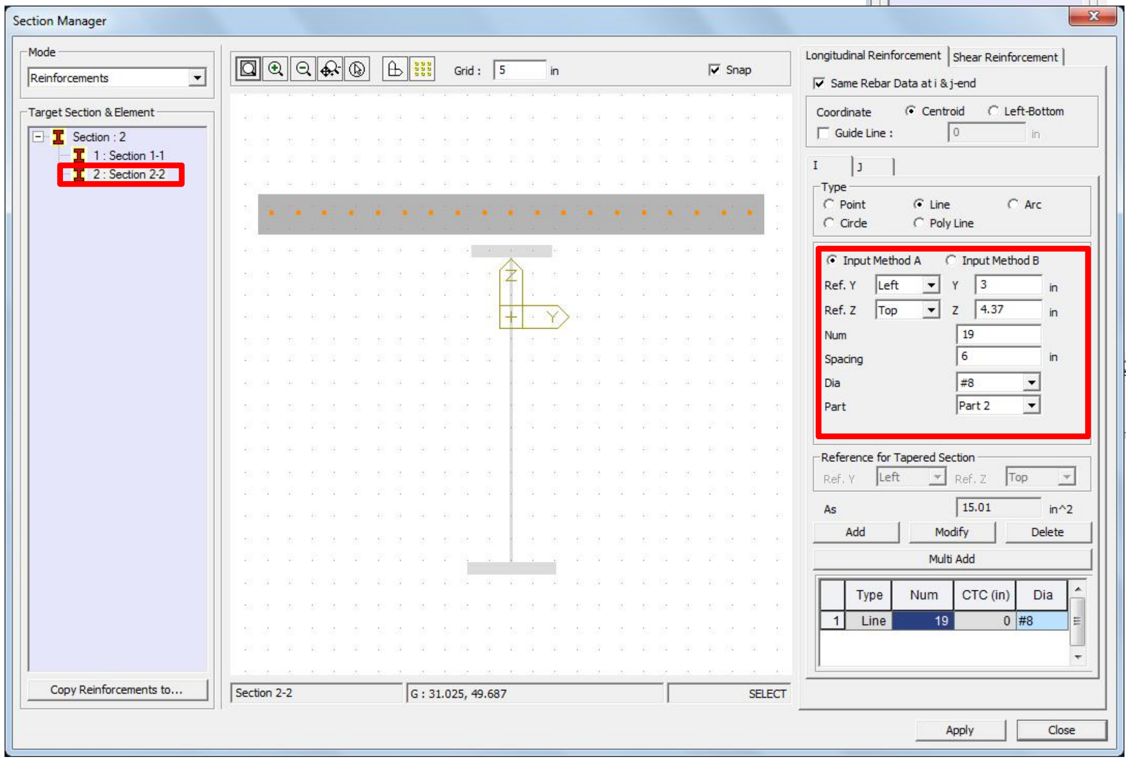
Click OK
Click Apply

Target Section & Element > Select '2 : Section 2-2'
Transverse Stiffener > Check Web
Transverse Stiffener > Click ...
Stiffener Type > Flat
Transverse Stiffener > Select
Transverse Stiffener > Fy > 36ksi
Transverse Stiffener > Pitch > 90in
Transverse Stiffener > H > 5in
Transverse Stiffener > B > 1.5in

Click OK
Click Apply
Click Close

G. Input Design Information

Image 3-16. Longitudinal Reinforcement Dialog Box



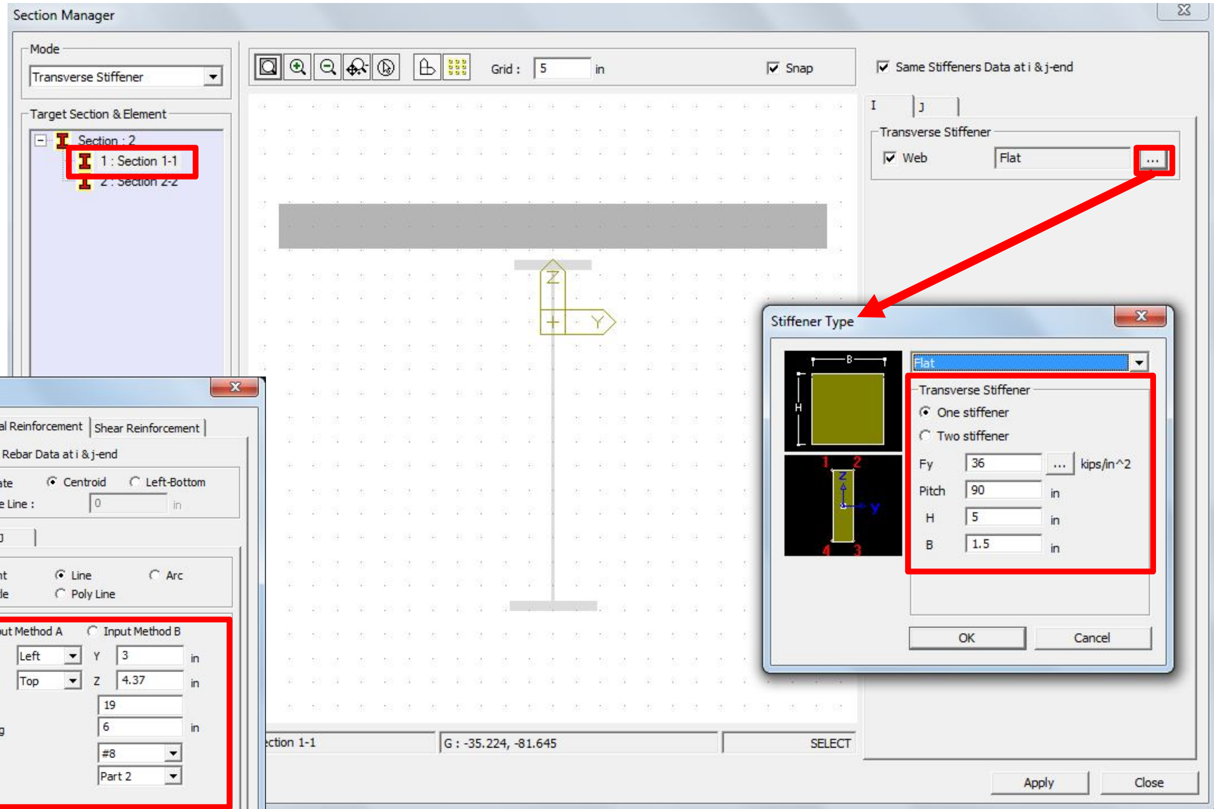


Image 3-17. Transverse Stiffener Dialog Box

G. Input Design Information

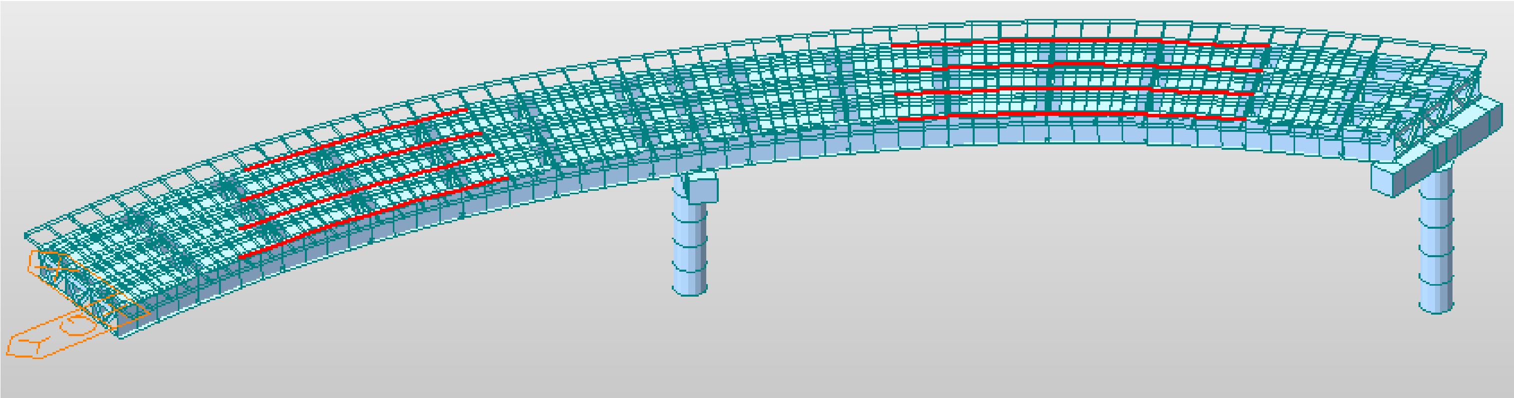


Fig : Section 1-1 Model View

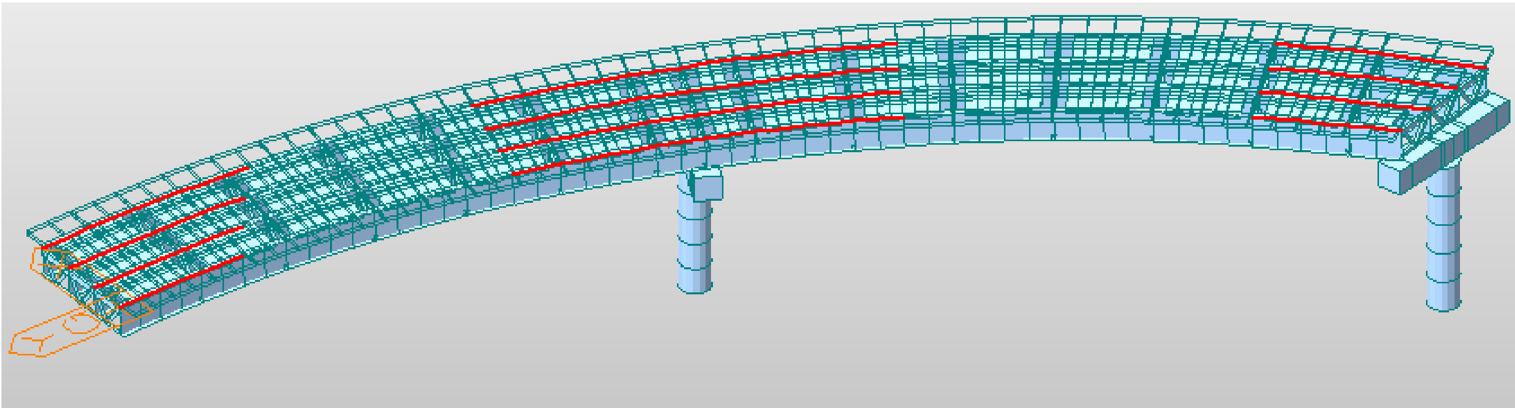


Fig : Section 2-2 Model View

G. Input Design Information

❑ Go to Design > Composite Design > Unbraced Length

Lb, Unbraced length is used for Lateral Torsional Buckling check in Composite Design.

Select all the composite girders.
Laterally Unbraced Length > 223in
Click Apply

❑ Go to Design > Composite Design > Design Position

Design positions are the locations at which the Composite Design will be performed.

Select all the composite girders.
Check Position > I & J
Click Apply

❑ Go to Design > Composite Design > Position for Design Output

Position for Design Output are the locations for which the detailed Design Report will be generated in Excel format.

Select elements 75.
Position > J
Click Apply

G. Input Design Information

Select elements 368.
Position > I
Click Apply

❑ Go to Design > Composite Design > Shear Connector

In this tutorial, the shear connectors will be provided in the negative flexure sections only i.e. Section 2-2.

Select all the composite girders with Section 2-2.
Check ☑ Both end parts(i & j) have the same type

Shear Connector > Category > C
Shear Connector > Pitch > 5in
Shear Connector > Height > 7in
Shear Connector > Dia > 0.875in
Shear Connector > Fu > 60ksi
Shear Connector > Spacing Shear Connector > 4in; This spacing is the transverse spacing between two adjacent shear connectors.
Shear Connector > Num. of Shear Connectors > 3; This is the number of shear connectors placed transversely in each row

Click Apply

G. Input Design Information

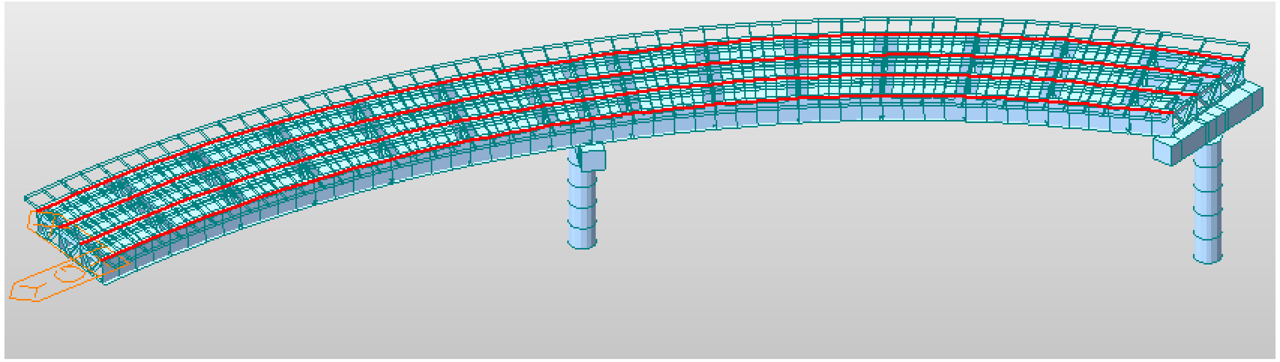


Fig : Design Positions Model View

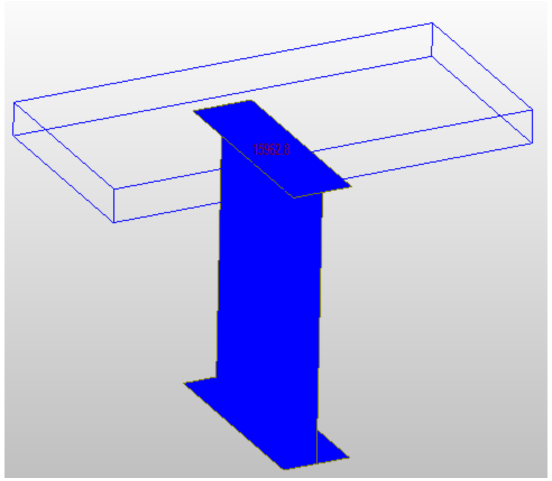


Fig : Element 75; Section 1-1; Positive Flexure

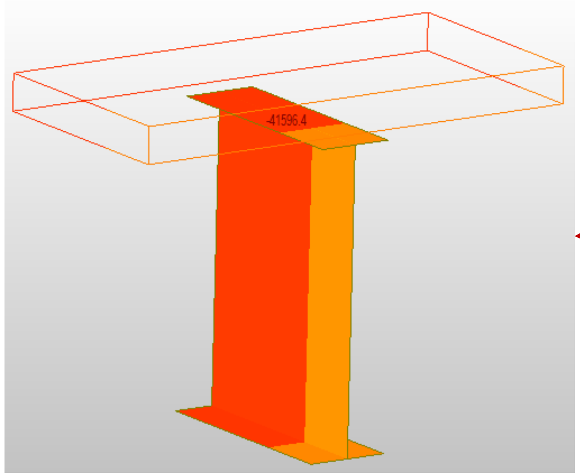


Fig : Element 368; Section 2-2; Negative Flexure

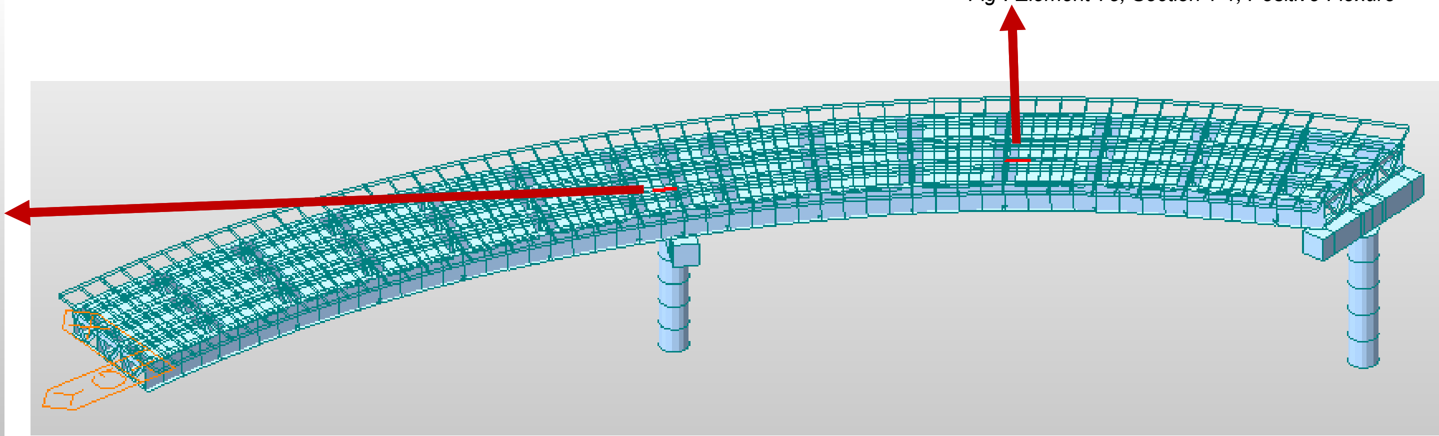


Fig : Positions for Design Output Model View

G. Input Design Information

❑ Go to Design > Composite Design > Fatigue Parameters

Select all the composite girders.
Check ☑ Both end parts(i & j) have the same type

Shear Connector > Category > C'
Shear Connector > (ADTT)SL > 1000
Shear Connector > N(n/cycle) > 1

Click Apply

❑ Go to Design > Composite Design > Curved Bridge Info.

This curved bridge information allows the software to consider the bridge as a curved bridge for Composite Design. Radius inputted here doesn't affect the design forces (lateral moment) due to Curvature. Design forces are solely calculated from the analysis results.

Select all the composite girders.
Check ☑ Both end parts(i & j) have the same type

Curved Bridge Info. > Girder Radius > 2040in

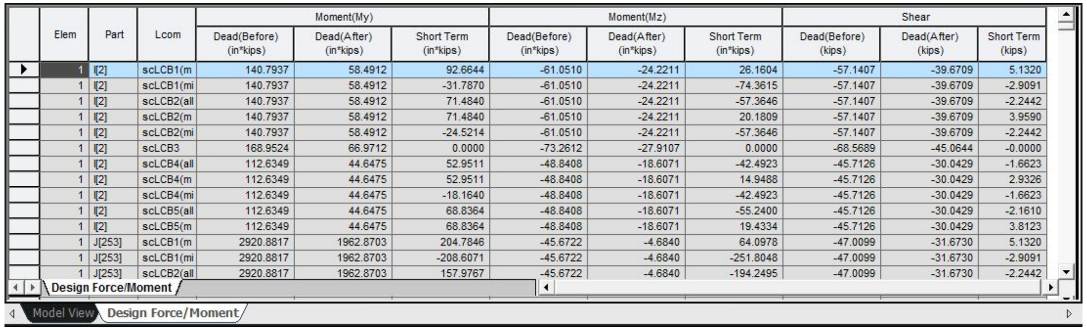
Click Apply

❑ Go to Design > Composite Design > Design Tables > Design Force/Moment

You can check the design forces used for Composite Design in this table. Refer to image 3-18.

❑ Go to Design > Composite Design > Design

Perform Composite Design.
"Composite steel girder design has been successfully completed"; this message in the message window indicates the completion of Composite Design.



Elem	Part	Lcom	Moment(My)			Moment(Mz)			Shear		
			Dead(Before) (in*kips)	Dead(After) (in*kips)	Short Term (in*kips)	Dead(Before) (in*kips)	Dead(After) (in*kips)	Short Term (in*kips)	Dead(Before) (kips)	Dead(After) (kips)	Short Term (kips)
1	I[2]	scLCB1(m	140.7937	58.4912	92.6644	-61.0510	-24.2211	26.1604	-57.1407	-39.6709	5.1320
1	I[2]	scLCB1(mi	140.7937	58.4912	-31.7870	-61.0510	-24.2211	-74.3615	-57.1407	-39.6709	-2.9091
1	I[2]	scLCB2(all	140.7937	58.4912	71.4840	-61.0510	-24.2211	-57.3646	-57.1407	-39.6709	-2.2442
1	I[2]	scLCB2(m	140.7937	58.4912	71.4840	-61.0510	-24.2211	20.1809	-57.1407	-39.6709	3.9590
1	I[2]	scLCB2(mi	140.7937	58.4912	-24.5214	-61.0510	-24.2211	-57.3646	-57.1407	-39.6709	-2.2442
1	I[2]	scLCB3	168.9524	66.9712	0.0000	-73.2612	-27.9107	0.0000	-68.5689	-45.0644	-0.0000
1	I[2]	scLCB4(all	112.6349	44.6475	52.9511	-48.8408	-18.6071	-42.4923	-45.7126	-30.0429	-1.6623
1	I[2]	scLCB4(m	112.6349	44.6475	52.9511	-48.8408	-18.6071	14.9488	-45.7126	-30.0429	2.9326
1	I[2]	scLCB4(mi	112.6349	44.6475	-18.1640	-48.8408	-18.6071	-42.4923	-45.7126	-30.0429	-1.6623
1	I[2]	scLCB5(all	112.6349	44.6475	68.8364	-48.8408	-18.6071	-55.2400	-45.7126	-30.0429	-2.1610
1	I[2]	scLCB5(m	112.6349	44.6475	68.8364	-48.8408	-18.6071	19.4334	-45.7126	-30.0429	3.8123
1	J[253]	scLCB1(m	2920.8817	1962.8703	204.7846	-45.6722	-4.6840	64.0978	-47.0099	-31.6730	5.1320
1	J[253]	scLCB1(mi	2920.8817	1962.8703	-208.6071	-45.6722	-4.6840	-251.8048	-47.0099	-31.6730	-2.9091
1	J[253]	scLCB2(all	2920.8817	1962.8703	157.9767	-45.6722	-4.6840	-194.2495	-47.0099	-31.6730	-2.2442

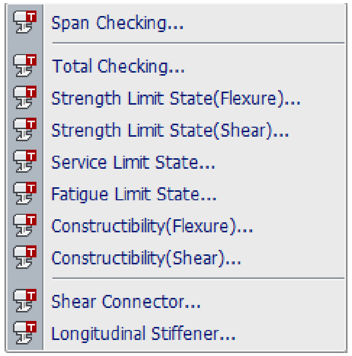
Design Force/Moment
Model View | Design Force/Moment

Image 3-18. Design Force/Moment Table

H. View Design Results

❑ Go to Design > Composite Design > Design Results Table

Design Results Table has the following results in tabular format:



❑ Go to Design > Composite Design > Print Result

Print Result option generates a detailed design report for the design positions which were selected in Positions for Design Output. Detailed design report encompasses all the relevant clauses from AASHTO LRFD 2012 and all the formulae used for the Composite Design.

Note: In this tutorial, the results in the Design Results Table and the Design Report will be discussed simultaneously.
Note: Any check which fails to satisfy the requisite condition for Composite Design is in red and the CHECK is reported to be NG(Not Good).

H. View Design Results

Span Checking…

This table shows the most critical members in positive and negative flexure for each span. The advantage is that, just by looking at this table you can notice all the spans which are failing in any check.

Records Activation Dialog > Choose the Spans as per Span Information and the condition of Positive/Negative; Refer to image 3-19

The Span Checking Results Table is as shown in image 3-20.

Total Checking…

This table summarizes all the check results for each and every element in a single table.

Records Activation Dialog > Choose the Elements, part of the elements and the condition of Positive/Negative for which the Total Checking Results are to be viewed.;
Refer to image 3-21

The Span Checking Results Table is as shown in image 3-22.

Note: Span Checking and the Total Checking results are not available in the Design Report.

H. View Design Results

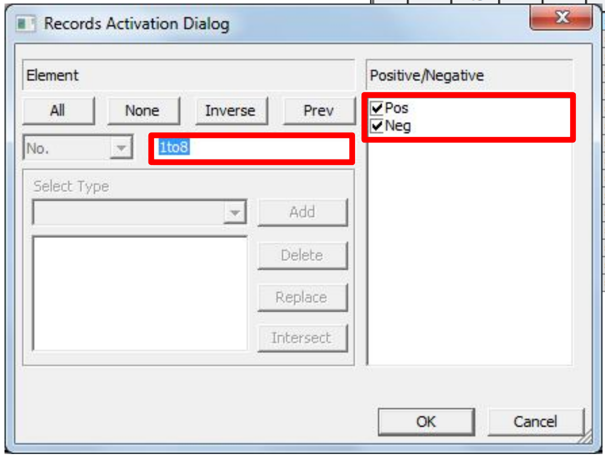


Image 3-19. Records Activation Dialog

Strength Limit(Flexure)			Strength Limit(Shear)									Service Limit							Fatigue Limit							Constructibility(Flexure)						
Lcom	Mu/Mr	CHK	Elem	part	Lcom	Vu/Vr	do	bt	lt	As	CHK	Elem	part	Lcom	tcw Ratio	tcf Ratio	tft Ratio	CHK	Elem	part	Lcom	Gamma(Delta_f) Ratio	fcf Ratio	Vcf Ratio	CHK	Elem	part	CS	tcw Ratio	tcf Ratio	tft Ratio	CHK
scLCB1	1.047	NG	102	I[112]	scLCB1	0.54	OK	OK	OK	OK	OK	98	I[11]	scLCB5	0.7477	0.5624	0.4424	OK	98	I[11]	scLCB8	0.7221	0.0000	0.6035	OK	98	I[11]	CS2	0.431	0.929	1.064	NG
scLCB1	0.768	OK	102	I[112]	scLCB1	0.54	OK	OK	OK	OK	OK	445	J[118]	scLCB5	-	0.3338	0.7392	OK	98	I[11]	scLCB8	0.7221	0.0000	0.6035	OK	445	J[118]	CS2	0.532	0.908	0.722	OK
scLCB1	0.806	OK	393	J[66]	scLCB1	0.44	OK	OK	OK	OK	OK	394	J[11]	scLCB5	0.5816	0.4488	0.3626	OK	40	I[7]	scLCB8	0.0017	0.0000	0.4317	OK	394	J[11]	CS2	0.360	0.673	0.775	OK
scLCB1	0.543	OK	393	J[66]	scLCB1	0.44	OK	OK	OK	OK	OK	386	J[59]	scLCB5	-	0.2095	0.5104	OK	40	I[7]	scLCB8	0.0017	0.0000	0.4317	OK	386	J[59]	CS2	0.328	0.520	0.441	OK
scLCB1	0.961	OK	83	I[10]	scLCB1	0.63	OK	OK	OK	OK	OK	83	I[10]	scLCB5	0.6637	0.5054	0.4028	OK	83	I[10]	scLCB8	0.5341	0.0000	0.8110	OK	83	I[10]	CS2	0.397	0.835	0.964	OK
scLCB1	0.590	OK	83	I[10]	scLCB1	0.63	OK	OK	OK	OK	OK	430	J[103]	scLCB5	-	0.2607	0.5445	OK	83	I[10]	scLCB8	0.5341	0.0000	0.8110	OK	430	J[103]	CS2	0.415	0.615	0.514	OK
scLCB1	0.747	OK	381	J[10]	scLCB1	0.56	OK	OK	OK	OK	OK	381	J[10]	scLCB5	0.5436	0.4204	0.3406	OK	27	I[6]	scLCB8	0.1233	0.0000	0.4221	OK	381	J[10]	CS2	0.337	0.622	0.715	OK
scLCB1	0.400	OK	381	J[10]	scLCB1	0.56	OK	OK	OK	OK	OK	31	I[46]	scLCB5	-	0.1675	0.3719	OK	27	I[6]	scLCB8	0.1233	0.0000	0.4221	OK	373	I[283]	CS2	0.263	0.392	0.334	OK
scLCB1	0.784	OK	68	I[9]	scLCB1	0.53	OK	OK	OK	OK	OK	68	I[9]	scLCB5	0.4935	0.3861	0.3166	OK	68	I[9]	scLCB8	0.3320	0.0000	0.6790	OK	68	J[318]	CS2	0.259	0.573	0.667	OK
scLCB1	0.393	OK	68	I[9]	scLCB1	0.53	OK	OK	OK	OK	OK	417	I[327]	scLCB5	-	0.1719	0.3691	OK	68	I[9]	scLCB8	0.3320	0.0000	0.6790	OK	417	I[327]	CS2	0.277	0.444	0.376	OK
scLCB1	0.533	OK	368	J[9]	scLCB1	0.48	OK	OK	OK	OK	OK	368	J[9]	scLCB5	0.4435	0.3496	0.2888	OK	14	I[5]	scLCB8	0.0006	0.0000	0.2716	OK	368	I[278]	CS2	0.237	0.432	0.496	OK
scLCB1	0.290	OK	368	J[9]	scLCB1	0.48	OK	OK	OK	OK	OK	360	I[270]	scLCB5	-	0.1217	0.2730	OK	14	I[5]	scLCB8	0.0006	0.0000	0.2716	OK	360	I[270]	CS2	0.196	0.311	0.264	OK
scLCB2	0.451	OK	53	I[8]	scLCB3	0.31	OK	OK	OK	OK	OK	53	I[8]	scLCB5	0.3023	0.2348	0.1911	OK	53	I[8]	scLCB8	0.2762	0.0000	0.3782	OK	53	J[305]	CS2	0.159	0.389	0.455	OK
scLCB3	0.176	OK	53	I[8]	scLCB3	0.31	OK	OK	OK	OK	OK	403	J[75]	scLCB5	-	0.0685	0.1703	OK	53	I[8]	scLCB8	0.2762	0.0000	0.3782	OK	404	I[314]	CS2	0.115	0.217	0.182	OK
scLCB1	0.454	OK	355	J[8]	scLCB3	0.36	OK	OK	OK	OK	OK	355	J[8]	scLCB5	0.3459	0.2751	0.2294	OK	1	I[2]	scLCB8	0.0020	0.0000	0.1451	OK	355	I[265]	CS2	0.192	0.373	0.431	OK
scLCB3	0.167	OK	355	J[8]	scLCB3	0.36	OK	OK	OK	OK	OK	346	J[22]	scLCB5	-	0.0733	0.1558	OK	1	I[2]	scLCB8	0.0020	0.0000	0.1451	OK	347	I[257]	CS2	0.129	0.198	0.168	OK

Image 3-20. Span Checking Results Table

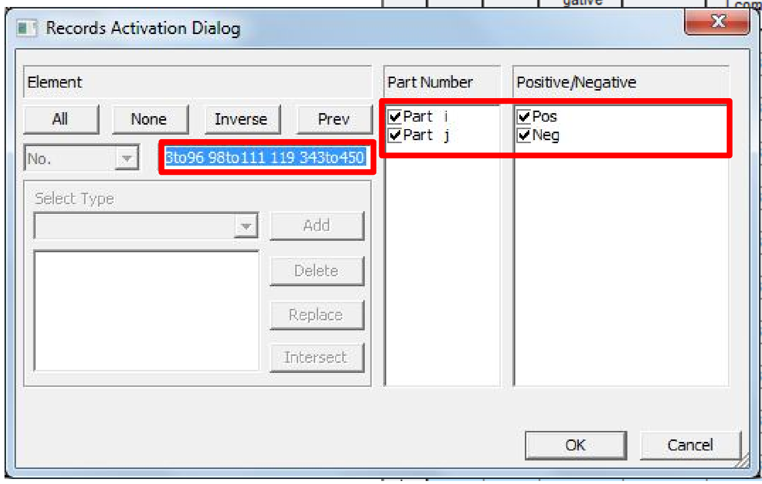


Image 3-21. Records Activation Dialog

Strength	Strength Limit(Shear)				Service Limit				Fatigue Limit			Constructibility(Flexure)					Constructibility(She	
Mu/phiMn	Lcom	Vu/phiVn	bt	lt	Lcom	tcw Ratio	tcf Ratio	tft Ratio	Lcom	Gamma(Delta_f) Ratio	Vcr Ratio	CS	tcw Ratio	tcf Ratio	tft Ratio	tdeck Ratio	CS	Vu/phiVn
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
0.0060	scLCB3	0.1337	OK	OK	scLCB5	-	0.0007	0.0068	scLCB8	0.0020	0.1451	CS2	0.0008	0.0088	0.0072	1.0824	CS2	0.0808
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
0.0247	scLCB3	0.1089	OK	OK	scLCB5	-	0.0133	0.0213	scLCB8	0.0062	0.1191	CS2	0.0171	0.0210	0.0187	2.8030	CS2	0.0665
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
0.0473	scLCB3	0.0936	OK	OK	scLCB5	-	0.0249	0.0413	scLCB8	0.0121	0.1057	CS2	0.0322	0.0488	0.0428	6.4096	CS2	0.0587
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
0.0703	scLCB3	0.0688	OK	OK	scLCB5	-	0.0336	0.0673	scLCB8	0.0183	0.0797	CS2	0.0435	0.0769	0.0667	9.9924	CS2	0.0444
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
0.0881	scLCB3	0.0698	OK	OK	scLCB5	-	0.0405	0.0834	scLCB8	0.0241	0.0746	CS2	0.0529	0.0930	0.0807	12.0894	CS2	0.0411
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
0.0921	scLCB3	0.0450	OK	OK	scLCB5	-	0.0466	0.0848	scLCB8	0.0239	0.0486	CS2	0.0604	0.0944	0.0826	12.3809	CS2	0.0268
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
0.1347	scLCB3	0.0490	OK	OK	scLCB5	-	0.0674	0.1207	scLCB8	0.0304	0.0644	CS2	0.1150	0.1524	0.1309	19.5321	CS2	0.0316
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
0.1527	scLCB3	0.0202	OK	OK	scLCB5	-	0.0717	0.1418	scLCB8	0.0322	0.0286	CS2	0.1225	0.1712	0.1464	21.8425	CS2	0.0155
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
0.1658	scLCB3	0.0265	OK	OK	scLCB5	-	0.0734	0.1511	scLCB8	0.0342	0.0384	CS2	0.1265	0.1859	0.1584	23.6357	CS2	0.0127
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
0.1551	scLCB1	0.0125	OK	OK	scLCB5	-	0.0747	0.1407	scLCB8	0.0267	0.0201	CS2	0.1280	0.1737	0.1489	22.2117	CS2	0.0034

Image 3-22. Total Checking Results Table

H. View Design Results

Strength Limit State(Flexure)…

This table shows the Check results for Strength Limit State in flexure as per Article 6.10.6.2.

The Check Results Table for Strength Limit State(Flexure), is as shown in image 3-23.

The design report for Strength Limit State in Positive and Negative flexure is as shown in image 3-24.

Where,

My *: yield moment*
Mp *: plastic moment*
Mu *: moment due to the factored loads*
phiMn *: nominal flexural resistance of a section multiplied by phi of flexure*
fbu *: largest value of the compressive stress throughout the unbraced length in the flange under condition, calculated without consideration of flange lateral bending*
phiFn *: nominal flexure resistance of a flange*
Dp *:distance from the top of the concrete deck to the neutral axis of the composite section at the plastic moment*
Dt *: total depth of the composite section*

	Elem	part	Positive/Negative	Lcom	Type	CHK	My (in·kips)	Mp (in·kips)	Mu (in·kips)	phiMn (in·kips)	fbu (kips/in²)	phiFn (kips/in²)	Dp (in)	0.42Dt (in)
	346	I[256]	Pos	scLCB	-	OK	-	-	-	-	10.5535	68.3557	12.6431	35.4900
	346	J[22]	Neg	-	-	-	-	-	-	-	-	-	-	-
	346	J[22]	Pos	scLCB	-	OK	-	-	-	-	11.4329	68.3557	12.6431	35.4900
	347	I[257]	Neg	-	-	-	-	-	-	-	-	-	-	-
	347	I[257]	Pos	scLCB	-	OK	-	-	-	-	11.3462	68.3557	12.6431	35.4900
	347	J[23]	Neg	-	-	-	-	-	-	-	-	-	-	-
	347	J[23]	Pos	scLCB	-	OK	-	-	-	-	10.3109	68.3557	12.6431	35.4900
	348	I[258]	Neg	-	-	-	-	-	-	-	-	-	-	-
	348	I[258]	Pos	scLCB	-	OK	-	-	-	-	10.0635	68.3557	12.6431	35.4900
	348	J[24]	Neg	-	-	-	-	-	-	-	-	-	-	-
	348	J[24]	Pos	scLCB	-	OK	-	-	-	-	10.0089	68.3557	12.6431	35.4900
	349	I[259]	Neg	-	-	-	-	-	-	-	-	-	-	-
	349	I[259]	Pos	scLCB	-	OK	-	-	-	-	9.0196	68.3557	12.6431	35.4900
	349	J[25]	Neg	-	-	-	-	-	-	-	-	-	-	-
	349	J[25]	Pos	scLCB	-	OK	-	-	-	-	7.1898	68.3557	12.6431	35.4900
	350	I[260]	Neg	-	-	-	-	-	-	-	-	-	-	-
	350	I[260]	Pos	scLCB	-	OK	-	-	-	-	5.9060	68.3557	12.6431	35.4900
	350	J[26]	Neg	-	-	-	-	-	-	-	-	-	-	-
	350	J[26]	Pos	scLCB	MY-M	OK	-	-	-	-	4.6368	68.3557	12.6431	35.4900

Image 3-23. Strength Limit State(Flexure) Results Table

H. View Design Results

II. Strength Limit State - Flexural Resistance

1. Flexure

■ Positive moment

1) Design Forces and Stresses

Loadcombination Name : scLCB1

Loadcombination Type : MY-MAX

Component		M_u (kips·in)				V_u (kips)	T (kips·in)
		Steel (M_{D1})	Long-term (M_{D2})	Short-term	Sum		
Forces	(+)	23950.617	15626.865	13374.975	52952.456	-77.268	-355.313

Component		f_{c1} (ksi)			
		Steel (M_{D1})	Long-term (M_{D2})	Short-term	Sum
Stresses	Top	-9.146	-2.398	-0.801	-12.344
	Bot	8.477	4.710	3.738	16.925

2) Cross-section Proportions

① Web Proportions (AASHTO LRFD Bridge, 2012, 6.10.2.1)

$\frac{D}{t_w}$ = 138.000 ≤ 300 ...... OK

② Flange Proportions (AASHTO LRFD Bridge, 2012, 6.10.2.2)

$\frac{b_f}{2t_w}$ = 4.500 ≤ 12 ...... OK

b_f = 16.000 ≥ D/6 = 11.500 ...... OK

t_f = 2.000 ≥ $1.1t_w$ = 0.550 ...... OK

$I_{yc} = \frac{t_{fc} \cdot b_{fc}^3}{12}$ = 682.667 in^4

$I_{yt} = \frac{t_{ft} \cdot b_{ft}^3}{12}$ = 972.000 in^4

0.1 ≤ $\frac{I_{yc}}{I_{yt}}$ = 0.702 ≤ 10.0 ...... OK

3) Flexural Strength Limit State in positive flexure

▪ Section Classification (AASHTO LRFD Bridge, 2012, 6.10.6.2)

min (F_{yc} , F_{yt}) = 70.000 ksi ≤ 70.0 ksi ...... OK

$\frac{D}{t_w}$ = 138.000 ≤ 150 ...... OK

$\frac{2 \cdot D_{cp}}{t_w}$ = 0.000 ≤ $3.76 \sqrt{\frac{E_s}{F_{yc}}}$ = 76.531 ...... OK

in which :

D_{cp} = 0.000 in

ŸE Noncompact section for Curved Bridge

75_J | 368_I | Shear Connectors | Longitudinal Stiffeners

IV. Strength Limit State - Flexural Resistance

1. Flexure

■ Negative moment

1) Design Forces and Stresses

Loadcombination Name : scLCB1

Loadcombination Type : MZ-MIN

Component		M_u (kips·in)				V_u (kips)	T (kips·in)
		Steel (M_{D1})	Long-term (M_{D2})	Short-term	Sum		
Forces	(-)	-50886.247	-32572.063	-13011.546	-96469.856	379.884	141.034

Component		f_{c1} (ksi)			
		Steel (M_{D1})	Long-term (M_{D2})	Short-term	Sum
Stresses	Top	14.278	8.154	2.669	25.101
	Bot	-13.283	-8.288	-3.183	-24.755

2) Cross-section Proportions

① Web Proportions (AASHTO LRFD Bridge, 2012, 6.10.2.1)

$\frac{D}{t_w}$ = 122.667 ≤ 300 ...... OK

② Flange Proportions (AASHTO LRFD Bridge, 2012, 6.10.2.2)

$\frac{b_f}{2t_w}$ = 4.000 ≤ 12 ...... OK

b_f = 18.000 ≥ D/6 = 11.500 ...... OK

t_f = 2.500 ≥ $1.1t_w$ = 0.619 ...... OK

$I_{yc} = \frac{t_{fc} \cdot b_{fc}^3}{12}$ = 1666.667 in^4

$I_{yt} = \frac{t_{ft} \cdot b_{ft}^3}{12}$ = 1215.000 in^4

0.1 ≤ $\frac{I_{yc}}{I_{yt}}$ = 1.372 ≤ 10.0 ...... OK

③ Minimum Negative Flexure Concrete Deck Reinforcement (AASHTO LRFD Bridge, 2012, 6.10.1.7)

A_{rs} = 15.010 ≥ $0.01A_{deck}$ = 11.160 in^2 ...... OK

in which :

A_{deck} = 1116.000 in^2

3) Flexural Strength Limit State in negative flexure

▪ Section Classification (AASHTO LRFD Bridge, 2012 6.10.6.3)

min (F_{yc}, F_{yt}) = 70.000 ≤ 70.0 ksi ...... OK

$\frac{I_{yc}}{I_{yt}}$ = 1.37 ≥ 0.3 ...... OK

$\frac{2 D_c}{t_w}$ = 121.753 > $5.7 \sqrt{\frac{E_s}{F_{yc}}}$ = 116.018 ...... NG

75_J | 368_I | Shear Connectors | Longitudinal Stiffeners

Image 3-24. Strength Limit State-Flexure Resistance Design Report

H. View Design Results

Strength Limit State(Shear)…

This table shows the Check results for Strength Limit State in Shear as per Article 6.10.6.3.

The Check Results Table for Strength Limit State(Flexure), is as shown in image 3-25.

The design report for Strength Limit State in Positive and Negative flexure is as shown in image 3-26.

Where,

***Vu** : shear due to the factored load*
***phiVn** : nominal shear resistance multiplied by phi*
***bt_lim1** : 2.0+(D/30) as per Eq. 6.10.11.1.2-1*
***bt_lim2** : 16tp as per Eq. 6.10.11.1.2-2*
***bt_lim3** : bf/4 as per Eq. 6.10.11.1.2-2*
***bt** : projected width of transverse stiffener as per Article 6.10.11.1.2*
***It_lim** : limiting moment of inertia of transverse stiffener*
***It** : Moment of Inertia of transverse stiffener as per Article 6.10.11.1.3*

Image 3-26. Strength Limit State-Shear Resistance Design Report

V. Strength Limit State - Shear Resistance

1. Shear

■ Max

1) Design Forces and Stresses

Loadcombination Name : scLCB1

Loadcombination Type : FZ-MIN

Component		M_u (kips·in) / f_{ct} (ksi)				V_u (kips)	T (kips·in)
		Steel	Long-term	Short-term	Sum		
Forces	(+)	23950.617	15626.865	10018.409	49595.890	-85.262	-339.651
Stresses	Top	-9.146	-2.398	-0.600	-12.143	-	-
	Bot	8.477	4.710	2.800	15.987		

2) Shear Resistance (AASHTO LRFD Bridge, 2012, 6.10.9)

▪ Ratio of the shear-buckling resistance to the shear yield strength, C (AASHTO LRFD Bridge, 2012, 6.10.9.3.2)

shear-buckling coefficient of stiffened Webs

$$k = 5 + \frac{5}{\left(\frac{d_0}{D}\right)^2} = 7.939$$

$$\frac{D}{t_w} = 138.000 > 1.40\sqrt{\frac{E \cdot k}{F_{yw}}} = 95.000$$

therefore,

$$C = \frac{1.57}{\left(\frac{D}{t_w}\right)^2} \cdot \left(\frac{E \cdot k}{F_{yw}}\right) = 0.380$$

▪ Nominal Resistance of Stiffened interior Webs (AASHTO LRFD Bridge, 2012, 6.10.9.3.2)

$V_p = 0.58F_{yw} \cdot D \cdot t_w = 1000.500$ kips

$2D \cdot t_w$ … = 1.015 ≤ 2.500

kips …… OK

Longitudinal Stiffeners

	Elem	part	Lcom	Type	CHK	Vu (kips)	phiVn (kips)	bt_lim1 (in)	bt_lim2 (in)	bt_lim3 (in)	bt (in)	It_lim (in^4)	It (in^4)
	20	J[272]	scLCB1	FZ-MAX	OK	92.4433	708.3549	4.3000	24.0000	4.0000	5.0000	62.1816	62.5000
	21	I[37]	scLCB1	FZ-MAX	OK	111.9422	708.3549	4.3000	24.0000	4.0000	5.0000	62.1816	62.5000
	21	J[273]	scLCB1	FZ-MAX	OK	126.5699	708.3549	4.3000	24.0000	4.0000	5.0000	62.1816	62.5000
	22	I[38]	scLCB1	FZ-MAX	OK	151.0565	708.3549	4.3000	24.0000	4.0000	5.0000	62.1816	62.5000
	22	J[274]	scLCB1	FZ-MAX	OK	165.6846	708.3549	4.3000	24.0000	4.0000	5.0000	62.1816	62.5000
	23	I[39]	scLCB1	FZ-MAX	OK	186.7036	850.3163	4.3000	24.0000	4.5000	5.0000	62.1816	62.5000
	23	J[275]	scLCB1	FZ-MAX	OK	201.9425	850.3163	4.3000	24.0000	5.0000	5.0000	62.1816	62.5000
	24	I[40]	scLCB1	FZ-MAX	OK	242.8536	850.3163	4.3000	24.0000	5.0000	5.0000	62.1816	62.5000
	24	J[276]	scLCB1	FZ-MAX	OK	258.0930	850.3163	4.3000	24.0000	5.0000	5.0000	62.1816	62.5000
	25	I[41]	scLCB1	FZ-MAX	OK	282.8914	850.3163	4.3000	24.0000	5.0000	5.0000	62.1816	62.5000
	25	J[277]	scLCB1	FZ-MAX	OK	298.1313	850.3163	4.3000	24.0000	5.0000	5.0000	62.1816	62.5000
	26	I[42]	scLCB1	FZ-MAX	OK	365.6245	850.3163	4.3000	24.0000	5.0000	5.0000	62.1816	62.5000
	26	J[278]	scLCB1	FZ-MAX	OK	380.8649	850.3163	4.3000	24.0000	5.0000	5.0000	62.1816	62.5000
	27	I[6]	scLCB1	FZ-MIN	OK	-279.3709	850.3163	4.3000	24.0000	5.0000	5.0000	62.1816	62.5000
	27	J[279]	scLCB1	FZ-MIN	OK	-263.6937	850.3163	4.3000	24.0000	4.5000	5.0000	62.1816	62.5000
	28	I[43]	scLCB1	FZ-MIN	OK	-221.6682	850.3163	4.3000	24.0000	4.5000	5.0000	62.1816	62.5000
	28	J[280]	scLCB1	FZ-MIN	OK	-205.9909	850.3163	4.3000	24.0000	4.5000	5.0000	62.1816	62.5000
	29	I[44]	scLCB1	FZ-MIN	OK	-166.2912	850.3163	4.3000	24.0000	4.5000	5.0000	62.1816	62.5000
	29	J[281]	scLCB1	FZ-MIN	OK	-150.6137	850.3163	4.3000	24.0000	4.5000	5.0000	62.1816	62.5000
	30	I[45]	scLCB1	FZ-MIN	OK	-115.8761	850.3163	4.3000	24.0000	4.5000	5.0000	62.1816	62.5000
	30	J[282]	scLCB1	FZ-MIN	OK	-100.1984	850.3163	4.3000	24.0000	4.5000	5.0000	62.1816	62.5000

Image 3-25. Strength Limit State(Shear) Results Table

H. View Design Results

Service Limit State…

This table shows the Check results for Service Limit State as per Article 6.10.4.2.

The Check Results Table for Service Limit State, is as shown in image 3-27.

The design report for Service Limit State is as shown in image 3-28.

Where,

fs *: bending stress on web plate*
fcrw *: bending stress limit on web plate*
fcf *: compression-flange stress*
fcf_lim *: limitation of comp.-flange stress*
fct *: tension-flange stress*
fct_lim *: limitation of tension-flange stress*

Image 3-28. Service Limit State Design Report

VI. Service Limit State

■ Positive moment

1) Design Forces and Stresses

Loadcombination Name : scLCB5
Loadcombination Type : MY-MAX

Component		M_s (kips-in) / $f_{c,t}$ (ksi)			
		Steel	Long-term	Short-term	Sum
Forces	(+)	19160.493	11961.549	9935.696	41057.738
Stresses	Top	-7.317	-1.835	-0.595	-9.747
	Bot	6.782	3.605	2.777	13.163

2) Permanent deformation (AASHTO LRFD Bridge, 2012, 6.10.4.2)

▪ Flange Lateral bending Stress (AASHTO LRFD Bridge, 2012, 6.10.1.6)

Because of discretely braced tension flange.

$$f_l = \frac{M_w}{S_l} = \frac{M_w}{t_{ft}(b_{ft})^2/6} = \frac{88.735}{108.000} = 14.252 \text{ ksi}$$

f_l = 14.252 ≤ $0.6F_{yf}$ = 42.000 ksi …… OK

▪ Compression Flange (AASHTO LRFD Bridge, 2012, 6.10.4.2.2)

f_f = -9.747 ksi ≤ $0.95 R_h F_{yf}$ = 64.938 ksi …… OK

▪ Tension Flange (AASHTO LRFD Bridge, 2012, 6.10.4.2.2)

$f_f + f_l / 2$ = 20.290 ksi ≤ $0.95 R_h F_{yf}$ = 64.938 ksi …… OK

in which :

f_f = flange stress due to the Service II loads calculated without consideration of flange lateral bending

F_{yf} = specified minimum yield strength of a flange (ksi)

▪ check stress of the concrete deck

Compact composite section in positive flexure utilized in shored construction

$0.6 f_c'$ = 2.700 ksi …… OK

397.245) · (24.380) / 122.879) · (7.958) = -0.312 ksi

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	Elem	part	Positive/ Negative	Lcom	Type	CHK	fc (kips/in^2)	fcrw (kips/in^2)	fcf (kips/in^2)	fcf_lim (kips/in^2)	ftf (kips/in^2)	ftf_lim (kips/in^2)
	7	I[24]	Pos	scLCB5	MZ-MAX	OK	-	-	-4.2483	64.9380	9.2078	64.9380
	7	J[259]	Neg	-	-	-	-	-	-	-	-	-
	7	J[259]	Pos	scLCB5	MZ-MAX	OK	-	-	-3.9363	64.9380	7.8234	64.9380
	8	I[25]	Neg	-	-	-	-	-	-	-	-	-
	8	I[25]	Pos	scLCB5	MY-MAX	OK	-	-	-3.4035	64.9380	5.7738	64.9380
	8	J[260]	Neg	-	-	-	-	-	-	-	-	-
	8	J[260]	Pos	scLCB5	MZ-MAX	OK	-	-	-2.8223	64.9380	5.4158	64.9380
	9	I[26]	Neg	-	-	-	-	-	-	-	-	-
	9	I[26]	Pos	scLCB5	MZ-MAX	OK	-	-	-1.9799	64.9380	4.4322	64.9380
	9	J[261]	Neg	-	-	-	-	-	-	-	-	-
	9	J[261]	Pos	scLCB5	MZ-MAX	OK	-	-	-1.0844	64.9380	2.9660	64.9380
	10	I[27]	Neg	scLCB5	MY-MIN	OK	-0.6659	28.4393	-0.6659	65.2325	0.2531	65.2325
	10	I[27]	Pos	-	-	-	-	-	-	-	-	-
	10	J[262]	Neg	scLCB5	FZ-MAX	OK	-1.9098	43.2549	-1.9098	65.2325	1.3056	65.2325
	10	J[262]	Pos	-	-	-	-	-	-	-	-	-
	11	I[28]	Neg	scLCB5	MY-MIN	OK	-3.4142	47.0683	-3.4142	65.2325	2.5675	65.2325
	11	I[28]	Pos	-	-	-	-	-	-	-	-	-
	11	J[263]	Neg	scLCB5	MX-MIN	OK	-5.1232	49.0565	-5.1232	65.2325	4.0290	65.2325
	11	J[263]	Pos	-	-	-	-	-	-	-	-	-
	12	I[29]	Neg	scLCB5	MY-MIN	OK	-7.1501	49.9399	-7.1501	65.2325	5.7304	65.2325
	12	I[29]	Pos	-	-	-	-	-	-	-	-	-

Service Limit State

Image 3-27. Service Limit State Results Table

H. View Design Results

Fatigue Limit State…

This table shows the Check results for Fatigue Limit State as per Article 6.10.5.1 and 6.10.5.3.

The Check Results Table for Fatigue Limit State, is as shown in image 3-29.

The design report for Fatigue Limit State is as shown in image 3-30.

Where,

***Lcom** : Load combinations used in the calculation*
***γ(Δf)** : Range of Fatigue Limit State*
***(Δf)n** : Nominal Fatigue Resistance*
***Vu** : maximum shear elasticity stress on web plate*
***Vcr** : shear resistance value*

Image 3-30. Fatigue Limit State Design Report

VIII. Fatigue Limit State

■ **Fatigue moment**

1) Design Forces and Stresses

Loadcombination Name : scLCB8

Component		LCB	M_u (kips-in) / $f_{c,t}$ (ksi)			
			Steel	Long-term	Short-term	Sum
Forces	Top(Tens.)	-	19160.493	11961.549	-3211.178	-3211.178
	Top(Comp.)	-	0.000	0.000	0.000	0.000
	Bot(Tens.)	-	0.000	0.000	0.000	0.000
	Bot(Comp.)	-	19160.493	11961.549	-3211.178	-3211.178
Stresses	Top(Tens.)	-	-6.931	-1.686	0.162	0.162
	Top(Comp.)	-	0.000	0.000	0.000	0.000
	Bot(Tens.)	-	0.000	0.000	0.000	0.000
	Bot(Comp.)	-	6.395	3.456	-0.868	-0.868

Loadcombination Name : scLCB8

Component	V_u (kips)
Shear Force	-69.626

2) Load-Induced Fatigue (AASHTO LRFD Bridge, 2012, 6.6.1.2)

The stress from unfactored DL = 9.851 ksi (- : Compression)
The stress from fatige LCB = -0.868 ksi

Check Load-Induced Fatigue. [The stress from unfactored DL is the tensile stress.]

No	Category	$(ADTT)_{SL}$	Number of stress (n
1	C'	1000.000	1.000

(ADTT)SL(1000.00) > Constant-Amplitude Fatigue Thresholds from Table 6.6.1.2. 745.00)

1.2.5-3 Constant-Amplitude Fatigue Thresholds

$(\Delta F)_n$ = 12.000 ksi ...... OK

ectors | Longitudinal Stiffeners

	Elem	part	CHK	Lcom	Gamma(df) (kips/in^2)	(df)n (kips/in^2)	Lcom	Vu (kips/in^2)	Vcr (kips/in^2)
	13	J[265]	OK	scLCB8	2.0800	12.0000	scLCB8	234.5721	540.7591
	14	I[5]	OK	scLCB8	0.0067	12.0000	scLCB8	-146.5665	540.7591
	14	J[266]	OK	scLCB8	0.4805	12.0000	scLCB8	-134.3773	540.7591
	15	I[31]	OK	scLCB8	0.9018	12.0000	scLCB8	-113.6819	540.7591
	15	J[267]	OK	scLCB8	1.2049	12.0000	scLCB8	-101.4926	540.7591
	16	I[32]	OK	scLCB8	1.4489	12.0000	scLCB8	-83.5094	540.7591
	16	J[268]	OK	scLCB8	1.6953	12.0000	scLCB8	-71.3199	540.7591
	17	I[33]	OK	scLCB8	1.8952	12.0000	scLCB8	-56.8262	540.7591
	17	J[269]	OK	scLCB8	2.0860	12.0000	scLCB8	-44.6366	540.7591
	18	I[34]	OK	scLCB8	2.9536	12.0000	scLCB8	-34.0574	379.7924
	18	J[270]	OK	scLCB8	3.1488	12.0000	scLCB8	-22.3559	379.7924
	19	I[35]	OK	scLCB8	3.2597	12.0000	scLCB8	35.4403	379.7924
	19	J[271]	OK	scLCB8	3.3914	12.0000	scLCB8	47.1420	379.7924
	20	I[36]	OK	scLCB8	3.4549	12.0000	scLCB8	62.7343	379.7924
	20	J[272]	OK	scLCB8	3.4877	12.0000	scLCB8	74.4363	379.7924
	21	I[37]	OK	scLCB8	3.4368	12.0000	scLCB8	90.2373	379.7924
	21	J[273]	OK	scLCB8	3.4418	12.0000	scLCB8	101.9395	379.7924
	22	I[38]	OK	scLCB8	3.3679	12.0000	scLCB8	120.9755	379.7924
	22	J[274]	OK	scLCB8	3.2798	12.0000	scLCB8	132.6780	379.7924
	23	I[39]	OK	scLCB8	2.3430	12.0000	scLCB8	149.8292	540.7591
	23	J[275]	OK	scLCB8	2.3155	12.0000	scLCB8	162.0203	540.7591
	24	I[40]	OK	scLCB8	2.2790	12.0000	scLCB8	194.3328	540.7591

Fatigue Limit State

Image 3-29. Fatigue Limit State Results Table

H. View Design Results

Constructibility(Flexure)…

This table shows the Constructibility Check results for flexure as per Article 6.10.3.2.

The Constructibility Check Results Table for flexure, is as shown in image 3-31.

The design report for Constructibility(Flexure) is as shown in image 3-32.

Where,

fbuw *: bending stress on web plate*
phiFcrw *: bending stress limit on web plate*
fbuc *: compression-flange flexural stress*
phifc *: limitation of compression-flange flexural stress*
fbut *: tension-flange flexural stress*
phift *: limitation of tension -flange flexural stress*
fdeck *: concrete deck flexure elasticity*
phifr *: concrete deck flexure elasticity limit state*

H. View Design Results

Constructibility(Shear)…

This table shows the Constructibility Check results for shear as per Article 6.10.3.3.

The Constructibility Check Results Table for shear, is as shown in image 3-33.

The design report for Constructibility(Shear) is as shown in image 3-34.

Where,

CS *: most critical construction stage for shear before composite action*
Step *: step in the most critical Construction stage*
Vu *: shear due to the factored load*
phiVcr *: shear-buckling resistance multiplied by phi*

Elem	part	Positive/Negative	Lcom	CS	Step	CHK	fbuw (kips/in^2)	phiFcrw (kips/in^2)	fbuc (kips/in^2)	phiFc (kips/in^2)	fbut (kips/in^2)	phift (kips/in^2)	fdeck (kips/in^2)	phifr (kips/in^2)
12	I[29]	Neg	scLCB3	CS2	1	OK	5.7992	67.5756	10.3286	68.6658	11.8255	68.6658	0.2827	0.4582
12	J[264]	-	-	-	-	-	-	-	-	-	-	-	-	-
12	J[264]	Neg	scLCB3	CS2	1	OK	7.9048	67.5756	14.8221	68.6658	17.0368	68.6658	0.3853	0.4582
13	I[30]	-	-	-	-	-	-	-	-	-	-	-	-	-
13	I[30]	Neg	scLCB3	CS2	1	OK	10.2147	67.5756	20.1595	68.6658	23.2574	68.6658	0.4979	0.4582
13	J[265]	-	-	-	-	-	-	-	-	-	-	-	-	-
13	J[265]	Neg	scLCB3	CS2	1	OK	12.7711	67.5756	19.9160	68.6658	22.5486	68.6658	0.6226	0.4582
14	I[5]	-	-	-	-	-	-	-	-	-	-	-	-	-
14	I[5]	Pos	scLCB3	CS2	1	OK	0.0473	57.8753	1.0006	68.6658	0.8162	68.6658	-0.0021	0.4582
14	J[266]	-	-	-	-	-	-	-	-	-	-	-	-	-
14	J[266]	Pos	scLCB3	CS2	1	OK	1.5137	57.8753	1.5730	68.6658	1.4562	68.6658	-0.0686	0.4582
15	I[31]	-	-	-	-	-	-	-	-	-	-	-	-	-
15	I[31]	Pos	scLCB3	CS2	1	OK	2.8098	57.8753	4.8705	68.6658	4.2832	68.6658	-0.1274	0.4582

Constructibility(Flexure)

Image 3-31. Constructibility (Flexure) Results Table

H. View Design Results

VII. Constructibility

1. Flexure

■ **Positive moment**

1) Design Forces and Stresses

Construction Stage CS2

Step : 1

Component		M_u (kips·in) / $f_{c,t}$ (ksi) Steel Section Only
Force	(+)	28740.740
Stress (f_{bu})	Top	-10.975
	Bot	10.172

2) Check slenderness of web (AASHTO LRFD Bridge, 2012, 6.10.6.2.3-1)

$\frac{2 \cdot D_c}{t_w}$ = 143.541 > $5.7\sqrt{\frac{E_s}{F_{yc}}}$ = 116.018 ...... Slender Web

in which :

D_c = 35.885 in

3) Discretely Braced Flanges in Compression (AASHTO LRFD Bridge, 2012, 6.10.3.2.1)

▪ Web Load-Shedding Factor, Rb (AASHTO LRFD Bridge, 2012, 6.10.1.10.2)

In constructibility (AASHTO LRFD Bridge, 2012, 6.10.3.2.1)

R_b = 1.000

▪ Limiting Unbraced Length, Lp (AASHTO LRFD Bridge, 2012, 6.10.8.2.3)

$L_p = 1.0\, r_t \sqrt{\frac{E}{F_{yc}}}$ = 86.292 in

in which :

r_t = effective radius of gyration for lateral torsional buckling

$= \frac{b_{fc}}{\sqrt{12\left(1 + \frac{1}{3}\frac{D_c \cdot t_w}{b_{fc} \cdot t_{fc}}\right)}}$ = 4.240 in

▪ Moment Gradient Modifer, Cb (AASHTO LRFD Bridge, 2012, 6.10.8.2.3)

Calculation of Stress (C6.4.10)

f_0 = 10.025 ksi

f_2 = 10.975 ksi

75_J | 368_I | Shear Connectors | Longitudinal Stiffeners

Image 3-32. Constructibility-Flexure Design Report

3. Shear

■ **Max**

1) Design Forces

Construction Stage CS2

Step : 1

Component	V_u (kips) Steel Section Only
Force	239.518

2) Shear requirement for webs (AASHTO LRFD Bridge, 2012, 6.10.3.3)

▪ Ratio of the shear-buckling resistance to the shear yield strength, C (AASHTO LRFD Bridge, 2012, 6.10.9.3.2)

shear-buckling coefficient of stiffened Webs

$k = 5 + \frac{5}{\left(\frac{d_0}{D}\right)^2}$ = 7.939

$\frac{D}{t_w}$ = 122.667 > $1.40\sqrt{\frac{E \cdot k}{F_{yw}}}$ = 95.000

therefore,

$C = \frac{1.57}{\left(\frac{D}{t_w}\right)^2} \cdot \left(\frac{E \cdot k}{F_{yw}}\right)$ = 0.480

▪ Nominal Resistance of Stiffened interior Webs

$V_p = 0.58 F_{yw} \cdot D \cdot t_w$ = 1125.563 kips

$\frac{2D \cdot t_w}{b_{fc} \cdot t_{fc} + b_{ft} \cdot t_{ft}}$ = 0.817 ≤ 2.500

therefore,

75_J | 368_I | Shear Connectors | Longitudinal Stiffeners

Image 3-34. Constructibility-Shear Design Report

	Elem	part	Lcom	CS	Step	CHK	Vu (kips)	phiVcr (kips)
	407	J[79]	scLCB3	CS2	1	OK	45.0812	850.3163
	408	I[318]	scLCB3	CS2	1	OK	-256.2217	850.3163
	408	J[81]	scLCB3	CS2	1	OK	-243.6682	850.3163
	409	I[319]	scLCB3	CS2	1	OK	-191.2078	850.3163
	409	J[82]	scLCB3	CS2	1	OK	-178.6537	850.3163
	410	I[320]	scLCB3	CS2	1	OK	-162.3577	850.3163
	410	J[83]	scLCB3	CS2	1	OK	-149.8030	850.3163
	411	I[321]	scLCB3	CS2	1	OK	-125.1395	850.3163
	411	J[84]	scLCB3	CS2	1	OK	-112.5841	850.3163
	412	I[322]	scLCB3	CS2	1	OK	-100.4050	708.3549
	412	J[85]	scLCB3	CS2	1	OK	-88.5836	708.3549
	413	I[323]	scLCB3	CS2	1	OK	-79.0249	708.3549
	413	J[86]	scLCB3	CS2	1	OK	-67.2028	708.3549

Constructibility(Shear)

Image 3-33. Constructibility (Shear) Results Table

H. View Design Results

Shear Connector…

This table shows the Shear Connector Check results for Fatigue Limit State and Strength Limit State as per Article 6.10.10.2 and 6.10.10.4 respectively.

The Check Results Table for Shear Connector, is as shown in image 3-35.

The design report for Shear Connector is as shown in image 3-36.

Where,

H/D *: Height to Diameter Ratio (> 4.0)*
(H/D)lim *: Height to Diameter Ratio Limit Value(=4.0)*
p *: Pitch*
p_lim1 *: Pitch Limit Value ->nZI/(Vsr)*
p_lim2 *: Pitch Limit Value -> 4*d*
s *: shear connector spacing(Transverse Cross Section)*
edge *: distance of the top compression flange edge_lim (=1.0 in)*
Cover *: Value of Cover (> 2.0 in)*
Penetration *: The depth of penetration of the shear connector(>2.0in)*
n *: number of shear connectors in each row transversely*
n_Req *: Total number of shear connectors required*

IX. Shear Connertors

Element	75
Position	J

There is no Shear Connector Information. Skip this check.

Element	368
Position	I

Loadcombination Name : scLCB8

1. Types (ASHTO LRFD Bridge Design Specifications, 2012, 6.10.10.1.1)

$\frac{H}{d}$ = 8.000 ≥ 4.000

in which :

H = 7.000 in (height of stud)
d = 0.875 in (diameter of stud)

2. Pitch (AASHTO LRFD Bridge, 2012, 6.10.10.1.2)

1) Shear Fatigue Resistance (AASHTO LRFD Bridge, 2012, 6.10.10.2)

$(ADTT)_{SL}$ ≥ 960
$Z_r = 5.5 \cdot d^2$ = 4.211 kips

in which :

d = 0.875 in (Diameter of stud)
$(ADTT)_{SL}$ = 1000.000 (Article 3.6.1.4.2)

75_J | 368_I | Shear Connectors | Longitudinal Stiffener

Image 3-30. Fatigue Limit State Design Report

Image 3-36. Shear Connector Design Report

5. Strength Limit State

1) Factored Shear Resistance of a single shear connector (AASHTO LRFD Bridge, 2012, 6.10.10.4.1)

$Q_{ncal} = 0.5A_{sc} \cdot \sqrt{(f_c' E_c)}$ = 38.502 kips
$Q_{nlim} = A_{sc} \cdot F_u$ = 36.079 kips
∴ $Q_n = \min(Q_{ncal}, Qn_{lim})$ = 36.079 kips
$Q_r = \Phi_{sc} \cdot Q_n$ = 30.667 kips

in which :

f_c' = 4.500 ksi
E_c = 3644.147 ksi
A_{sc} = 0.601 in^2
F_u = 60.000 ksi
Φ_{sc} = 0.850

2) Nominal Shear Force (Positive Flexure, AASHTO LRFD Bridge, 2012, 6.10.10.4.2)

- Nominal Shear Force

$P_{1p} = 0.85f_c' \cdot b_s \cdot t_s$ = 3924.450 kips
$P_{2p} = F_{yw} \cdot D \cdot t_w + F_{yt} \cdot b_{ft} \cdot t_{ft} + F_{yc} \cdot b_{fc} \cdot t_{fc}$ = 8590.625 kips
∴ $P_p = \min(P_{1p}, P_{2p})$ = 3924.450 kips
$P_{1n} = F_{yw} \cdot D \cdot t_w + F_{yt} \cdot b_{ft} \cdot t_{ft} + F_{yc} \cdot b_{fc} \cdot t_{fc}$ = 8590.625 kips
$P_{2n} = 0.45f_c' \cdot b_s \cdot t_s$ = 2077.650 kips
∴ $P_n = \min(P_{1n}, P_{2n})$ = 2077.650 kips
$P_T = P_p + P_n$ = 3924.450 + 2077.650 = 6002.100 kips
F_T = 656.112 kips
∴ $P = \sqrt{[(P_T)^2 + (F_T)^2]}$ = 6037.855 kips

in which :

75_J | 368_I | Shear Connectors | Longitudinal Stiffeners

Elem	part	Lcom	Type	CHK	H/D (in)	(H/D)lim (in)	p (in)	p_lim1 (in)	s (in)	p_lim2 (in)	edge (in)	edge_lim (in)	Cover (in)	Penetration (in)	n	n_req
11	I[28]	scLCB8	-	OK	8.0000	4.0000	5.0000	40.1182	4.0000	3.5000	4.5625	1.0000	4.5000	4.5000	3.000	197.000
11	J[263]	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
12	I[29]	scLCB8	-	OK	8.0000	4.0000	5.0000	32.7521	4.0000	3.5000	4.5625	1.0000	4.5000	4.5000	3.000	197.000
12	J[264]	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
13	I[30]	scLCB8	-	OK	8.0000	4.0000	5.0000	24.3975	4.0000	3.5000	4.5625	1.0000	4.5000	4.5000	3.000	197.000
13	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Shear Connector

Image 3-35. Shear Connector Results Table

H. View Design Results

Longitudinal Stiffener…

This table shows the Check results for Longitudinal Stiffener as per Article 6.10.11.3.

In this tutorial, longitudinal stiffener is not entered. Once the user enters the longitudinal stiffener in Section Properties dialog box, The design report for Longitudinal Stiffener is as shown in image 3-38.

Where,

***bl** : Projected width*
***bl_lim** : Limit of projected width*
***I** : Moment of inertia of cross-section*
***I_lim** : Limit of moment of inertia of cross-section*
***r** : Turning Radius*
***r_lim** : Limit of turning radius*
***fs** : Horizontal stiffeners flexure elasticity*
***phiRhFys** : Horizontal stiffeners flexure elasticity*

X. Stiffeners

1. Longitudinal Stiffeners

Element	75
Position	J

1) Longitudinal Stiffeners (AASHTO LRFD Bridge, 2012, 6.10.11.3)

① Projecting Width (AASHTO LRFD Bridge, 2012, 6.10.11.3.2)

b_l = 5.000 ≤ $0.48\, t_s \sqrt{\frac{E}{F_{ys}}}$ = 23.120 in ...... OK

in which :

t_s = 2.000 in (thickness of longitudinal stiffener)

F_{ys} = 50.000 ksi

② Moment of Inertia and Radius of Gyration (AASHTO LRFD Bridge, 2012, 6.10.11.3)

I_l = 167.792 ≥ $D \cdot t_w^3 \cdot (2.4 \cdot (\frac{d_0}{D})^2 - 0.13)\beta$ 76.967 in^4 ...... OK

r = 3.402 ≥ $\dfrac{0.16\, d_0 \cdot \sqrt{\frac{F_{ys}}{E}}}{\sqrt{(1 - 0.6\frac{F_{yc}}{R_h \cdot F_{ys}})}}$ = 1.599 in ...... OK

in which :

I_l : Moment of inertia of a longitudinal web stiffener (in^4)

r : Radius of gyration of a longitudinal web stiffener (in)

d_0 = 90.000 in (transverse stiffener spacinig)

t_w = 0.500 in (thickness of web)

β = Z/6 + 1 = 2.257

Z = $\min[\frac{0.95\, d_0^2}{R \cdot t_w}, 10.0]$ = 7.544

R = 2040.000 in (Girder radius)

2) Flexural stress in the longitudianl stiffeners (AASHTO LRFD Bridge, 2012, 6.10.11.3.1)

f_s = 7.775 ≤ $\Phi_f \cdot R_h \cdot F_{ys}$ = 48.826 ksi ...... OK

in which :

Φ_f = 1.000

75_J | 368_I | Shear Connectors | **Longitudinal Stiffeners**

Image 3-38. Long. Stiffener Design Report

Cross Frame Design

Steel plate girder bridges make use of traditional cross-frame diaphragms to stabilize the compression flange of girders. These braces are required during construction, especially during deck placement, to prevent lateral torsional buckling of bridge girders. Girder buckling capacity is a function of cross-frame diaphragm spacing as well as strength and stiffness.

Bracings may be temporary or permanent. Most of them are required during wet concrete construction condition. Once the concrete has hardened, the bracing is redundant. Also leaving the bracing in place means that they will take up loads and thus have to be designed.

Midas Civil provides Steel design as per AASHTO-LRFD 2012(US). This feature can be used to design the steel bracings.

Design Steps:

A. Generate Load Combinations
B. Input Design Information
C. View Design Results

Note: Cross Frame Design is included in this tutorial only for completeness of Steel Composite I girder bridge design using midas Civil. The input design parameters and the design results for Cross Frame Design are not discussed in this tutorial. For any explanation you can refer to our online help manual or previous tutorials on steel design.

A. Generate Load Combinations

In this tutorial we will Auto Generate Load Combinations for Steel Design as per AASHTO LRFD 2012.

❑Go to Results > Load Combinations

Click Steel Design

Click Auto Generation...

Automatic Generation of Load Combinations > Design Code > AASHTO-LRFD 12
Automatic Generation of Load Combinations > Manipulations of CS Load Cases > Select CS Only Refer to image 4-1

Click OK
You can view the Auto generated load combinations as in image 4-2.
Click Close

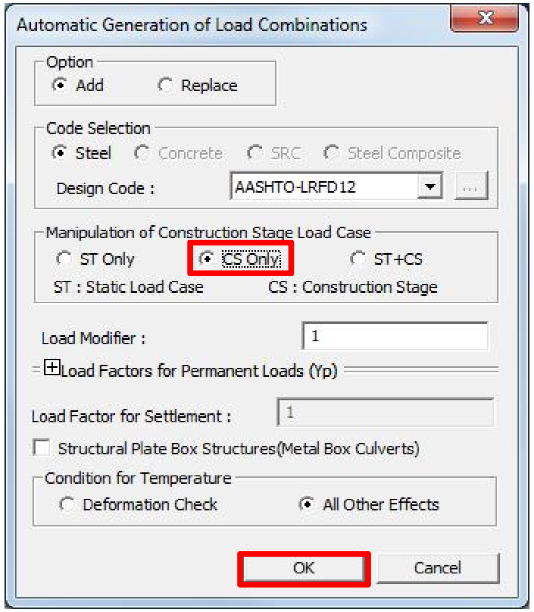
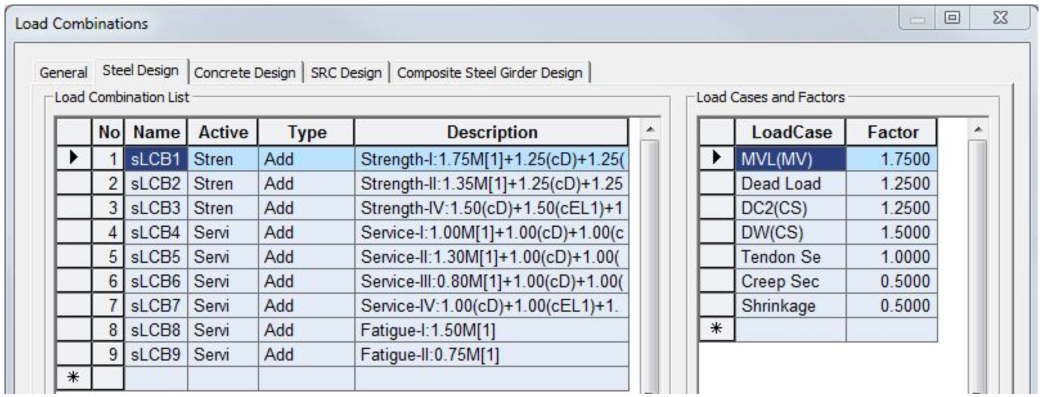


Image 4-1. Auto Generation Load Combinations Dialog Box



No	Name	Active	Type	Description
1	sLCB1	Stren	Add	Strength-I:1.75M[1]+1.25(cD)+1.25(
2	sLCB2	Stren	Add	Strength-II:1.35M[1]+1.25(cD)+1.25
3	sLCB3	Stren	Add	Strength-IV:1.50(cD)+1.50(cEL1)+1
4	sLCB4	Servi	Add	Service-I:1.00M[1]+1.00(cD)+1.00(c
5	sLCB5	Servi	Add	Service-II:1.30M[1]+1.00(cD)+1.00(
6	sLCB6	Servi	Add	Service-III:0.80M[1]+1.00(cD)+1.00(
7	sLCB7	Servi	Add	Service-IV:1.00(cD)+1.00(cEL1)+1.
8	sLCB8	Servi	Add	Fatigue-I:1.50M[1]
9	sLCB9	Servi	Add	Fatigue-II:0.75M[1]

LoadCase	Factor
MVL(MV)	1.7500
Dead Load	1.2500
DC2(CS)	1.2500
DW(CS)	1.5000
Tendon Se	1.0000
Creep Sec	0.5000
Shrinkage	0.5000

Image 4-2. Auto Generated Load Combinations

Cross Frame Design

B. Input Design Information

❑ Go to Design > Steel Design > Design Code

Steel Design Code > AASHTO-LRFD12(US)
Steel Design Code > Check ☑ All Beams/Girders are Laterally Braced
Click OK

❑ Go to Design > Steel Design > Strength Reduction Factor

Click Update By Code
Click OK
Note: You can manually enter the strength reduction factors as well.

❑ Go to Design > Steel Design > Modify Steel Material

Material List > Select material ID 1
Steel Material Selection > Code > ASTM09(S)
Steel Material Selection > Grade > A709-HPS70W
Click Modify
Click Close

❑ Go to Design > Steel Design > Steel Code Check

Perform Steel Code Check.

C. View Design Results

"*** End Writing Steel Code Checking Result to Table."; this message in the message window indicates the completion of Steel Code Check. After the check is complete, a new window, "AASHTO-LRFD12 Code Checking Result Dialog" pops out automatically. Refer to image 4-3.

Code Checking Result Dialog > Sorted by > Property
Note: You can see that the check is NG (Not Good) for Angle Section L8xL8x7/8. Thus this section will be changed.

Code Checking Result Dialog > Select NG | 280 | 4 | ☑ | Angle sec, L8X8X7/8 | 1.080 | 0.118 | A709-HPS70W | 70.0000
Click Change...
Change Steel Properties Dialog > Click Search Satisfied Section
Refer to image 4-4.
Select L8X8X1 | OK | ☑ | 1- | 0.961 | 0.103 | 8.0000 | 8.0000 | 15.100
Click Change & Close

Select OK | 280 | 4 | ☑ | Angle sec, L8X8X1 | 0.961 | 0.103 | A709-HPS70W | 70.0000
Click Update...
Update Changed Properties Dialog > Click Select All Changed Properties

Click <-
Analysis/design results will be deleted. Continue> Click Yes
Click Re-analysis
Click Re-check

Note: All the Steel Code Check Results are OK now. Refer to image 4-5

C. View Design Results

Code Checking Result Dialog > Sorted by > Member

Note: You can select any member and check the design results for it.

For example: Select [OK | 118 | 3 | ☑ | End Diaphragm, W16X45 / 0.080 | 0.020 | A709-HPS70W | 70.0000]
Click [Graphic...] to see the graphic report. Refer to image 4-6.
Click [Detail...] to see the detailed report. Refer to image 4-7.
Click [Summary...] to see the summary of the design results for the selected members. Refer to image 4-8.
Code Checking Result Dialog > Click [>>] to see the Steel Code Check Results table.
Code Checking Result Dialog > Click [Close]

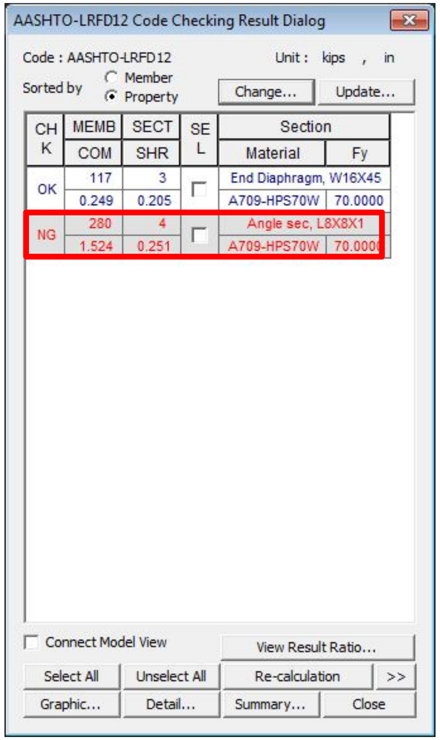


Image 4-3. Steel Code Checking Result Dialog Before Section Update

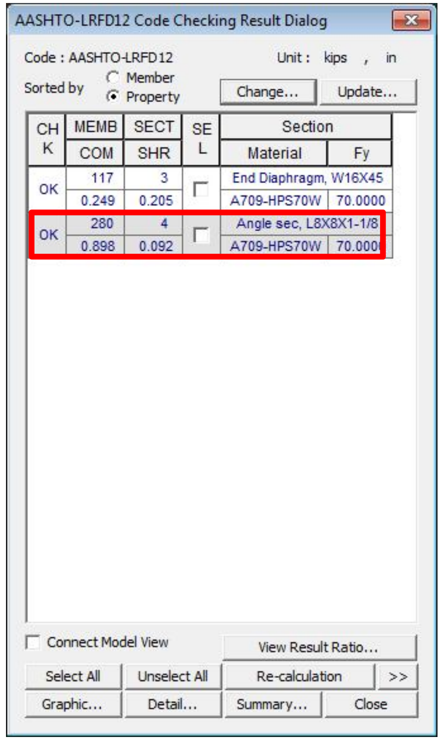


Image 4-5. Steel Code Checking Result Dialog After Section Update

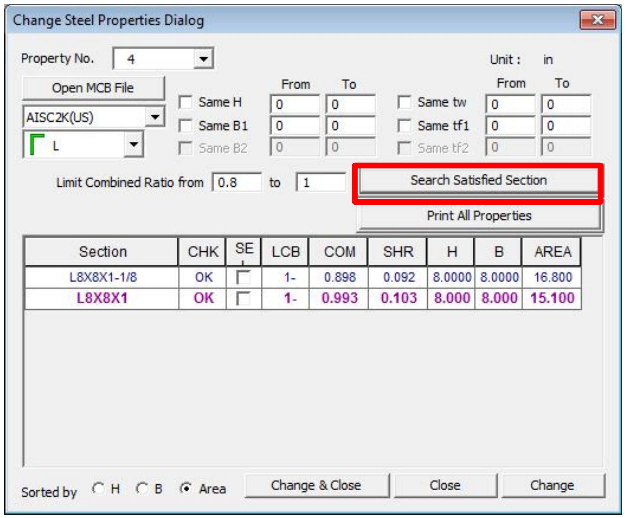
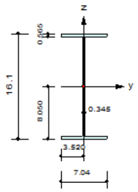


Image 4-4. Change Section Properties Dialog Box

C. View Design Results

1. Design Information

Design Code : AASHTO-LRFD12
Unit System : kips, in
Member No : 117
Material : A709-HPS70W (No:1)
(Fy = 70.0000, Es = 29000.0)
Section Name : End Diaphragm (No:3)
(Rolled : W16X45).
Member Length : 95.7907



Depth	16.1000	Web Thick	0.34500
Top F Width	7.04000	Top F Thick	0.56500
Bot.F Width	7.04000	Bot.F Thick	0.56500
Area	13.3000	Asz	5.55450
Qyb	117.566	Qzb	6.19520
Iyy	586.000	Izz	32.8000
Ybar	3.52000	Zbar	8.05000
Syy	72.7000	Szz	9.34000
ry	6.65000	rz	1.57000

2. Member Forces

Axial Force Fxx = 34.9113 (LCB: 1-, POS:J)
Bending Moments My = -945.63, Mz = -21.128
End Moments Myi = -448.89, Myj = -317.94 (for Lb)
Myi = -448.89, Myj = -317.94 (for Ly)
Mzi = 15.3959, Mzj = -21.128 (for Lz)
Shear Forces Fyy = 0.45588 (LCB: 3, POS:I)
Fzz = 14.2319 (LCB: 1+, POS:J)

3. Design Parameters

Unbraced Lengths Ly = 95.7907, Lz = 95.7907, Lb = 95.7907
Effective Length Factors Ky = 1.00, Kz = 1.00
Moment Factor / Bending Coefficient
Cmy = 1.00, Cmz = 1.00, Cb = 1.00

4. Checking Results

Slenderness Ratio
KL/r = 57.4 < 120.0 (Memb:120, LCB: 1-)................................ O.K
Axial Strength
Pu/phiPn = 34.911/884.450 = 0.039 < 1.000 .... O.K
Bending Strength
Muy/phiMny = 945.63/4664.32 = 0.203 < 1.000 .... O.K
Muz/phiMnz = 21.128/799.816 = 0.026 < 1.000 .... O.K

Image 4-6. Graphic Steel Code Check Report

```
----------------------------------------------------------------------------------------
MIDAS/Civil - Steel Code Checking [ AASHTO-LRFD12 ]                          Version 8.3.1
========================================================================================

    ( ). Compute magnified moments.
         -. Muy  =  B1y*My(DL+LL) + B2y*My(WL(EL)) =     -317.94 in-kips.
         -. Muz  =  B1z*Mz(DL+LL) + B2z*Mz(WL(EL)) =      -21.13 in-kips.

========================================================================================
  [[[*]]]   CHECK AXIAL STRENGTH.
========================================================================================

    ( ). Check slenderness ratio of axial tension member (l/r).
         [ AASHTO-LRFD12 Specification 6.8.4 ]
         -. l/r  =     61.0  <   200.0  ---> O.K.

    ( ). Calculate axial tensile strength (phiPn).
         [ AASHTO-LRFD12 Specification 6.8.2.1-1 ]
         -. Resistance factor for tension  :  phi = 0.95
         -. phiPn  =  phi*Fy*Ag  =        884.45 kips.

    ( ). Check ratio of axial strength (Pu/phiPn).
              Pu                 34.91
         -. -------  =  ---------------  =  0.039  <  1.000  ---> O.K.
             phiPn              884.45

========================================================================================
  [[[*]]]   CHECK SECTION PROPORTION LIMITS
========================================================================================

    ( ). Check Web Proportions (AASHTO-LRFD12 Eq.6.10.2.1.1-1)

         -. Limit Value  = 150
         -. D/tw  =   43.39  <  Limit Value  ---> O.K.

    ( ). Check Flange Proportions (AASHTO-LRFD12 Eq.6.10.2.2-1)

         -. bf/2tf  =    6.23  <    12.00  ---> O.K.
```

Image 4-7. Detailed Steel Code Check Report

```
-----------------------------------------------------------------------------------------
MIDAS/Civil - Steel Code Checking [ AASHTO-LRFD12 ]                        Version 8.3.1
=========================================================================================

 *.PROJECT     :
 *.UNIT SYSTEM : kips, in
=================================================================================================
   [ AASHTO-LRFD12 ]  CODE CHECKING SUMMARY SHEET --- SELECTED MEMBERS IN ANALYSIS MODEL.
-------------------------------------------------------------------------------------------------
      MEMB  SECT Section              Len     Ly    Cb    Ky  B1y  B2y     Pu     Muy     Muz
 CHK   COM   SHR Material       Fy  LCB       Lb     Lz          Kz  B1z  B2z    pPn    pMny    pMnz
=================================================================================================
       117     3 End Diaphragm, W16X45  95.7907 95.7907 1.00 1.00 1.00 1.00 34.9113 -945.63 -21.128
  OK  0.25  0.21 A709-HPS~ 70.0000    1- 95.7907 95.7907      1.00 1.00 1.00 884.450 4664.32 799.816
-------------------------------------------------------------------------------------------------
```

Image 4-8. Steel Code Check Summary Report

Pier & Pier Table Design

Traditionally, piers have been designed using conventional methods of strength of materials regardless of member dimensions. In this approach, it is assumed that longitudinal strains vary linearly over the depth of the member and the shear distribution remains uniform. Furthermore, separate designs are carried out for V_u and M_u at different locations along the member.

In midas Civil as well, all pier components, regardless of dimensions, can be designed in accordance with the conventional strength of materials assumptions described above. This approach is currently standard engineering practice. Pier table components can be designed as simple beams in midas Civil.

Design Steps:

A. Generate Load Combinations
B. Input Design Information
C. View Design Results

Note: Pier & Pier Table Design is included in this tutorial only for completeness of Steel Composite I girder bridge design using midas Civil. The input design parameters and the design results for Pier & Pier Table are not discussed in this tutorial. For any explanation you can refer to our online help manual or previous tutorials on concrete design.

A. Generate Load Combinations

In this tutorial we will Auto Generate Load Combinations for Concrete Design as per AASHTO LRFD 2012.

❑Go to Results > Load Combinations

Click Concrete Design

Click Auto Generation...

Automatic Generation of Load Combinations > Design Code > AASHTO-LRFD 12
Automatic Generation of Load Combinations > Manipulations of CS Load Cases > Select CS Only Refer to image 5-1

Click OK
You can view the Auto generated load combinations as in image 5-2.
Click Close

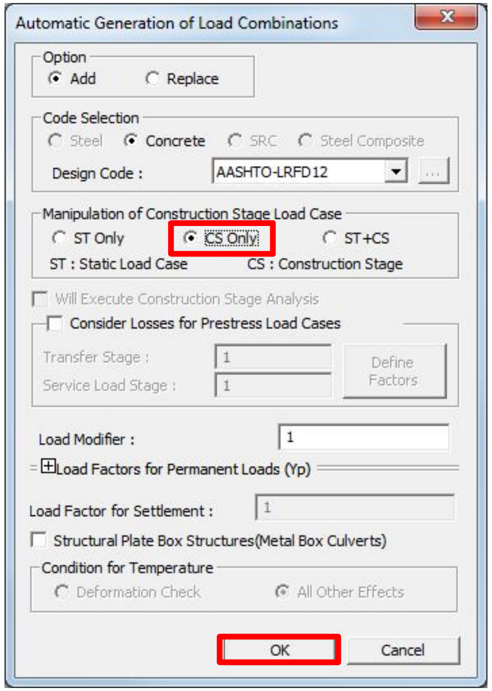


Image 5-1. Auto Generation Load Combinations Dialog Box

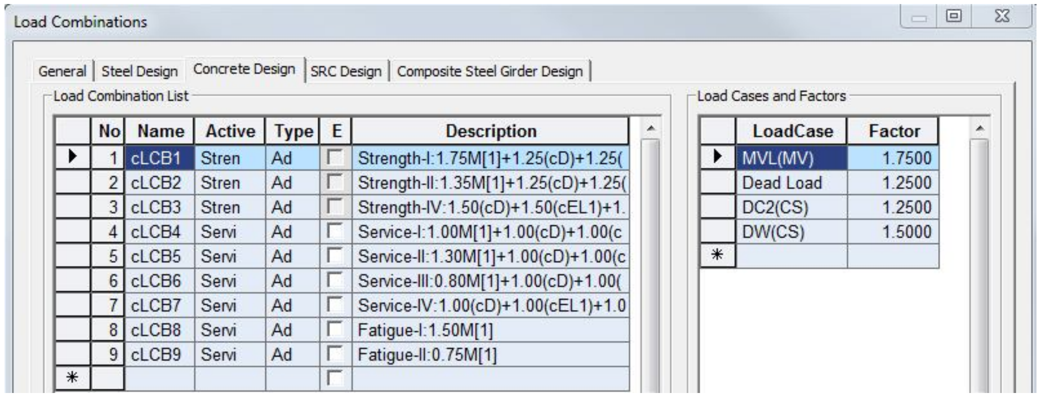


Image 5-2. Auto Generated Load Combinations

B. Input Design Information

❑ Go to Design > RC Design > Design Code

Concrete Design Code > AASHTO-LRFD12(US)
Concrete Design Code > Check ☑ Apply Special Provisions for Seismic Design
Concrete Design Code > Select ⦿ Seismic Zone 3
Click OK

❑ Go to Design > RC Design > Strength Reduction Factor

Click Update By Code
Click OK
Note: You can manually enter the strength reduction factors as well.

❑ Go to Design > RC Design > Modify Concrete Material

Material List > Select material ID 2

ID	Name	fc|fck|R	Chk	Lambda
2	Concrete	4.5	X	1

Concrete Material Selection > Code > ASTM(RC)
Concrete Material Selection > Grade > C4500
Rebar Selection > Code > ASTM(RC)
Rebar Selection > Grade of Main Rebar > 60
Rebar Selection > Grade of Sub Rebar > 50
Click Modify
Click Close

❑ Go to Design > RC Design > Limiting Maximum Rebar Ratio

Click OK

❑ Go to Design > RC Design > Beam Section Data for Design

Section List > Select section ID 5

ID	Name
5	P1_C

Stirrup Data > Size > #10
Stirrup Data > Number > 5
Stirrup Data > Dt > 2in
Stirrup Data > Db > 2in

Refer to image 5-3.

Click Add/Replace
Click Cancel

❑ Go to Design > RC Design > Concrete Code Design > Beam Design

"*** Finished Writing RC Beam Design Result to Table."; this message in the message window indicates the completion of RC Beam (Pier Table) Design. After the design is complete, a new window, "AASHTO-LRFD12 RC-Beam Design Result Dialog" pops out automatically. Refer to image 5-4.

C. View Design Results

Note: All the Pier Table Design Results are OK. Refer to image 5-4

RC Beam Design Result Dialog > Sorted by > Select ◉ Member

Note: You can select any member and check the design results for it.

For example: Select

317		P1_C		4.50000	I	***T
5	☑	60.00	60.00	60.0000	M	OK
89.907		0.000	0.000	50.0000	J	OK

Click [Graphic...] to see the graphic report. Refer to image 5-5.
Click [Detail...] to see the detailed report. Refer to image 5-6.
Click [Summary...] to see the summary of the design results for the selected members. Refer to image 5-7.

RC Beam Design Result Dialog > Click [>>] to see the Pier Table Design Results table.

RC Beam Design Result Dialog > Click [Close]

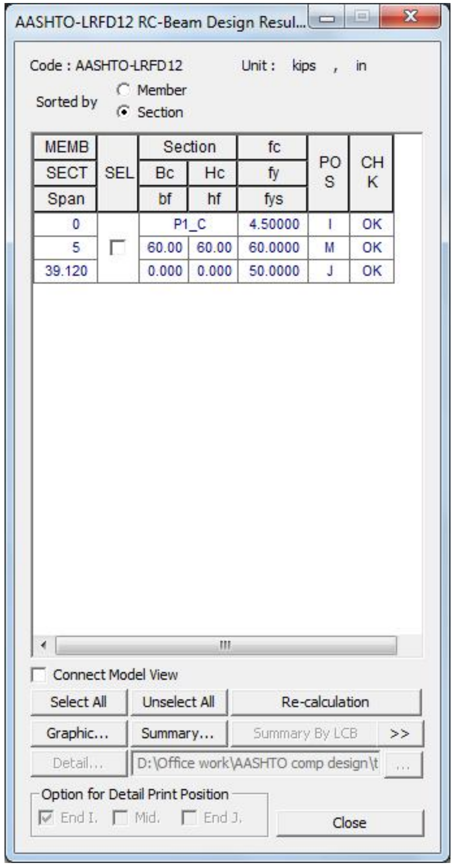
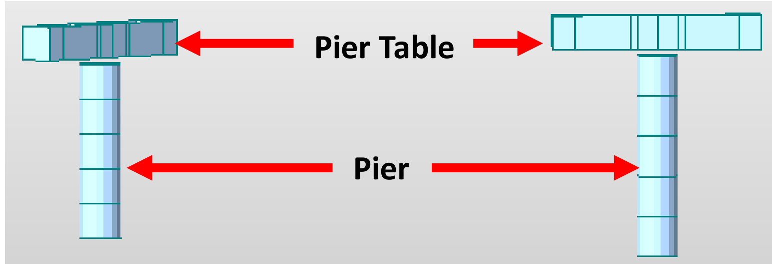


Image 5-3. Beam Section Data for Design Dialog Box

Fig : Pier & Pier Table Model View



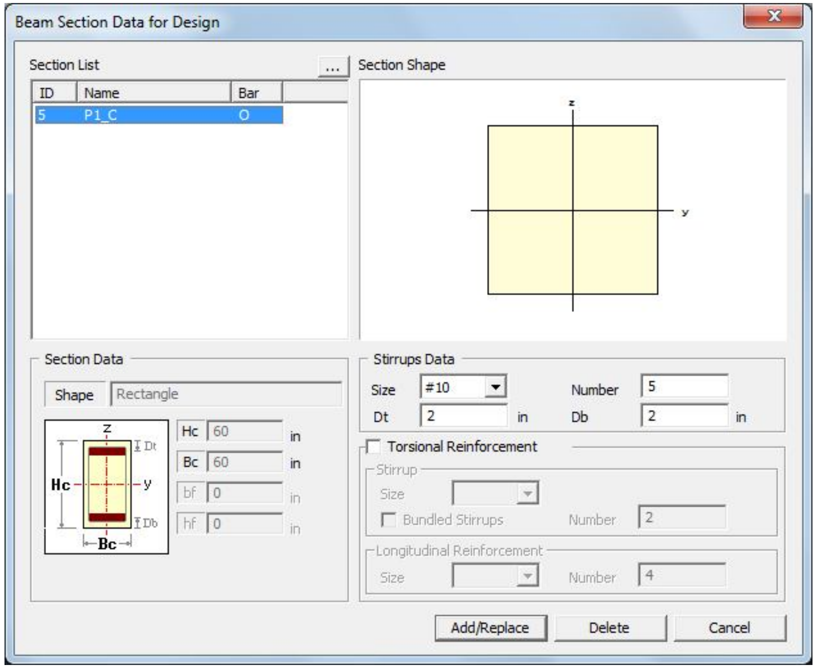


Image 5-4. RC-Beam Design Result Dialog Box

C. View Design Results

1. Design Information

Member Number : 317
Design Code : AASHTO-LRFD12
Unit System : kips, in
Material Data : fc = 4.5, fy = 60, fys = 50 ksi
Beam Span : 89.9068 in
Section Property : P1_C (No : 5)

2. Section Diagram

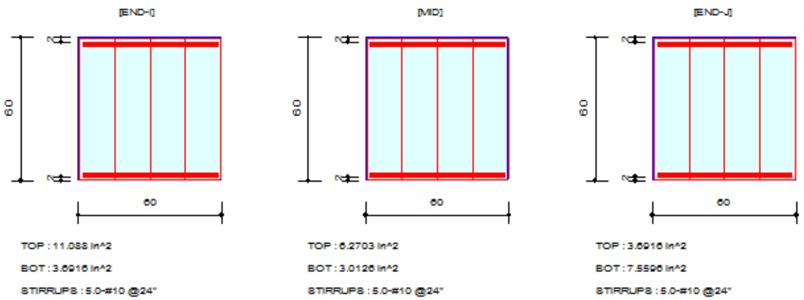


Image 5-5. Graphic RC Beam Design Report

```
==========================================================================================================
 [ AASHTO-LRFD12 ]  RC-BEAM DESIGN SUMMARY SHEET --- SELECTED MEMBERS IN ANALYSIS MODEL.
----------------------------------------------------------------------------------------------------------

*.MEMB =     317,  SECT =       5 (P1_C, SB),  Span = 89.9068
*.Bc    =  60.000,  Hc    =  60.000
*.fc    = 4.50000,  fy    = 60.0000,  fys   = 50.0000
----------------------------------------------------------------------------------------------------------
 POS CHK  |     N-Mu ( LCB)  AsTop |   P-Mu ( LCB)  AsBot |      Vu ( LCB)      Av       Tu ( LCB)      St   Stirrups
----------+-----------------------+----------------------+-----------------------------------------------------------
   I OK   | 32951.5 (  1-) 11.088 | 0.00000 (  1+) 3.6916 | 639.919 (  1-) 3.1750 23117.9 (  1+) 16.985 5.0-#10 @24"
   M OK   | 18723.2 (  1-) 6.2703 | 9101.32 (  1+) 3.0126 | 631.139 (  1-) 3.1750 23117.9 (  1+) 13.749 5.0-#10 @24"
   J OK   | 0.00000 (  1+) 3.6916 | 22715.3 (  1+) 7.5596 | 610.083 (  1-) 3.1750 23117.9 (  1+) 15.702 5.0-#10 @24"
----------------------------------------------------------------------------------------------------------
```

Image 5-7. RC Beam Design Summary Report

```
--------------------------------------------------------------------------------
 MIDAS/Civil - RC-Beam Design     [ AASHTO-LRFD12 ]                   Civil 2014
================================================================================

     ( ). Check moment capacity.
          -. c          =       1.1602 in.
          -. Cc         =        219.66 kips.
          -. Ts         =        221.50 kips.
          -. Mr         =      11417.95 kips-in.
          -. Mu/Mr      =   0.000  --->  O.K !

================================================================================
   [[[*]]]   ANALYZE SHEAR CAPACITY.
================================================================================

     ( ). Compute shear parameter.
          -. phi    =    0.90
          -. Av     =      6.3500 in^2.
          -. bv     =   60.00 in.
          -. dv     =   MAX[ dv, 0.9*d, 0.72*Hc ]  =  57.27 in.
          -. theta  =   44.13 Deg.    [Clause 5.8.3.4.2]
          -. beta   =    1.13

     ( ). Compute shear strength of concrete.
          -. Vu       =       639.92 kips.
          -. Vc       = 0.0316*beta*SQRT[fc']*bv*dv =      260.69 kips.
          -. phiVc    = phi * Vc =       234.62 kips.
          -. Vn_lim   = 0.25*fc'*bv*dv =      3866.06 kips.

     ( ). Compute stirrup spacing.
          -. Maximum spacing smax    = MIN[ 0.8*dv, 24 in ] =   24.000 in.
          -. Vu > phiVc   --->  Required shear reinforcement.
          -. Calculate spacing s1 = (phi*Av*fys*dv*cot(theta)) / (Vu-phiVc)  =   41.631 in.
          -. Applied spacing s      = MIN[ smax, s1 ] =   24.000 in.

     ( ). Compute shear strength of reinforcement.
          -. Vs       = Av*fys*dv*cot(theta) / s =      781.15 kips.
          -. Vs_lim   = 0.25*fc'*bv*dv - Vc =      3605.36 kips.
                      = MIN[ Vs, Vs_lim ] =      781.15 kips.
                    s = phi*Vs =      703.03 kips.
```

Image 5-6. Detailed RC Beam Design Report

B. Input Design Information

Select all the pier members.

❑ Go to Design > RC Design > Concrete Code Design > Column Design

"*** Finished Writing RC Column Design Result to Table."; this message in the message window indicates the completion of RC Column (Pier) Design. After the design is complete, a new window, "AASHTO-LRFD12 RC-Column Design Result Dialog" pops out automatically. Refer to image 5-8.

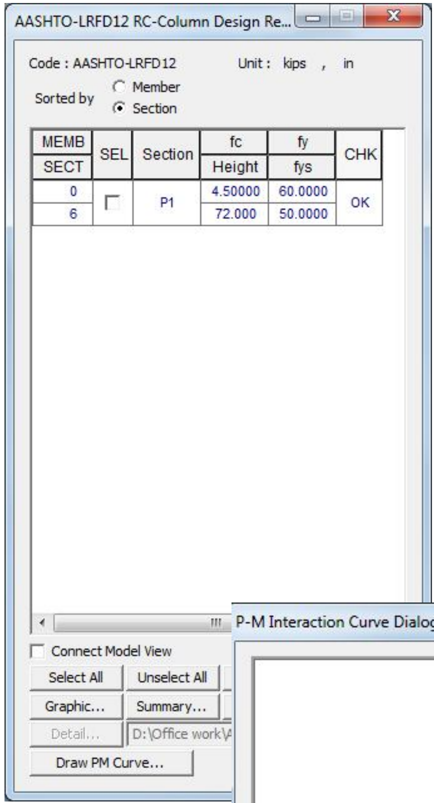


Image 5-8. RC Column Design Result Dialog Box

C. View Design Results

Note: All the Pier Design Results are OK. Refer to image 5-8

Graphic, Detail and Summary Report can be generated for RC column members similar to the RC beam members.

RC Column Design Result Dialog > Sorted by
> Select Member

Note: You can select any member and check the PM Curve for it.

For example: Select

335	☑	P1	4.50000	60.0000	OK
6			62.400	50.0000	

Click Draw PM Curve... to see the PM Curve for member 335.
Refer to image 5-9.

PM Interaction Curve Dialog > Click Close
RC Beam Design Result Dialog > Click Close

Image 5-9. PM Interaction Curve Dialog Box