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## Bridge Superstructure Design

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As the ultimate versatile, integrated tool for modeling, analysis, and design of bridge structures, CSiBridge can apply appropriate code-specific design processes to concrete box girder bridge design, design when the superstructure includes Precast Concrete Box bridges with a composite slab and steel I-beam bridges with composite slabs. The ease with which these tasks can be accomplished makes CSiBridge the most productive bridge design package in the industry.

Design using CSiBridge is based on load patterns, load cases, load combinations and design requests. The design output can then be displayed graphically and printed using a customized reporting format.

It should be noted that the design of bridge superstructure is a complex subject and the design codes cover many aspects of this process. CSiBridge is a tool to help the user with that process. Only the aspects of design documented in this manual are automated by the CSiBridge design capabilities. The user must check the results produced and address other aspects not covered by CSiBridge.

1.1 Organization

This manual is designed to help you become productive using CSiBridge design in accordance with the available codes when modeling concrete box girder
bridges and precast concrete girder bridges. Chapter 2 describes loads and load combinations. Chapter 3 describes Live Load Distribution factors. Chapter 4 describes defining the design request, which includes the design request name, a bridge object name (i.e., the bridge model), check type (i.e., the type of design), station range (i.e., portion of the bridge to be designed), design parameters (i.e., overwrites for default parameters) and demand sets (i.e., loading combinations). Chapter 5 identifies code-specific algorithms used by CSiBridge in completing concrete box girder bridges. Chapter 6 provides code-specific algorithms used by CSiBridge in completing concrete box and multicell box girder bridges. Chapter 7 describes code-specific design parameters for precast I and U girder. Chapter 8 explains how to design and optimize a steel I-beam bridge with composite slab. Chapter 9 describes how to design and optimize a steel U-beam bridge with composite slab. Chapter 10 describes how to run a Design Request using an example that applies the AASHTO LRFD 2007 code, and Chapter 11 describes design output for the example in Chapter 10, which can be presented graphically as plots, in data tables, and in reports generated using the Advanced Report Writer feature.

1.2 Recommended Reading/Practice

It is strongly recommended that you read this manual and review any applicable “Watch & Learn” Series™ tutorials, which are found on our web site, www.csiamerica.com, before attempting to design a concrete box girder or precast concrete bridge using CSiBridge. Additional information can be found in the online Help facility available from within the software’s main menu.
Chapter 2
Define Loads and Load Combinations

This chapter describes the steps that are necessary to define the loads and load combinations that the user intends to use in the design of the bridge superstructure. The user may define the load combinations manually or have CSiBridge automatically generate the code generated load combinations. The appropriate design code may be selected using the Design/Rating > Superstructure Design > Preference command.

When the code generated load combinations are going to be used, it is important for users to define the load pattern type in accordance with the applicable code. The load pattern type can be defined using the Loads > Load Patterns command. The user options for defining the load pattern types are summarized in Tables 2-1 and 2-2 for the CAN/CSA S6 code.

2.1 Load Pattern Types

Tables 2-1 and 2-2 show the permanent, transient, and exceptional load pattern types that can be defined in CSiBridge. The tables also show the CSA abbreviation and the load pattern descriptions. Users may choose any name to identify a load pattern type.
Table 2-1 PERMANENT Load Pattern Types Used in the CAN/CSA S6 Code

<table>
<thead>
<tr>
<th>CSiBridge Load Pattern Type</th>
<th>CSA</th>
<th>Description of Load Pattern</th>
</tr>
</thead>
<tbody>
<tr>
<td>CREEP</td>
<td>K</td>
<td>Force effects due to creep</td>
</tr>
<tr>
<td>DEAD</td>
<td>D</td>
<td>Dead load of structural components and non-structural attachments</td>
</tr>
<tr>
<td>HORIZ. EARTH PR</td>
<td>E</td>
<td>Horizontal earth pressures</td>
</tr>
<tr>
<td>EARTH SURCHARGE</td>
<td>E</td>
<td>Earth surcharge loads</td>
</tr>
<tr>
<td>PRESTRESS</td>
<td>P</td>
<td>Hyperstatic forces from post-tensioning</td>
</tr>
</tbody>
</table>

Table 2-2 TRANSIENT Load Pattern Types Used in the CAN/CSA S6 Design Code

<table>
<thead>
<tr>
<th>CSiBridge Load Pattern Type</th>
<th>CSA</th>
<th>Description of Load Pattern</th>
</tr>
</thead>
<tbody>
<tr>
<td>VEHICLE COLLISION</td>
<td>H</td>
<td>Vehicular collision force</td>
</tr>
<tr>
<td>VESSEL COLLISION</td>
<td>H</td>
<td>Vessel collision force</td>
</tr>
<tr>
<td>QUAKE</td>
<td>EQ</td>
<td>Earthquake</td>
</tr>
<tr>
<td>FRICTION</td>
<td>K</td>
<td>Friction effects</td>
</tr>
<tr>
<td>ICE</td>
<td>F</td>
<td>Ice loads</td>
</tr>
<tr>
<td>-</td>
<td>IM</td>
<td>Vehicle Dynamic Load Allowance</td>
</tr>
<tr>
<td>BRIDGE LL</td>
<td>L</td>
<td>Vehicular live load</td>
</tr>
<tr>
<td>SETTLEMENT</td>
<td>S</td>
<td>Force effects due settlement</td>
</tr>
<tr>
<td>TEMP GRADIENT</td>
<td>K</td>
<td>Temperature gradient loads</td>
</tr>
<tr>
<td>TEMPERATURE</td>
<td>K</td>
<td>Uniform temperature effects</td>
</tr>
<tr>
<td>STEAM FLOW</td>
<td>F</td>
<td>Water load and steam pressure</td>
</tr>
<tr>
<td>WIND–LIVE LOAD</td>
<td>V</td>
<td>Wind on live load</td>
</tr>
<tr>
<td>WIND</td>
<td>W</td>
<td>Wind loads on structure</td>
</tr>
</tbody>
</table>

2.2 Design Load Combinations

The code generated design load combinations make use of the load pattern types noted in Tables 2-1 and 2-2. Table 2-3 shows the load factors and combinations that are required in accordance with the CAN/CSA S6 code.
Chapter 2 - Define Loads and Load Combinations

Table 2-3 Load Combinations and Load Factors Used in the CAN/CSA S6 Code

<table>
<thead>
<tr>
<th>Loads</th>
<th>Permanent Loads</th>
<th>Transitory Loads</th>
<th>Exceptional Loads</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>D E P L ^1 K W V S</td>
<td>EQ F A H</td>
<td></td>
</tr>
<tr>
<td><strong>Fatigue limit state</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>FLS Combination 1</td>
<td>1.00 1.00 1.00 1.00</td>
<td>0 0 0 0 0 0</td>
<td>0 0 0 0 0</td>
</tr>
<tr>
<td><strong>Serviceability limit states</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SLS Combination 1</td>
<td>1.00 1.00 1.00 0.90 0.80 0</td>
<td>0 0 1.00</td>
<td>0 0 0 0 0</td>
</tr>
<tr>
<td>SLS Combination 2 ^2</td>
<td>0 0 0 0.90 0 0</td>
<td>0 0 0 0 0</td>
<td>0 0 0 0 0</td>
</tr>
<tr>
<td><strong>Ultimate limit states</strong> ^3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>ULS Combination 1</td>
<td>D E P</td>
<td>1.70 0 0 0 0 0</td>
<td>0 0 0 0 0</td>
</tr>
<tr>
<td>ULS Combination 2</td>
<td>D E P</td>
<td>1.60 1.15 0 0 0</td>
<td>0 0 0 0 0</td>
</tr>
<tr>
<td>ULS Combination 3</td>
<td>D E P</td>
<td>1.40 1.00 0.50 ^4 0.50 0</td>
<td>0 0 0 0 0</td>
</tr>
<tr>
<td>ULS Combination 4</td>
<td>D E P</td>
<td>0 1.25 1.65 ^4 0 0 0 0</td>
<td>0 0 0 0 0</td>
</tr>
<tr>
<td>ULS Combination 5</td>
<td>D E P</td>
<td>0 0 0 0 1.00 0 0</td>
<td>0 0 0 0 0</td>
</tr>
<tr>
<td>ULS Combination 6 ^5</td>
<td>D E P</td>
<td>0 0 0 0 0 0 1.30 0</td>
<td>0 0 0 0 0</td>
</tr>
<tr>
<td>ULS Combination 7</td>
<td>D E P</td>
<td>0 0 0 0.90 ^4 0 0</td>
<td>0 0 1.30 0</td>
</tr>
<tr>
<td>ULS Combination 8</td>
<td>D E P</td>
<td>0 0 0 0 0 0 0 1.00</td>
<td>0 0 0 0 0</td>
</tr>
<tr>
<td>ULS Combination 9</td>
<td>D E P</td>
<td>0 0 0 0 0 0 0 0</td>
<td>0 0 0 0 0</td>
</tr>
</tbody>
</table>

1. For the construction live load factor, see CSA Clause 3.16.3.
2. For superstructure vibration only.
3. For ultimate limit states, the maximum or minimum values of specified in Table CSA Table 3.2 shall be used.
4. For wind loads determined from wind tunnel tests, the load factors shall be specified in CSA Clause 3.10.5.2.
5. For long spans, it is possible that a combination of ice load F and wind load W will require investments.

Table 2-4 shows the maximum and minimum factors for the permanent loads in accordance with the CAN/CSA S6 code.

Table 2-4 Load Factors for Permanent Loads, Earth Pressure, and Hydrostatic Pressure and Prestress, αE and αP Used in the CAN/CSA S6 Code

<table>
<thead>
<tr>
<th>Dead Load</th>
<th>Maximum αD</th>
<th>Minimum αD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Factory-produced components, excluding wood</td>
<td>1.10</td>
<td>0.95</td>
</tr>
<tr>
<td>Cast-in-place concrete, wood, and all non-structural components</td>
<td>1.20</td>
<td>0.90</td>
</tr>
<tr>
<td>Wearing surfaces, based on nominal or specified thickness</td>
<td>1.50</td>
<td>0.65</td>
</tr>
<tr>
<td>Earth fill, negative skin friction on piles</td>
<td>1.25</td>
<td>0.80</td>
</tr>
<tr>
<td>Water</td>
<td>1.10</td>
<td>0.90</td>
</tr>
</tbody>
</table>
Table 2-4 Load Factors for Permanent Loads, Earth Pressure, and Hydrostatic Pressure and Prestress, $\alpha_E$ and $\alpha_P$ Used in the CAN/CSA S6 Code

<table>
<thead>
<tr>
<th>Dead load in combination with earthquakes</th>
<th>Maximum $\alpha_D$</th>
<th>Minimum $\alpha_D$</th>
</tr>
</thead>
<tbody>
<tr>
<td>All dead loads for ULS Combination 5 (see CSA Table 3.1)</td>
<td>1.25</td>
<td>0.80</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Earth pressure and hydrostatic pressure</th>
<th>Maximum $\alpha_E$</th>
<th>Minimum $\alpha_E$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Passive earth pressure, considered as a load</td>
<td>1.25</td>
<td>0.50</td>
</tr>
<tr>
<td>At-rest earth pressure</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Active earth pressure</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Backfill pressure</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hydrostatic pressure</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Prestress</th>
<th>Maximum $\alpha_P$</th>
<th>Minimum $\alpha_P$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Secondary prestress effects</td>
<td>1.05</td>
<td>0.95</td>
</tr>
</tbody>
</table>

Two combinations for each permanent load pattern are required because of the maximum and minimum factors. When the default load combinations are used, CSI-Bridge automatically creates both load combinations (one for the maximum and one for the minimum factor), and then automatically creates a third combination that represents an enveloped combination of the max/min combos.

2.3 Default Load Combinations

Default design load combinations can be activated using the Design/Rating > Load Combinations > Add Default command. Users can set the load combinations by selecting the “Bridge” option. Users may select the desired limit states and load cases using the Code Generated Load Combinations for Bridge Design form. The form shown in Figure 2-1 illustrates the options when the CAN/CSA S6 code has been selected for design.
After the desired limit states and load cases have been selected, CSiBridge will generate all of the code-required load combinations. These can be viewed using the Home > Display > Show Tables command or by using the Show/Modify button on the Define Combinations form, which is shown in Figure 2-2.
The load combinations denoted as ULS1-1, ULS1-2, and so forth refer to Ultimate I load combinations. The load case ULS1Group1 is the name given to enveloped load combination of all of the Ultimate I combinations. Enveloped load combinations will allow for some efficiency later when the bridge design requests are defined (see Chapter 4).
Chapter 3
Determine Live Load Distribution Factors

This chapter describes the algorithms used by CSiBridge to determine the live load distribution factors used to assign live load demands to individual girders. An explanation is given with respect to how the distribution factors are applied in a shear, stress, and moment check in accordance with the CAN/CSA-S6-14 code. The live load distribution factors are applicable only to superstructures that have a deck that includes multi-cell concrete box, precast I or U girders with composite slabs, or steel I girders with composite slab.

Legend:
Girder = beam + tributary area of composite slab or web +tributary area of top and bottom slab
Section Cut = all girders present in the cross-section at the cut location

3.1 Algorithm for Determining Live Load Distribution Factors (LLDF)

CSiBridge gives the user a choice of four methods to address distribution of live load to individual girders.

Method 1 – The LLD factors are specified directly by the user.

Method 2 – CSiBridge calculates the LLD factors by following procedures outlined in CAN/CSA-S6-14 Section 5.6.5.
Method 3 – CSiBridge reads the calculated live load demands directly from individual girders (available only for Area or Solid models).

Method 4 – CSiBridge distributes the live load uniformly into all girders.

It is important to note that to obtain relevant results, the definition of a Moving Load case must be adjusted depending on which method is selected.

- When the LLD factors are user specified or specified in accordance with the code (Method 1 or 2), only one lane with a MultiLane Scale Factor = 1 should be loaded into a Moving Load cases included in the demand set combinations. The vehicle classes defined in the moving load case shall comprise the truck and lane load as defined in clause 3.8.3.

- When CSiBridge reads the LLD factors directly from individual girders (Method 3, applicable to area and solid models only) or when CSiBridge applies the LLD factors uniformly (Method 4), multiple traffic lanes with relevant Multilane Scale Factors should be loaded in accordance with code requirements.

### 3.2 Determine Live Load Distribution Factors

At every section cut, the following geometric information is evaluated to determine the LLD factors.

- span length—the length of span for which moment or shear is being calculated. For more information on span length of continuous spans see section 3.3 of this manual.

- the number of girders

- girder designation—the first and last girder are designated as exterior girders and the other girders are classified as interior girders

- roadway width—measured as the distance between curbs/barriers; medians are ignored

- overhang—consists of the horizontal distance from the centerline of the exterior web of the left exterior beam at deck level to the interior edge of the curb or traffic barrier
Chapter 3 - Determine Live Load Distribution Factors

- the beams—includes the area, moment of inertia, torsion constant, center of gravity
- the thickness of the composite slab t1 and the thickness of concrete slab haunch t2
- the tributary area of the composite slab—which is bounded at the interior girder by the midway distances to neighboring girders and at the exterior girder; includes the entire overhang on one side, and is bounded by the midway distances to neighboring girder on the other side
- Young’s modulus for both the slab and the beams—angle of skew support.

CSiBridge then evaluates the parameters $F_t$ and $F_s$ in accordance with CAN/CSA-S6-14 sections 5.6.4.2 and 5.6.4.3. The center of gravity of the composite slab measured from the bottom of the beam is calculated as the sum of the beam depth, thickness of the concrete slab haunch t2, and one-half the thickness of the composite slab t1. Spacing of the girders is calculated as the average distance between the centerlines of neighboring girders.

CSiBridge then verifies that the selected LLD factors are compatible with the type of model: spine, area, or solid. If the LLD factors are read by CSiBridge directly from the individual girders, the model type must be area or solid. This is the case because with the spine model option, CSiBridge models the entire cross section as one frame element and there is no way to extract forces on individual girders. All other model types and LLDF method permutations are allowed.

3.3 Moment region

For continuous spans CSiBridge calculates the span length $L_e$ per clause 5.6.4.6. Each section cut is assigned two span lengths – one for M+ region and one for M- region based on Figure 5.1 of the code. For each demands set specified in the design request the program then determines the moment region type based on the sign of M3 caused by moving load case design type present in the demand set. The moment region type is reported in the results tables.

For non-continuous span both M+ and M- span length are set equal to true distance between span supports. Therefore the LLD factors calculated for M+ and M- regions are also equal.
3.4 Apply LLD Factors

The application of live load distribution factors varies, depending on which method has been selected: user specified; in accordance with code; directly from individual girders; or uniformly distributed onto all girders.

3.4.1 User Specified

When this method is selected, CSiBridge reads the girder designations (i.e., exterior and interior) and assigns live load distribution factors to the individual girders accordingly.

3.4.2 Calculated by CSiBridge in Accordance with Code

When this method is selected, CSiBridge considers the data input by the user for highway class, number of lanes, and modification factor for multi-lane loading.

Depending on the section type, CSiBridge validates several section parameters against requirements specified in the clause 5.6.2 of the code. When any of the parameter values are outside the range required by the code, the section cut is excluded from the Design Request.

At every section cut, CSiBridge then evaluates the live load distribution factors for moment and shear, for exterior and interior girders, and for M+ and M- regions using formulas specified in the code – clause 5.6.4.2 and clause 5.6.4.3. After evaluation, the LLDF values are assigned to individual girders based on their designation (exterior, interior). The same value equal to the average of the LLDF calculated for the left and right girders is assigned to both exterior girders. Similarly, all interior girders use the same LLDF equal to the average of the LLDF of all of the individual interior girders.

3.4.3 Forces Read Directly from Girders

When this method is selected, CSiBridge sets the live load distribution factor for all girders to 1.
3.4.4 Uniformly Distributed to Girders

When this method is selected, the live load distribution factor is equal to $1/n$ where $n$ is the number of girders in the section. All girders have identical LLD factors disregarding their designation (exterior, interior) and demand type (shear, moment).

3.5 Generate Virtual Combinations

When the method for determining the live load distribution factors is user-specified, code-specified, or uniformly distributed (Methods 1, 2 or 4), CSiBridge generates virtual load combination for every valid section cut selected for design. The virtual combinations are used during a stress check and check of the shear and moment to calculate the forces on the girders. After those forces have been calculated, the virtual combination are deleted. The process is repeated for all section cuts selected for design.

Four virtual COMBO cases for each moment region (M+ and M-) are generated for each COMBO that the user has specified in the Design Request (see Chapter 4). The program analyzes the design type of each load case present in the user specified COMBO and multiplies all non-moving load case types by $1/n$ (where $n$ is the number of girders) and the moving load case type by the section cut values of the LLD factors (exterior moment, exterior shear, interior moment and interior shear LLD factors). This ensures that dead load is shared evenly by all girders, while live load is distributed based on the LLD factors.

The program then completes a stress check and a check of the shear and the moment for each section cut selected for design.

3.5.1 Stress Check

At the Section Cut being analyzed, the girder stresses at all stress output points are read from CSiBridge for every virtual COMBO generated. To ensure that live load demands are shared equally irrespective of lane eccentricity by all girders, CSiBridge uses averaging when calculating the girder stresses. It calculates the stresses on a beam by integrating axial and M3 moment demands on all the beams in the entire section cut and dividing the demands by the number of girders. Similarly, P and M3 forces in the composite slab are integrated and
stresses are calculated in the individual tributary areas of the slab by dividing
the total slab demand by the number of girders.

When stresses are read from analysis into design, the stresses are multiplied by
\( n \) (where \( n \) is number of girders) to make up for the reduction applied in the
Virtual Combinations.

3.5.2 Shear or Moment Check
At the Section Cut being analyzed, the entire section cut forces are read from
CSiBridge for every Virtual COMBO generated. The forces are assigned to in-
dividual girders based on their designation. (For each moment region forces
from two virtual Combinations — one for shear and one for mo-
ment—generated for exterior beam are assigned to both exterior beams, and
similarly, Virtual Combinations for interior beams are assigned to interior
beams.)

3.6 Read Forces/Stresses Directly from Girders
When the method for determining the live load distribution is based on forces
read directly from the girders, the method varies based on which Design Check
has been specified in the Design Request (see Chapter 4).

3.6.1 Stress Check
At the Section Cut being analyzed, the girder stresses at all stress output points
are read from CSiBridge for every COMBO specified in the Design Request.
CSiBridge calculates the stresses on a beam by integrating axial, M3 and M2
moment demands on the beam at the center of gravity of the beam. Similarly P,
M3 and M2 demands in the composite slab are integrated at the center of gravi-
ty of the slab tributary area.

3.6.2 Shear or Moment Check
At the Section Cut being analyzed, the girder forces are read from CSiBridge
for every COMBO specified in the Design Request. CSiBridge calculates the
demands on a girder by integrating axial, M3 and M2 moment demands on the
girder at the center of gravity of the girder.
This chapter describes the Bridge Design Request, which is defined using the Design/Rating > Superstructure Design > Design Requests command.

Each Bridge Design Request is unique and specifies which bridge object is to be designed, the type of check to be performed (e.g., concrete box stress, precast composite stress, and so on), the station range (i.e., the particular zone or portion of the bridge that is to be designed), the design parameters (i.e., parameters that may be used to overwrite the default values automatically set by the program) and demand sets (i.e., the load combination(s) to be considered). Multiple Bridge Design Requests may be defined for the same bridge object.

Before defining a design request, the applicable code should be specified using the Design/Rating > Superstructure > Preferences command.

Figure 4-1 shows the Bridge Design Request form when the bridge object is for a concrete box girder bridge, and the check type is concrete box stress. Figure 4-2 shows the Bridge Design Request form when the bridge object is for a Composite I or U girder bridge and the check type is precast composite stress. Figure 4-3 shows the Bridge Design Request form when the bridge object is for a Steel I-Beam bridge and the check type is composite strength.
Figure 4-1 Bridge Design Request - Concrete Box Girder Bridges

Figure 4-2 Bridge Design Request - Composite I or U Girder Bridges

4 - 2 Name and Bridge Object
4.1 Name and Bridge Object

Each Bridge Design Request must have a unique name. Any name can be used.

If multiple Bridge Objects are used to define a bridge model, select the bridge object to be designed for the Design Request. If a bridge model contains only a single bridge object, the name of that bridge object will be the only item available from the Bridge Object drop-down list.

4.2 Check Type

The Check Type refers to the type of design to be performed and the available options depend on the type of bridge deck being modeled.

For a Concrete Box Girder bridge, the following check types are available:

- Concrete Box Stress
- Concrete Box Flexure
- **Concrete Box Shear**

For a Multi-Cell Concrete Box Girder bridge, the following check types are available:

- **Concrete Box Stress**
- **Concrete Box Flexure**
- **Concrete Box Shear**

For bridge models with *precast I or U Beams with Composite Slabs*, the following check types are available:

- **Precast Comp Stress**
- **Precast Comp Shear**
- **Precast Comp Flexure**

For bridge models with *steel I or U-beam with composite slab superstructures*, the following check types are available:

- **Steel Comp Strength**
- **Steel Comp Service**
- **Steel Comp Constructability Staged**
- **Steel Comp Constructability NonStaged**

The bold type denotes the name that appears in the check type drop-down list. A detailed description of the design algorithm can be found in Chapter 5 for concrete box girder bridges, in Chapter 6 for multi-cell box girder bridges, in Chapter 7 for precast I or U beam with composite slabs, and in Chapter 8 for steel I-beam with composite slab.
4.3 Station Range

The station range refers to the particular zone or portion of the bridge that is to be designed. The user may choose the entire length of the bridge, or specify specific zones using station ranges. Multiple zones (i.e., station ranges) may be specified as part of a single design request.

When defining a station range, the user specifies the Location Type, which determines if the superstructure forces are to be considered before or at a station point. The user may choose the location type as before the point, after the point, or both.

4.4 Design Parameters

Design parameters are overwrites that can be used to change the default values set automatically by the program. The parameters are specific to each code, deck type, and check type. Figure 4-4 shows the Superstructure Design Request Parameters form.

Table 4-1 shows the parameters for concrete box girder bridges. Table 4-2 shows the parameters for multi-cell concrete box bridges. Table 4-3 shows the parameters applicable when the superstructure has a deck that includes precast I or U
girders with composite slabs. Table 4-4 shows the parameters applicable when the superstructure has a deck that includes steel I-beams.

### Table 4-1 Design Request Parameters for Concrete Box Girders

<table>
<thead>
<tr>
<th>Concrete Box Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>Multi-Cell Concrete Box Stress Factor Compression Limit - Multiplier on $f'_c$ to calculate the compression stress limit</td>
</tr>
<tr>
<td>Multi-Cell Concrete Box Stress Factor Tension Limit - The tension limit factor may be specified using either MPa or ksi units for $f'_c$ and the resulting tension limit</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Concrete Box Shear</th>
</tr>
</thead>
<tbody>
<tr>
<td>Phi Concrete $\phi_c$ -- Resistance factor for concrete (see CSA Clause 8.4.6)</td>
</tr>
<tr>
<td>Phi PT $\phi_p$ -- Resistance factor for tendons (see CSA Clause 8.4.6)</td>
</tr>
<tr>
<td>Cracking Strength Factor -- Multiplies $\sqrt{f'_c}$ to obtain cracking strength</td>
</tr>
<tr>
<td>EpsilonX Negative Limit -- Longitudinal negative strain limit (see Clause 8.9.3.8)</td>
</tr>
<tr>
<td>EpsilonX Positive Limit -- Longitudinal positive strain limit (see Clause 8.9.3.8)</td>
</tr>
<tr>
<td>Tab slab rebar cover -- Distance from the outside face of the top slab to the centerline of the exterior closed transverse torsion reinforcement</td>
</tr>
<tr>
<td>Web rebar cover -- Distance from the outside face of the web to the centerline of the exterior closed transverse torsion reinforcement</td>
</tr>
<tr>
<td>Bottom Slab rebar cover -- Distance from the outside face of the bottoms lab to the centerline of the exterior closed transverse torsion reinforcement</td>
</tr>
<tr>
<td>Shear Rebar Material -- A previously defined rebar material label that will be used to determine the required area of transverse rebar in the girder</td>
</tr>
<tr>
<td>Longitudinal Rebar Material -- A previously defined rebar material that will be used to determine the required area of longitudinal rebar in the girder</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Concrete Box Flexure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Phi Concrete $\phi_c$ -- Resistance factor for concrete (see CSA Clause 8.4.6)</td>
</tr>
<tr>
<td>Phi PT $\phi_p$ -- Resistance factor for tendons (see CSA Clause 8.4.6)</td>
</tr>
<tr>
<td>Phi Rebar $\phi_s$ -- Resistance factor for reinforcing bars (see CSA Clause 8.4.6)</td>
</tr>
</tbody>
</table>
## Table 4-2 Design Request Parameters for Multi-Cell Concrete Box

| Multi-Cell Concrete Box Stress | Multi-Cell Concrete Box Stress Factor Compression Limit - Multiplier on $f'_c$ to calculate the compression stress limit
| Multi-Cell Concrete Box Stress Factor Tension Limit - The tension limit factor may be specified using either MPa or ksi units for $f'_c$, and the resulting tension limit |

| Multi-Cell Concrete Box Shear | Highway Class – The highway class shall be determined in accordance with CSA Clause 1.4.2.2, Table 1.1 for the average daily traffic and average daily truck traffic volumes for which the structure is designed
| Phi Concrete $\varphi_c$ -- Resistance factor for concrete (see CSA Clause 8.4.6)
| Phi PT $\varphi_p$ -- Resistance factor for tendons (see CSA Clause 8.4.6)
| Phi Rebar $\varphi_s$ -- Resistance factor for reinforcing bars (see CSA Clause 8.4.6)
| Cracking Strength Factor -- Multiplies $\sqrt{f'_c}$ to obtain cracking strength
| EpsilonX Negative Limit -- Longitudinal negative strain limit (see Clause 8.9.3.8)
| EpsilonX Positive Limit -- Longitudinal positive strain limit (see Clause 8.9.3.8)
| Shear Rebar Material -- A previously defined rebar material that will be used to determine the required area of transverse rebar in the girder
| Longitudinal Rebar Material -- A previously defined rebar material that will be used to determine the required area of longitudinal rebar in the girder |

| Multi-Cell Concrete Box Flexure | Highway Class – The highway class shall be determined in accordance with CSA Clause 1.4.2.2, Table 1.1 for the average daily traffic and average daily truck traffic volumes for which the structure is designed
| Phi Concrete $\varphi_c$ -- Resistance factor for concrete (see CSA Clause 8.4.6)
| Phi PT $\varphi_p$ -- Resistance factor for tendons (see CSA Clause 8.4.6)
<p>| Phi Rebar $\varphi_s$ -- Resistance factor for reinforcing bars (see CSA Clause 8.4.6) |</p>
<table>
<thead>
<tr>
<th>Table 4-3 Design Request Parameters for Precast I or U Beams</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Precast Comp Stress</strong></td>
</tr>
<tr>
<td>• Precast Comp Stress Factor Compression Limit - Multiplier on $f_c'$ to calculate the compression stress limit</td>
</tr>
<tr>
<td>• Precast Comp Stress Factor Tension Limit - The tension limit factor may be specified using either MPa or ksi units for $f_c'$ and the resulting tension limit</td>
</tr>
<tr>
<td><strong>Precast Comp Shear</strong></td>
</tr>
<tr>
<td>• Highway Class – The highway class shall be determined in accordance with CSA Clause 1.4.2.2, Table 1.1 for the average daily traffic and average daily truck traffic volumes for which the structure is designed</td>
</tr>
<tr>
<td>• Phi Concrete $\phi_c$ -- Resistance factor for concrete (see CSA Clause 8.4.6)</td>
</tr>
<tr>
<td>• Phi PT $\phi_p$ -- Resistance factor for tendons (see CSA Clause 8.4.6)</td>
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</tr>
<tr>
<td>• EpsilonX Negative Limit -- Longitudinal negative strain limit (see Clause 8.9.3.8)</td>
</tr>
<tr>
<td>• EpsilonX Positive Limit -- Longitudinal positive strain limit (see Clause 8.9.3.8)</td>
</tr>
<tr>
<td>• Shear Rebar Material – A previously defined rebar material label that will be used to determine the required area of transverse rebar in the girder.</td>
</tr>
<tr>
<td>• Longitudinal Rebar Material – A previously defined rebar material that will be used to determine the required area of longitudinal rebar in the girder.</td>
</tr>
<tr>
<td><strong>Precast Comp Flexure</strong></td>
</tr>
<tr>
<td>• Highway Class – The highway class shall be determined in accordance with CSA Clause 1.4.2.2, Table 1.1 for the average daily traffic and average daily truck traffic volumes for which the structure is designed</td>
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<td>• Phi Concrete $\phi_c$ -- Resistance factor for concrete (see CSA Clause 8.4.6)</td>
</tr>
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<td>• Phi PT $\phi_p$ -- Resistance factor for tendons (see CSA Clause 8.4.6)</td>
</tr>
<tr>
<td>• Phi Rebar $\phi_s$ -- Resistance factor for reinforcing bars (see CSA Clause 8.4.6)</td>
</tr>
</tbody>
</table>
4.5 Demand Sets

A demand set name is required for each load combination that is to be considered in a design request. The load combinations may be selected from a list of user defined or default load combinations that are program determined (see Chapter 2).

4.6 Live Load Distribution Factors

When the superstructure has a deck that includes precast I or U girders with composite slabs or multi-cell boxes, Live Load Distribution factors can be specified. LLD factors are described in Chapter 3.
Chapter 5
Design Concrete Box Girder Bridges

This chapter describes the algorithms applied in accordance with the CAN/CSA-S6-14 code for design and stress check of the superstructure of a concrete box type bridge deck section.

In CSiBridge, when distributing loads for concrete box design, the section is always treated as one beam, all load demands (permanent and transient) are distributed evenly to the webs for stress and flexure and proportionally to the slope of the web for shear. Torsion effects are always considered and assigned to the outer webs and the top and bottom slab.

With respect to shear and torsion check per Clause 8.9 of the code torsion is considered.

5.1 Stress Design

The following design parameters are defined by the user in the design request:

- \( \text{FactorCompLim} \) \( f'_c \) multiplier; Default Value = 0.6. The \( f'_c \) is multiplied by the \( \text{FactorCompLim} \) to obtain compression limit

- \( \text{FactorTensLim} \) \( \sqrt{f'_c} \) multiplier; Default Value = 0.4(MPa); The \( \sqrt{f'_c} \) is multiplied by the \( \text{FactorTensLim} \) to obtain tension limit
The stresses are evaluated at three points at the top fiber of the top slab and three points at the bottom fiber of the bottom slab: the left corner, the centerline web and the right corner of the relevant slab tributary area. The location is labeled in the output plots and tables.

Concrete strength $f'_c$ is read at every point, and compression and tension limits are evaluated using the $\text{FactorCompLim} - f'_c$ multiplier and $\text{FactorTensLim} - \sqrt{f'_c}$ multiplier.

The stresses are evaluated for each demand set. If the demand set contains live load, the program positions the load to capture extreme stress at each of the evaluation points.

Extremes are found for each point and the controlling demand set name is recorded.

The stress limits are evaluated by applying the preceding parameters.

## 5.2 Flexure Design

The following design parameters are defined by the user in the design request:

- $\Phi_C$ – Resistance Factor for concrete; Default Value = 0.75.
- $\Phi_P$ – Resistance Factor for prestressing strands; Default Value = 0.95
- $\Phi_S$ – Resistance Factor for reinforcing bars; Default Value = 0.90

### 5.2.1 Variables

- $M_r$ Factored flexural resistance
- $t_{slab eq}$ Thickness of top slab
- $b_{slab}$ Effective slab width
- $b_v$ Thickness of web
- $A_{slab}$ Effective area of slab
5.2.2 Design Process

The derivation of the moment resistance of the section is based on approximate stress distribution specified in Article 8.8.3. The natural relationship between concrete stress and strain is considered satisfied by an equivalent rectangular concrete compressive stress block of $\alpha_1 \phi' f'_c$ over a zone bounded by the edges of the cross-section and a straight line located parallel to the neutral axis at the distance $a = \beta_1 c$ from the extreme compression fiber. The distance $c$ is measured perpendicular to the neutral axis. The factor $\beta_1$ is taken as 0.97-0.0025$f'_c$ except that $\beta_1$ is not to be taken to be less than 0.67.
The flexural resistance is determined in accordance with Clause 8.8.3. The resistance is evaluated only for bending about horizontal axis 3. Separate capacity is calculated for positive and negative moment. The capacity is based on bonded tendons and mild steel located in tension zone as defined in the Bridge Object. Tendons and mild steel reinforcement located in compression zone are not considered. It is assumed that all defined tendons in a section, stressed or not, have $f_{pe}$ (effective stress after loses) larger than $0.5 \times f_{pu}$ (specified tensile strength). If a certain tendon should not be considered for the flexural capacity calculation, its area must be set to zero.

The section properties are calculated for the section before skew, grade, and superelevation are applied. This is consistent with the demands being reported in the section local axis. The effective width of the flange (slab) in compression is evaluated per Clause 5.8.2.1.

### 5.2.3 Algorithms

At each section:

- All section properties and demands are converted from CSiBridge model units to N, mm.
- The equivalent slab thickness is evaluated based on the tributary slab area and the slab width assuming a rectangular shape.

$$t_{sla} = \frac{A_{slab}}{b_{slab}}$$

- $\alpha_1$ and $\beta_1$ stress block factors are evaluated in accordance with 8.8.3 based on section $f'_{c}$

$\alpha_1 = 0.85 - 0.0015f'_{c} \geq 0.67$

$\beta_1 = 0.97 - 0.0025f'_{c} \geq 0.67$

- The tendon and rebar location, area, and material are read. Only bonded tendons are processed; unbonded tendons are ignored.
Tendons and rebars are split into two groups depending on the sign of moment they resist—negative or positive. A tendon or rebar is considered to resist a positive moment when it is located outside of the top fiber compression stress block and is considered to resist a negative moment when it is located outside of the bottom fiber compression stress block. The compression stress block extends over a zone bounded by the edges of the cross-section and a straight line located parallel to the neutral axis at the distance $a = \beta c$ from the extreme compression fiber. The distance $c$ is measured perpendicular to the neutral axis.

For each tendon group, an area weighted average of the following values is determined:

- sum of tendon areas $A_{ps}$
- distance from center of gravity of tendons $d_P$ to extreme compression fiber
- specified tensile strength of prestressing steel $f_{pu}$
- constant $k$

$$k = 2 \left( 1.04 - \frac{f_{py}}{f_{pu}} \right)$$

For each rebar group the following values are determined:

- sum of tension rebar areas $A_s$
- Tension rebars $d_s P$

- Moment resistance – first it is assumed that the equivalent compression stress block is within the top slab. Distance $c$ between the neutral axis and the compressive face is calculated in accordance with Clause C8.8.4.1.

$$c = \frac{\varphi_P A_{ps} f_{pu} + \varphi_s A_s f_s}{\alpha_1 \varphi_c f'_c \beta_1 b_{slab} + k A_{ps} f_{pu} \frac{f_{pu}}{d_p}}$$

The distance $c$ is compared to the equivalent slab thickness to determine if the section is a T-section or rectangular section.
If $c \beta > t_{slabeq}$, the section is a T-section.

- If the section is a T-section, the distance $c$ is recalculated in accordance with Clause C8.8.4.1.

$$c = \frac{\varphi_P A_{PS} f_{pu} + A_s f_s - \alpha_1 \varphi c f' c (b_{slab} - b_{webeq}) t_{slabeq}}{\alpha_1 \varphi c f' c \beta_1 b_{webeq} + k A_{PS} \frac{f_{pu}}{d_p}}$$

- Average stress in prestressing steel $f_{ps}$ is calculated in accordance with Clause 8.8.4.2

$$f_{ps} = f_{pu} (1 - k \frac{c}{d_p})$$

- Factored flexural resistance $M_r$ is calculated in accordance with Clause C8.8.4.1.

If the section is a T-section, then

$$M_r = \varphi_p A_{PS} f_{ps} \left( d_p - \frac{c \beta_1}{2} \right) + \varphi_s A_s f_y \left( d_s - \frac{c \beta_1}{2} \right) + \alpha_1 \varphi c f' c (b_{slab} - b_{webeq}) t_{slabeq} \left( \frac{c \beta_1}{2} - \frac{t_{slabeq}}{2} \right)$$

else

$$M_r = \varphi_p A_{PS} f_{ps} \left( d_p - \frac{c \beta_1}{2} \right) + \varphi_s A_s f_y \left( d_s - \frac{c \beta_1}{2} \right)$$

- Extreme moment $M_3$ demands are found from the specified demand sets and the controlling demand set name is recorded.

The process for evaluating negative moment resistance is analogous.

### 5.3 Shear Design

The following design parameters are defined by the user in the design request:

- $PhiC$ – Resistance Factor for concrete; Default Value = 0.75.
- $PhP$ – Resistance Factor for prestressing strands; Default Value = 0.95
− *PhiS* – Resistance Factor for reinforcing bars; Default Value = 0.90

− *FactRupture* - multiplies sqrt f’c [MPa] to obtain cracking strength; Default Value = 0.40

− *EpsXLimNeg* – limit on minimum longitudinal strain per Clause 8.9.3.8, Default Value = -0.2x10^-3

− *EpsXLimPos* – limit on maximum longitudinal strain per Clause 8.9.3.8, Default Value = 3.0x10^-3

− *CoverTop* – distance from the outside face of the top slab to the centerline of exterior closed transverse torsion reinforcement, Default Value = 50mm

− *CoverWeb* – distance from the outside face of the web to the centerline of exterior closed transverse torsion reinforcement, Default Value = 50mm

− *CoverBot* – distance from the outside face of the bottom slab to the centerline of exterior closed transverse torsion reinforcement, Default Value = 50mm

− *Shear Rebar Material* – A previously defined rebar material label that will be used to determine the required area of transverse rebar in the girder.

− *Longitudinal Rebar Material* - A previously defined rebar material label that will be used to determine the required area of longitudinal rebar in the girder.

### 5.3.1 Variables

\( V_{fsec} \) \hspace{1cm} \text{Factored shear demand per section cut excluding force in tendons}

\( V_f \) \hspace{1cm} \text{Factored shear demand per web cut excluding force in tendons}

\( N_f \) \hspace{1cm} \text{Applied factored axial force per section cut, taken as positive if tensile}

\( M_{fsec} \) \hspace{1cm} \text{Factored flexural moment demand per section cut}

\( M_f \) \hspace{1cm} \text{Factored flexural moment demand per web}
$T_f$  Factored torsional moment per section cut

$V_{2c}$  Shear in section cut excluding force in tendons

$V_{2\text{Tot}}$  Shear in section cut including force in tendons

$V_p$  Component in the direction of the applied shear of the effective pre-stressing force; if $V_p$ has the same sign as $V_f$, the component is resisting the applied shear

$d_v$  Effective shear depth in accordance with 8.9.1.5 of the code.

$d_{\text{girder}}$  Depth of girder

$b$  Minimum web width

$b_v$  Effective web width adjusted for presence of prestressing ducts in accordance with Section 8.9.1.6 of the code

$d_{PT\text{Top}}$  Distance from bottom fiber to center of prestressing steel near the top fiber

$d_{PT\text{Bot}}$  Distance from top fiber to center of prestressing steel near the bottom fiber

$A_{ps}$  Area of prestressing steel on the flexural tension side of the member

$f_{pu}$  Specified tensile strength of prestressing steel

$E_p$  Prestressing steel Young’s modulus

$A_{\text{vltens}}$  Area of non-prestressed steel on the flexural tension side of the member at the section under consideration

$A_{\text{vlncomp}}$  Area of non-prestressed steel on the flexural compression side of the member at the section under consideration

$E_s$  Reinforcement Young’s modulus
Longitudinal strain per Clause 8.9.3.8 of the code

\( \varepsilon_x \)

Max and min value of longitudinal strain as specified by the user in the Design Parameters

\( \varepsilon_{x, \text{LimMin}}, \varepsilon_{x, \text{LimMax}} \)

Young’s modulus of concrete

\( E_c \)

Area of concrete on the flexural tension side of the member

\( A_{CT} \)

Area of transverse shear reinforcement per unit length

\( A_{TS} \)

Minimum area of transverse shear reinforcement per unit length in accordance with Clause 8.9.1.3 of the code

\( A_{TS, \text{min}} \)

Area of required closed transverse torsion reinforcement per unit length per Clause 8.9.3.17

\( A_t \)

Area enclosed by the outside perimeter of a concrete cross-section, including the area of holes, if any.

\( A_{cp} \)

Area enclosed by the centerline of exterior closed transverse torsion reinforcement, including the area of voids, if any.

\( A_{oh} \)

Taken as 0.85 \( A_{oh} \) per Clause 8.9.3.17

\( p_c \)

Outside perimeter of a concrete section

\( p_h \)

Perimeter of closed transverse torsion reinforcement measured along its centerline

### 5.3.2 Design Process

The shear resistance is determined in accordance with paragraph 8.9.3.of the code (sectional design model derived from Modified Compression Field Theory). The procedure assumes that the concrete shear stresses are distributed uniformly over an area \( b_v \) wide and \( d_v \) deep, that the direction of principal compressive stresses (defined by angle \( \theta \)) remains constant over \( d_v \), and that the shear strength of the section can be determined by considering the biaxial stress conditions at just one location in the web. For design, the user should select only those sections that comply with these assumptions by defining appropriate station ranges in the Design Request (see Chapter 4).
The effective web width is taken as the minimum web width, measured parallel to the neutral axis, between the resultants of the tensile and compressive forces as a result of flexure. In determining the effective web width at a particular level, one-quarter the diameter of grouted ducts at that level is subtracted from the web width.

All defined tendons in a section, stressed or not, are assumed to be grouted. Each tendon at a section is checked for presence in the web and the minimum controlling effective web thicknesses are evaluated.

The tendon duct is considered as having effect on the web effective thickness even if only part of the duct is within the web boundaries. In such cases, the entire one-quarter of the tendon duct diameter is subtracted from the element thickness.

If several tendon ducts overlap in one web (when projected on the vertical axis), the diameters of the ducts are added for the sake of evaluation of the effective thickness. The effective web thickness is calculated at the top and bottom of each duct.

The Shear and Torsion Design is completed on per web basis. The D/C ratio is calculated and the required area of rebar is reported for each web. The section design shear force is distributed into individual webs assuming that the vertical shear that is carried by a web decreases with increased inclination of the web from vertical. Section torsion moments are assigned to external webs and slabs.

The rebar area and ratio are calculated using measurements normal to the web. Thus, vertical shear forces are divided by \( \cos(\text{alpha}_\text{web}) \). The rebar area calculated is the actual, normal cross-section of the bars. The rebar ratio is calculated using the normal width of the web, \( t_{\text{web}} = b_{\text{web}} \times \cos(\text{alpha}_\text{web}) \).

### 5.3.3 Algorithm

- All section properties and demands are converted from CSiBridge model units to N, mm.
- For every COMBO specified in the Design Request that contains envelopes, a new force demand set is generated. The new force demand set is built up from the maximum tension values of \( P \) and the maximum absolute values of \( V2 \) and \( M3 \) of the two StepTypes (Max and Min) present in the envelope.
COMBO case. The StepType of this new force demand set is named ABS and the signs of the P, V2 and M3 are preserved. The ABS case follows the industry practice where sections are designed for extreme shear and moments that are not necessarily corresponding to the same design vehicle position. The section cut is designed for all three StepTypes in the COMBO—Max, Min and ABS—and the controlling StepType is reported.

- In cases where the demand moment \( M_{fsec} < |V_{fsec} - V_p|d_v \) two new force demand sets are generated as follows:
  \[
  M_{fpos} = |V_{fsec} - V_p|d_{vpos} \\
  M_{fneg} = -|V_{fsec} - V_p|d_{vneg}.
  \]
  The acronyms “-CodeMinMuPos” and “-CodeMinMuNeg” are added to the end of the StepType name. The signs of the P and V2 are preserved.

- On the basis of the location and inclination of each web, the per-web demand values are evaluated

<table>
<thead>
<tr>
<th>Location</th>
<th>Outer Web</th>
<th>Inner Web</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear and Torsion Check</td>
<td>( \frac{V_{fsec} \kappa_{web}}{\cos \alpha_{web}} )</td>
<td>( \frac{V_{fsec} \kappa_{web}}{\cos \alpha_{web}} )</td>
</tr>
</tbody>
</table>

where \( \kappa_{web} = \frac{\cos(\mid \alpha_{web} \mid)}{\sum_{i=1}^{n_{web}} \cos(\mid \alpha_{web} \mid)} \)

- The component in the direction of the applied shear of the effective prestressing force, positive if resisting the applied shear, is evaluated:
  \[
  V_p = \frac{(V_2c - V_{2to}) \kappa_{web}}{\cos \alpha_{web}}
  \]

- Demand moment per web is calculated as
  \[
  M_f = \frac{M_{fsec}}{n_{web}}
  \]

- Effective shear depth \( d_v \) is evaluated per Clause 8.9.1.5.

If \( M_f > 0 \), then \( d_v = \max(0.72d_{girder}, 0.9d_{PTBo}) \)
If $M_u < 0$, then $d_v = \max(0.72d_{\text{girder}}, 0.9d_{\text{PTTop}})$

- The shear demand/capacity ratio (D/C) is calculated based on the maximum permissible shear capacity at a section in accordance with Section 8.9.3.3 of the code

$$\text{Shear} \frac{D}{C} = \frac{|V_f - V_p|}{0.25\phi_c f'_c b_v d_v}$$

- The combined shear and torsion demand/capacity ratio (D/C) is calculated based on web effective width to avoid crushing in accordance with Section 8.9.3.18 of the code

$$\text{Shear and Torsion} \frac{D}{C} = \frac{|V_f - V_p| + \frac{T_f p_h}{1.7A_{oh}^2}}{0.25\phi_c f'_c}$$

If the effective web thickness of the box section is less than $A_{oh}/ph$, the second term in this expression is replaced by $\frac{T_f}{1.7A_{oh}t}$, where $t$ is the minimum effective thickness of web

- The torsion demand/capacity ratio (D/C) is calculated based on slab effective thickness to avoid crushing in accordance with Section 8.9.3.18 of the code

$$\text{Torsion} \frac{D}{C} = \frac{\frac{T_f p_h}{1.7A_{oh}^2}}{0.25\phi_c f'_c}$$

If the minimum of the top or bottom slab effective thickness is less than $A_{oh}/ph$, the second term in this expression is replaced by $\frac{T_f}{1.7A_{oh}t}$, where $t$ is the minimum effective thickness of top or bottom slab

The maximum value of the D/C for Shear and Torsion at webs and Torsion at slabs is reported in the result table in a column labeled “TorDCRatio”

- Evaluate numerator and denominator of $\varepsilon_x$ (Clause 8.9.3.19)

$$\varepsilon_{\text{numerator}} = \frac{|M_f|}{d_v} + \sqrt{(V_f - V_p)^2 + \left(\frac{0.9p_h T_f}{2A_o}\right)^2 + 0.5N_f - A_{ps}0.7f_{pu}}$$
Adjust denominator values as follows

If $\varepsilon_{x denominator} = 0$ and $\varepsilon_{x numerator} \geq 0$ then $\varepsilon_x = \varepsilon_{x lim pos}$ and

If $\varepsilon_{x numerator} < 0$ then $\varepsilon_{x denominator} = 2(E_p A_{ps} + E_c A_{ct})$

Evaluate (eq. 5.8.3.4.2-4)

$$\varepsilon_x = \frac{\varepsilon_{x numerator}}{\varepsilon_{x denominator}}$$

Check if axial tension is large enough to crack the flexural compression face of the section.

If $\frac{N_f}{A_{girder}} > 0.4\sqrt{f_c'}$ then $\varepsilon_x = 2\varepsilon_x$

Check against the limit on the longitudinal strain specified in the Design Request

$$\varepsilon_x = \max(\varepsilon_x, \varepsilon_{x lim neg}) \text{ and } \varepsilon_x = \min(\varepsilon_x, \varepsilon_{x lim pos})$$

Evaluate the angle $\theta$ of inclination of diagonal compressive stresses as determined in Clause 8.9.3.7.

$$18^\circ \leq 29^\circ + 7000\varepsilon_x \leq 45^\circ$$

Evaluate the factor indicating the ability of diagonally cracked concrete to transmit tension and shear, as specified in Clause 8.9.3.7

$$\beta = \frac{0.4}{1 + 1500\varepsilon_x} \leq 0.525$$

Evaluate the nominal shear resistance provided by tensile stresses in the concrete per Clause 8.9.3.4.

$$V_c = 2.5\beta \phi f_{ct} b_v d_v \text{ where } f_{ct} < 3.2 \text{ MPa}$$
Evaluate how much shear demand is left to be carried by rebar and determine amount of required transverse reinforcement per unit of length – Clause 8.9.3.5.

\[ V_s = |V_f - V_p| - V_c \]

if \( V_s < 0 \), then \( A_{vs} = 0 \) else

\[ A_{vs} = \frac{V_s}{\phi_s f_y d_v \frac{1}{\tan \theta}} \]

Check against minimum transverse shear reinforcement per Clause 8.9.1.2 and 8.9.1.3.

if \( |V_f| > 0.2 \phi_c f_{cr} b_v d_v + 0.5 \phi_p |V_p| \) then

\[ A_{vsmin} = \frac{0.15 f_{cr} b_v}{f_y} \]

else \( A_{vsmin} = 0 \).

If \( V_s < 0 \), then \( A_{vs} = A_{vsmin} \) else \( A_{vs} = \max(A_{vsmin}, A_{vs}) \).

Recalculate \( V_s \) in accordance with Clause 8.9.3.5.

\[ V_s = \phi_s f_y A_{vs} d_v \frac{1}{\tan \theta} \]

Evaluate if torsion needs to be considered per Clause 8.9.1.1

\[ T_{cr} = 0.80 \phi_c f_{cr} \frac{A_{cp}^2}{P_c} \left[ 1 + \frac{f_{ce}}{0.80 \phi_c f_{cr}} \right]^{0.5} \]

where

\[ f_{ce} = \frac{N_f}{A_{box}} \] (\( N_f \) taken as positive when in compression)

Evaluate the longitudinal rebar on the flexure tension side in accordance with Clause 8.9.3.11 where \( V_s \) is not taken greater then \( V_f \).

If \( T_f < 0.25 T_{cr} \) then
Chapter 5 - Design Concrete Box Girder Bridges

\[ A_{v\text{ltens}} = \frac{|M_f|}{d_v} + 0.5N_f + \left( |V_u - V_p| - 0.5V_s \right) \frac{1}{\phi_s f_y} \tan \theta \]

else \n
\[ A_{v\text{ltens}} = \frac{|M_f|}{d_v} + 0.5N_f + \sqrt{\left( |V_u - V_p| - 0.5V_s \right)^2 + \left( \frac{0.45 p_h T_f}{2A_o} \right)^2} \frac{1}{\phi_s f_y} \tan \theta \]

- Evaluate the longitudinal rebar on the flexure compression side in accordance with Clause 8.9.3.12 where Vs is not taken greater than Vf.

If \( T_f < 0.25T_{cr} \) then \n
\[ A_{v\text{comp}} = \frac{0.5N_f + \left( |V_u - V_p| - 0.5V_s \right) \frac{1}{\phi_s f_y} - \frac{|M_f|}{d_v}}{d_v} \]

else \n
\[ A_{v\text{comp}} = \frac{0.5N_f + \sqrt{\left( |V_u - V_p| - 0.5V_s \right)^2 + \left( \frac{0.45 p_h T_f}{2A_o} \right)^2} \frac{1}{\phi_s f_y} - \frac{|M_f|}{d_v}}{d_v} \]

- Assign longitudinal rebar to the top or bottom side of the girder based on the moment sign.

If \( M_f < 0 \) then \( A_{v\text{htop}} = A_{v\text{ltens}} \) and \( A_{v\text{hbot}} = A_{v\text{lcomp}} \)

else \( A_{v\text{htop}} = A_{v\text{lcomp}} \) and \( A_{v\text{hbot}} = A_{v\text{ltens}} \)

- If \( T_f > 0.25T_{cr} \) then calculate required torsion rebar per unit length \n
\[ A_t = \frac{|T_f|}{2A_o \phi_s f_y \cot \theta} \]
This chapter describes the algorithms applied in accordance with the CAN/CSA-S6-14 code for design and stress checks when the superstructure has a deck that includes cast-in-place multi-cell concrete box and uses the Simplified Method of Analysis, as described in Section 5.6.4 of the code.

For MulticellConcBox design in CSiBridge each web and its tributary slabs are designed separately. Moments and shears due to live load are distributed to individual webs in accordance with the factors specified in Clauses 5.6.4.2 of the code. When CSiBridge calculates the Live Load Distribution Factors (LLDFs), the section and span qualification criteria stated in CAN/CSA-S6-14 5.6.2 are verified and non-compliant sections are not designed. The program assumes that the cross section satisfies requirements of clause 5.6.2 (j) of the code and treats the multi-cell box girders as voided slabs for the purposes of simplified methods of analysis. It is the designers responsibility to provide adequate diaphragms or cross frames to satisfy requirements of the clause 5.5.9(b)(ii).

With respect to shear and torsion check per Clause 8.9 of the code, torsion is ignored.
### 6.1 Stress Design

The following design parameters are defined by the user in the design request:

- \( \text{FactorCompLim} - f'_c \) multiplier; Default Value = 0.6. The \( f'_c \) is multiplied by the \( \text{FactorCompLim} \) to obtain compression limit
- \( \text{FactorTensLim} - \sqrt{f'_c} \) multiplier; Default Value = 0.4(MPa); The \( \sqrt{f'_c} \) is multiplied by the \( \text{FactorTensLim} \) to obtain tension limit

The stresses are evaluated at three points at the top fiber of the top slab and three points at the bottom fiber of the bottom slab: the left corner, the centerline web and the right corner of the relevant slab tributary area. The location is labeled in the output plots and tables.

Concrete strength \( f'_c \) is read at every point, and compression and tension limits are evaluated using the \( \text{FactorCompLim} - f'_c \) multiplier and \( \text{FactorTensLim} - \sqrt{f'_c} \) multiplier.

The stresses assume linear distribution and take into account axial (P) and either both bending moments (M2 and M3) or only P and M3, depending on which method for determining LLDF has been specified in the design request (see Chapters 3 and 4).

The stresses are evaluated for each demand set. Extremes are found for each point and the controlling demand set name is recorded.

The stress limits are evaluated by applying the preceding parameters.

### 6.2 Shear Design

The following design parameters are defined by the user in the design request:

- \( \text{Highway Class} \) – Highway Class per clause 1.4.2.2; Default Value = A, Typical value(s): A,B,C,D. The classification is used to determine \( F \) and \( C_I \) factors
- \( \text{PhiC} \) – Resistance Factor for concrete; Default Value = 0.75.
• \(PhP\) – Resistance Factor for prestressing strands; Default Value = 0.95
• \(PhiS\) – Resistance Factor for reinforcing bars; Default Value = 0.90
• \(FactRupture\) - multiplies \(\sqrt{f_c}\) [MPa] to obtain cracking strength; Default Value = 0.40
• EpsXLimNeg – limit on minimum longitudinal strain per Clause 8.9.3.8, Default Value = -0.2x10^{-3}
• EpsXLimPos – limit on maximum longitudinal strain per Clause 8.9.3.8, Default Value = 3.0x10^{-3}
• Shear Rebar Material – A previously defined rebar material label that will be used to determine the required area of transverse rebar in the girder.
• Longitudinal Rebar Material - A previously defined rebar material label that will be used to determine the required area of longitudinal rebar in the girder.

### 6.2.1 Variables

- \(V_f\) Factored shear demand per girder excluding force in tendons
- \(N_f\) Applied factored axial force, taken as positive if tensile
- \(M_f\) Factored moment at the section
- \(V_{2c}\) Shear in Section Cut excluding force in tendons
- \(V_{2\text{Tot}}\) Shear in Section Cut including force in tendons
- \(V_p\) Component in the direction of the applied shear of the effective prestressing force; if \(V_p\) has the same sign as \(V_f\), the component is resisting the applied shear
- \(d_v\) Effective shear depth in accordance with 8.9.1.5 of the code.
- \(d_{\text{girder}}\) Depth of girder
- \(b\) Minimum web width
$b_v$  Effective web width adjusted for presence of prestressing ducts in accordance with Section 8.9.1.6 of the code

d$PT_{Top}$  Distance from bottom fiber to center of prestressing steel near the top fiber

d$PT_{Bot}$  Distance from top fiber to center of prestressing steel near the bottom fiber

$A_{ps}$  Area of prestressing steel on the flexural tension side of the member

$f_{pu}$  Specified tensile strength of prestressing steel

$E_p$  Prestressing steel Young’s modulus

$A_{vl\text{tens}}$  Area of non-prestressed steel on the flexural tension side of the member at the section under consideration

$A_{vl\text{comp}}$  Area of non-prestressed steel on the flexural compression side of the member at the section under consideration

$E_s$  Reinforcement Young’s modulus

$\varepsilon_x$  Longitudinal strain per Clause 8.9.3.8 of the code

$\varepsilon_{xLimMin}, \varepsilon_{xLimMax}$  Max and min value of longitudinal strain as specified by the user in the Design Parameters

$E_c$  Young’s modulus of concrete

$A_{CT}$  Area of concrete on the flexural tension side of the member

$A_{VS}$  Area of transverse shear reinforcement per unit length

$A_{VS\text{min}}$  Minimum area of transverse shear reinforcement per unit length in accordance with Clause 8.9.1.3 of the code
6.2.2 Design Process

The shear resistance is determined in accordance with paragraph 8.9.3 of the code (sectional design model derived from Modified Compression Field Theory). The procedure assumes that the concrete shear stresses are distributed uniformly over an area $b_v$ wide and $d_v$ deep, that the direction of principal compressive stresses (defined by angle $\theta$) remains constant over $d_v$, and that the shear strength of the section can be determined by considering the biaxial stress conditions at just one location in the web. For design, the user should select only those sections that comply with these assumptions by defining appropriate station ranges in the Design Request (see Chapter 4).

The effective web width is taken as the minimum web width, measured parallel to the neutral axis, between the resultants of the tensile and compressive forces as a result of flexure. In determining the effective web width at a particular level, one-quarter the diameter of grouted ducts at that level is subtracted from the web width.

All defined tendons in a section, stressed or not, are assumed to be grouted. Each tendon at a section is checked for presence in the web and the minimum controlling effective web thicknesses are evaluated.

The tendon duct is considered as having effect on the web effective thickness even if only part of the duct is within the web boundaries. In such cases, the entire one-quarter of the tendon duct diameter is subtracted from the element thickness.

If several tendon ducts overlap in one web (when projected on the vertical axis), the diameters of the ducts are added for the sake of evaluation of the effective thickness. The effective web thickness is calculated at the top and bottom of each duct.

Shear design is completed on a per-web basis. Please refer to Chapter 3 for a description of the live load distribution to individual girders.

6.2.3 Algorithms

- All section properties and demands are converted from CSiBridge model units to N, mm.
For every COMBO specified in the Design Request that contains envelopes, a new force demand set is generated. The new force demand set is built up from the maximum tension values of P and the maximum absolute values of $V_2$ and $M_3$ of the two StepTypes (Max and Min) present in the envelope COMBO case. The StepType of this new force demand set is named ABS and the signs of the P, $V_2$ and $M_3$ are preserved. The ABS case follows the industry practice where sections are designed for extreme shear and moments that are not necessarily corresponding to the same design vehicle position. The section cut is designed for all three StepTypes in the COMBO—Max, Min and ABS—and the controlling StepType is reported.

In cases where the demand moment $M_f < |V_f - V_p|d_p$, two new force demand sets are generated as follows:

$$M_f^{pos} = |V_f - V_p|d_{vp}^{pos}$$
$$M_f^{neg} = -|V_f - V_p|d_{vneg}.$$

The acronyms “-CodeMinMuPos” and “-CodeMinMuNeg” are added to the end of the StepType name. The signs of the P and $V_2$ are preserved.

The component in the direction of the applied shear of the effective pre-stressing force, positive if resisting the applied shear, is evaluated:

$$V_p = \frac{V_{2c} - V_{2Tot}}{n_{girders}}$$

Effective shear depth $d_v$ is evaluated per Clause 8.9.1.5.

If $M_f > 0$, then $d_v = \max(0.72d_{girder}, 0.9d_{PTBot})$
If $M_f < 0$, then $d_v = \max(0.72d_{girder}, 0.9d_{PTTop})$

The demand/capacity ratio (D/C) is calculated based on the maximum permissible shear capacity at a section in accordance with Section 8.9.3.3 of the code

$$\frac{D}{C} = \frac{|V_f - V_p|}{0.25\phi c f'_c b_v d_p}$$

Evaluate numerator and denominator of $\varepsilon_c$ (Clause 8.9.3.8)
\[ \varepsilon_{\text{numerator}} = \left| \frac{M_f}{d_v} \right| + |V_f - V_p| + 0.5N_f - A_p s_0.7 f_{pu} \]

\[ \varepsilon_{\text{denominator}} = 2(E_p A_p s) \]

- Adjust denominator values as follows
  
  If \( \varepsilon_{\text{denominator}} = 0 \) and \( \varepsilon_{\text{numerator}} \geq 0 \) then \( \varepsilon_x = \varepsilon_{\text{lim pos}} \) and
  
  If \( \varepsilon_{\text{numerator}} < 0 \) then \( \varepsilon_{\text{denominator}} = 2(E_p A_p s + E_c A_{ct}) \)

- Evaluate (eq. 5.8.3.4.2-4)

\[ \varepsilon_x = \frac{\varepsilon_{\text{numerator}}}{\varepsilon_{\text{denominator}}} \]

- Check if axial tension is large enough to crack the flexural compression face of the section.

If \( \frac{N_f}{A_{\text{girder}}} > 0.4 \sqrt{f'_c} \) then \( \varepsilon_x = 2\varepsilon_x \)

- Check against the limit on the longitudinal strain specified in the Design Request

\[ \varepsilon_x = \max(\varepsilon_x, \varepsilon_{\text{limit neg}}) \text{ and } \varepsilon_x = \min(\varepsilon_x, \varepsilon_{\text{limit pos}}) \]

- Evaluate the angle \( \theta \) of inclination of diagonal compressive stresses as determined in Clause 8.9.3.7.

\[ 18^\circ \leq 29^\circ + 7000\varepsilon_x \leq 45^\circ \]

- Evaluate the factor indicating the ability of diagonally cracked concrete to transmit tension and shear, as specified in Clause 8.9.3.7

\[ \beta = \frac{0.4}{1 + 1500\varepsilon_x} \leq 0.525 \]

- Evaluate the nominal shear resistance provided by tensile stresses in the concrete per Clause 8.9.3.4.

\[ V_c = 2.5\beta \Phi_c f_{cr} b_v d_v \text{ where } f_{cr} < 3.2 \text{ MPa} \]
- Evaluate how much shear demand is left to be carried by rebar and determine amount of required transverse reinforcement per unit of length – Clause 8.9.3.5.

\[
V_s = |V_f - V_p| - V_c
\]

\[
if \ V_s < 0, \ then \ A_{vs} = 0 \ else
\]

\[
A_{vs} = \frac{V_s}{\phi_s f_y d_v \frac{1}{\tan \theta}}
\]

- Check against minimum transverse shear reinforcement per Clause 8.9.1.2 and 8.9.1.3.

\[
if \ |V_f| > 0.2 \phi_c f_r b_v d_v + 0.5 \phi_p |V_p| \ then
\]

\[
A_{vs\text{min}} = \frac{0.15 f_r b_v}{f_y}
\]

else \( A_{vs\text{min}} = 0 \).

If \( V_s < 0 \), then \( A_{vs} = A_{vs\text{min}} \) else \( A_{vs} = \max(A_{vs\text{min}}, A_{vs}) \).

- Recalculate \( V_s \) in accordance with Clause 8.9.3.5.

\[
V_s = \phi_s f_y A_{vs} d_v \frac{1}{\tan \theta}
\]

- Evaluate the longitudinal rebar on the flexure tension side in accordance with Clause 8.9.3.11 where \( V_s \) is not taken greater then \( V_c \).

\[
A_{vl\text{tens}} = \frac{|M_f|}{d_v} + 0.5 N_f + (|V_u - V_p| - 0.5 V_s) \frac{1}{\tan \theta}
\]

\[
\phi_s f_y
\]

- Evaluate the longitudinal rebar on the flexure compression side in accordance with Clause 8.9.3.12 where \( V_s \) is not taken greater then \( V_c \).

\[
A_{vl\text{comp}} = \frac{0.5 N_f + (|V_u - V_p| - 0.5 V_s) \frac{1}{\tan \theta} - \frac{|M_f|}{d_v}}{\phi_s f_y}
\]
Assign longitudinal rebar to the top or bottom side of the girder based on the moment sign.

If $M_f < 0$ then $A_{vl\text{top}} = A_{vl\text{tens}}$ and $A_{vl\text{bot}} = A_{vl\text{com}}$

else $A_{vl\text{top}} = A_{vl\text{com}}$ and $A_{vl\text{bot}} = A_{vl\text{tens}}$

### 6.3 Flexure Design

The following design parameters are defined by the user in the design request:

- **Highway Class** – Highway Class per clause 1.4.2.2; Default Value = A, Typical value(s): A,B,C,D. The classification is used to determine $F$ and $C_f$ factors

- **$\Phi_C$** – Resistance Factor for concrete; Default Value = 0.75.

- **$\Phi_P$** – Resistance Factor for prestressing strands; Default Value = 0.95

- **$\Phi_S$** – Resistance Factor for reinforcing bars; Default Value = 0.90

### 6.3.1 Variables

- **$M_f$**  Factored flexural resistance
- **$t_{slab\text{eq}}$**  Thickness of top slab
- **$b_{slab}$**  Effective slab width
- **$b_v$**  Thickness of web
- **$A_{slab}$**  Effective area of slab
- **$a$**  Depth of equivalent stress block in accordance with 8.8.3
- **$A_{PS}$**  Area of PT in tension zone
- **$d_P$**  Distance from extreme compression fiber to the centroid of the prestressing tendons in tension zone
- **$A_S$**  Area of reinforcement in tension zone
Design Process

The derivation of the moment resistance of the section is based on approximate stress distribution specified in Article 8.8.3. The natural relationship between concrete stress and strain is considered satisfied by an equivalent rectangular concrete compressive stress block of \( \alpha \phi \sigma'_{c} \) over a zone bounded by the edges of the cross-section and a straight line located parallel to the neutral axis at the distance \( a = \beta_{1} c \) from the extreme compression fiber. The distance \( c \) is measured perpendicular to the neutral axis. The factor \( \beta_{1} \) is taken as 0.97-0.0025 \( f'_{c} \) except that \( \beta_{1} \) is not to be taken to be less than 0.67.

The flexural resistance is determined in accordance with Clause 8.8.3. The resistance is evaluated only for bending about horizontal axis 3. Separate capacity is calculated for positive and negative moment. The capacity is based on bonded tendons and mild steel located in tension zone as defined in the Bridge Object. Tendons and mild steel reinforcement located in compression zone are not considered. It is assumed that all defined tendons in a section, stressed or not, have \( f_{pe} \) (effective stress after loses) larger than 0.5 \( f_{pu} \) (specified tensile strength). If a certain tendon should not be considered for the flexural capacity calculation, its area must be set to zero.
The section properties are calculated for the section before skew, grade, and superelevation are applied. This is consistent with the demands being reported in the section local axis. The effective width of the flange (slab) in compression is evaluated per Clause 5.8.2.1.

6.3.3 Algorithms

At each section:

- All section properties and demands are converted from CSiBridge model units to N, mm.
- The slab effective width is evaluated based on Clause 5.8.2.1
- The equivalent slab thickness is evaluated based on the tributary slab area and the slab width assuming a rectangular shape.

\[ t_{	ext{slab eq}} = \frac{A_{\text{slab}}}{b_{\text{slab}}} \]

- \(\alpha_1\) and \(\beta_1\) stress block factors are evaluated in accordance with 8.8.3 based on section

\[ \alpha_1 = 0.85 - 0.0015f'_c \geq 0.67 \]
\[ \beta_1 = 0.97 - 0.0025f'_c \geq 0.67 \]

- The tendon and rebar location, area, and material are read. Only bonded tendons are processed; unbonded tendons are ignored.

Tendons and rebars are split into two groups depending on the sign of moment they resist—negative or positive. A tendon or rebar is considered to resist a positive moment when it is located outside of the top fiber compression stress block and is considered to resist a negative moment when it is located outside of the bottom fiber compression stress block. The compression stress block extends over a zone bounded by the edges of the cross-section and a straight line located parallel to the neutral axis at the distance \(a = \beta_1 c\) from the extreme compression fiber. The distance \(c\) is measured perpendicular to the neutral axis.
For each tendon group, an area weighted average of the following values is determined:

- sum of tendon areas \( A_{ps} \)
- distance from center of gravity of tendons \( d_P \) to extreme compression fiber
- specified tensile strength of prestressing steel \( f_{pu} \)
- constant \( k \)

\[
k = 2 \left( 1.04 - \frac{f_{pu}}{f_{pu}} \right)
\]

For each rebar group the following values are determined:

- sum of tension rebar areas \( A_s \)
- distance from extreme compression fiber to the centroid of tension rebars \( d_s \)

- Moment resistance – first it is assumed that the equivalent compression stress block is within the top slab. Distance \( c \) between the neutral axis and the compressive face is calculated in accordance with Clause C8.8.4.1.

\[
c = \frac{\varphi_P A_{PS} f_{pu} + \varphi_s A_s f_s}{\alpha_1 \varphi_c f' \beta_1 b_{stab} + k A_{PS} \frac{f_{pu}}{d_p}}
\]

The distance \( c \) is compared to the equivalent slab thickness to determine if the section is a T-section or rectangular section.

If \( c \beta > t_{slabeq} \), the section is a T-section.

If the section is a T-section, the distance \( c \) is recalculated in accordance with Clause C8.8.4.1.
Chapter 6 – Design of Multi-cell Concrete Box using Simplified Method of Analysis

\[ c = \frac{\varphi_p A_{PS} f_{PS} + A_s f_s - \alpha_1 \varphi_c f' c (b_{stab} - b_{webeq}) t_{stabeq}}{\alpha_1 \varphi_c f' c \beta_1 b_{webeq} + k A_{PS} f_{pu} \frac{f_{pu}}{d_p}} \]

- Average stress in prestressing steel \( f_{ps} \) is calculated in accordance with Clause 8.8.4.2

\[ f_{PS} = f_{PU} \left(1 - k \frac{c}{d_p}\right) \]

- Factored flexural resistance \( M_r \) is calculated in accordance with Clause C8.8.4.1.

If the section is a T-section, then

\[ M_r = \varphi_p A_{PS} f_{PS} \left(d_p - \frac{c \beta_1}{2}\right) + \varphi_s A_s f_y \left(d_s - \frac{c \beta_1}{2}\right) + \alpha_1 \varphi_c f' c (b_{stab} - b_{webeq}) t_{stabeq} \left(\frac{c \beta_1}{2} - \frac{t_{stabeq}}{2}\right) \]

else

\[ M_r = \varphi_p A_{PS} f_{PS} \left(d_p - \frac{c \beta_1}{2}\right) + \varphi_s A_s f_y \left(d_s - \frac{c \beta_1}{2}\right) \]

- Extreme moment M3 demands are found from the specified demand sets and the controlling demand set name is recorded.

The process for evaluating negative moment resistance is analogous.
Chapter 7
Design Precast Concrete Girder Bridges

This chapter describes the algorithms applied in accordance with the CAN/CSA-S6-14 code for design and stress check when the superstructure has a deck that includes precast I or U girders with composite slabs. The algorithm is based on the Simplified Method of Analysis, as described in Section 5.6.4 of the code.

For PrecastComp design in CSiBridge each beam and tributary composite slab are designed separately. Moments and shears due to live load are distributed to individual beams in accordance with the factors specified in clause 5.6.4.3 of the code. When CSiBridge calculates the Live Load Distribution Factors (LLDFs), the section and span qualification criteria stated in CAN/CSA-S6-14 5.6.2 for slab on girder bridges are verified and non-compliant sections are not designed.

With respect to shear and torsion check per Clause 8.9 of the code, torsion is ignored.

7.1 Stress Design

The following design parameters are defined by the user in the design request:

- $FactorCompLim - f'_c$ multiplier; Default Value = 0.6. The $f'_c$ is multiplied by the $FactorCompLim$ to obtain compression limit
- \( \text{FactorTensLim} - \sqrt{f'_{c}} \) multiplier; Default Value = 0.4(MPa); The \( \sqrt{f'_{c}} \) is multiplied by the \( \text{FactorTensLim} \) to obtain tension limit.

The stresses are evaluated at three points at the top fiber of the composite slab: the left corner, the centerline beam and the right corner of the composite slab tributary area. The location of stress output points at the slab bottom fiber and beam top and bottom fiber depends on the type of precast beam present in the section cut. The location is labeled in the output plots and tables.

Concrete strength \( f'_{c} \) is read at every point and compression and tension limits are evaluated using the \( \text{FactorCompLim} - f'_{c} \) multiplier and \( \text{FactorTensLim} - \sqrt{f'_{c}} \) multiplier.

The stresses assume linear distribution and take into account axial (\( P \)) and either both bending moments (\( M_{2} \) and \( M_{3} \)) or only \( P \) and \( M_{3} \), depending on which method for determining LLDF has been specified in the Design Request (see Chapters 3 and 4).

The stresses are evaluated for each demand set. Extremes are found for each point and the controlling demand set name is recorded.

The stress limits are evaluated by applying the preceding parameters.

### 7.2 Shear Design

The following design parameters are defined by the user in the design request:

- \( \text{Highway Class} \) – Highway Class per clause 1.4.2.2; Default Value = A, Typical value(s): A,B,C,D. The classification is used to determine \( F \) and \( C_{f} \) factors.

- \( \text{PhiC} \) – Resistance Factor for concrete; Default Value = 0.75.

- \( \text{PhP} \) – Resistance Factor for prestressing strands; Default Value = 0.95

- \( \text{PhiS} \) – Resistance Factor for reinforcing bars; Default Value = 0.90

- \( \text{FactRupture} \) - multiplies sqrt \( f'_{c} \) [MPa] to obtain cracking strength; Default Value = 0.40
Chapter 7 - Design Precast Concrete Girder Bridges

- EpsXLimNeg – limit on minimum longitudinal strain per Clause 8.9.3.8, Default Value = -0.2x10^{-3}
- EpsXLimPos – limit on maximum longitudinal strain per Clause 8.9.3.8, Default Value = 3.0x10^{-3}
- Shear Rebar Material – A previously defined rebar material label that will be used to determine the required area of transverse rebar in the girder.
- Longitudinal Rebar Material - A previously defined rebar material label that will be used to determine the required area of longitudinal rebar in the girder.

7.2.1 Variables

- \( V_f \) Factored shear demand per girder excluding force in tendons
- \( N_f \) Applied factored axial force taken as positive if tensile
- \( M_f \) Factored moment at the section
- \( V_{2c} \) Shear in Section Cut excluding force in tendons
- \( V_{2Tot} \) Shear in Section Cut including force in tendons
- \( V_p \) Component in the direction of the applied shear of the effective prestressing force; if \( V_p \) has the same sign as \( V_f \), the component is resisting the applied shear
- \( d_v \) Effective shear depth in accordance with 8.9.1.5 of the code.
- \( d_{girder} \) Depth of girder
- \( d_{compslab} \) Depth of composite slab (includes concrete haunch t2)
- \( d_{PTBot} \) Distance from top of composite slab to center of gravity of tendons in the bottom of the precast beam
- \( b_v \) Minimum web width of beam
- \( A_{ps} \) Area of prestressing steel on the flexural tension side of the member,
7.2.2 Design Process

The shear resistance is determined in accordance with paragraph 8.9.3 of the code (sectional design model derived from Modified Compression Field Theory). The procedure assumes that the concrete shear stresses are distributed uniformly over an area $b_w$ wide and $d_v$ deep, that the direction of principal compressive stresses (defined by angle $\theta$) remains constant over $d_v$, and that the shear strength of the section can be determined by considering the biaxial stress conditions at just one location in the web. For design, the user should select only those sections that comply with these assumptions by defining appropriate station ranges in the Design Request (see Chapter 4).

It is assumed that the precast beams are pre-tensioned, and therefore, no ducts are present in webs. The effective web width is taken as the minimum web width, measured parallel to the neutral axis, between the resultants of the tensile and compressive forces as a result of flexure.
Shear design is completed on a per-girder basis. Please refer to Chapter 3 for a description of the live load distribution to individual girders.

7.2.3 Algorithms

- All section properties and demands are converted from CSiBridge model units to N, mm.

- For every COMBO specified in the Design Request that contains envelopes, two new force demand sets are generated. The new force demand sets are built up from the maximum tension values of $P$ and the maximum and minimum values of $V_2$ and minimum values of $M_3$ of the two StepTypes (Max and Min) present in the envelope COMBO case. The StepType of these new force demand sets are named MaxM3MinV2 and MinM3MaxV2, respectively. The signs of all force components are preserved. The two new cases are added to comply with industry practice where sections are designed for extreme shear and moments that are not necessarily corresponding to the same design vehicle position. The section cut is designed for all four StepTypes in the COMBO—Max, Min, MaxM3MinV2, and MinM3MaxV2—and the controlling StepType is reported.

- In cases where the demand moment $M_f < |V_f - V_p|d_v$ two new force demand sets are generated as follows:
  \[ M_{fpos} = |V_f - V_p|d_{vpos} \]
  \[ M_{fneg} = -|V_f - V_p|d_{vneg} \]
  The acronyms “-CodeMinMuPos” and “-CodeMinMuNeg” are added to the end of the StepType name. The signs of the P and V2 are preserved.

- The component in the direction of the applied shear of the effective pre-stressing force, positive if resisting the applied shear, is evaluated:
  \[ V_p = \frac{V_2c - V_2{\text{Tot}}}{n_{\text{girders}}} \]

- Effective shear depth $d_v$ is evaluated per Clause 8.9.1.5.

  If $M_f > 0$, then $d_v = \max(0.72d_{\text{girders}}, 0.9d_{\text{PTBot}})$

  If $M_f < 0$, then $d_v = \max(0.72d_{\text{girders}}, 0.9d_{\text{PTTop}})$
The demand/capacity ratio (D/C) is calculated based on the maximum permissible shear capacity at a section in accordance with Section 8.9.3.3 of the code

\[ \frac{D}{C} = \frac{|V_f - V_p|}{0.25\phi f'c b_d d_v} \]

- Evaluate numerator and denominator of \( \varepsilon_x \) (Clause 8.9.3.8)

\[ \varepsilon_{x_{\text{numerator}}} = \frac{|M_f|}{d_v} + |V_f - V_p| + 0.5N_f - A_{ps}0.7f_{pu} \]

\[ \varepsilon_{x_{\text{denominator}}} = 2(E_p A_{ps}) \]

- Adjust denominator values as follows

If \( \varepsilon_{x_{\text{denominator}}} = 0 \) and \( \varepsilon_{x_{\text{numerator}}} \geq 0 \) then \( \varepsilon_x = \varepsilon_{x_{\text{limpos}}} \) and

If \( \varepsilon_{x_{\text{numerator}}} < 0 \) then \( \varepsilon_{x_{\text{denominator}}} = 2(E_p A_{ps} + E_c A_{ct}) \)

- Evaluate (eq. 5.8.3.4.2-4)

\[ \varepsilon_x = \frac{\varepsilon_{x_{\text{numerator}}}}{\varepsilon_{x_{\text{denominator}}}} \]

- Check if axial tension is large enough to crack the flexural compression face of the section.

If \( \frac{N_f}{A_{girder}} > 0.4\sqrt{f'_c} \) then \( \varepsilon_x = 2\varepsilon_x \)

- Check against the limit on the longitudinal strain specified in the Design Request

\[ \varepsilon_x = \max(\varepsilon_x, \varepsilon_{x_{\text{limneg}}}) \text{ and } \varepsilon_x = \min(\varepsilon_x, \varepsilon_{x_{\text{limpos}}}) \]

- Evaluate the angle \( \theta \) of inclination of diagonal compressive stresses as determined in Clause 8.9.3.7.

\[ 18^\circ \leq 29^\circ + 7000\varepsilon_x \leq 45^\circ \]
- Evaluate the factor indicating the ability of diagonally cracked concrete to transmit tension and shear, as specified in Clause 8.9.3.7

\[
\beta = \frac{0.4}{1 + 1500 \varepsilon_x} \leq 0.525
\]

- Evaluate the nominal shear resistance provided by tensile stresses in the concrete per Clause 8.9.3.4.

\[
V_c = 2.5 \beta \phi_c f_{cr} b_v d_v \text{ where } f_{cr} < 3.2 \text{ MPa}
\]

- Evaluate how much shear demand is left to be carried by rebar and determine amount of required transverse reinforcement per unit of length – Clause 8.9.3.5.

\[
V_s = |V_f - V_p| - V_c
\]

if \(V_s < 0\), then \(A_{vs} = 0\) else

\[
A_{vs} = \frac{V_s}{\phi_s f_y d_v \tan \theta}
\]

- Check against minimum transverse shear reinforcement per Clause 8.9.1.2 and 8.9.1.3.

if \(|V_f| > 0.2 \phi_c f_{cr} b_v d_v + 0.5 \phi_p |V_p|\) then

\[
A_{vs\text{min}} = \frac{0.15 f_{cr} b_v}{f_y}
\]

else \(A_{vs\text{min}} = 0\).

If \(V_s < 0\), then \(A_{vs} = A_{vs\text{min}}\) else \(A_{vs} = \max(A_{vs\text{min}}, A_{vs})\).

- Recalculate \(V_s\) in accordance with Clause 8.9.3.5.

\[
V_s = \phi_s f_y A_{vs} d_v \frac{1}{\tan \theta}
\]
- Evaluate the longitudinal rebar on the flexure tension side in accordance with Clause 8.9.3.11 where $V_s$ is not taken greater then $V_c$.

$$A_{vl\text{tens}} = \frac{|M_f|}{d_v} + 0.5N_f + \left(|V_u - V_p| - 0.5V_s\right) \frac{1}{\tan \theta}$$

- Evaluate the longitudinal rebar on the flexure compression side in accordance with Clause 8.9.3.12 where $V_s$ is not taken greater then $V_c$.

$$A_{vl\text{comp}} = \frac{0.5N_f + \left(|V_u - V_p| - 0.5V_s\right) \frac{1}{\tan \theta} - \frac{|M_f|}{d_v}}{\phi_s f_y}$$

- Assign longitudinal rebar to the top or bottom side of the girder based on the moment sign.

  If $M_f < 0$ then $A_{vl\text{top}} = A_{vl\text{tens}}$ and $A_{vl\text{bot}} = A_{vl\text{comp}}$

  else $A_{vl\text{top}} = A_{vl\text{comp}}$ and $A_{vl\text{bot}} = A_{vl\text{tens}}$

### 7.3 Flexure Design

The following design parameters are defined by the user in the design request:

- **Highway Class** – Highway Class per clause 1.4.2.2; Default Value = A, Typical value(s): A,B,C,D. The classification is used to determine $F$ and $C_r$ factors.

- **$\phi_C$** – Resistance Factor for concrete; Default Value = 0.75.

- **$\phi_P$** – Resistance Factor for prestressing strands; Default Value = 0.95

- **$\phi_S$** – Resistance Factor for reinforcing bars; Default Value = 0.90

### 7.3.1 Variables

- $M_f$ - Nominal flexural resistance
- $t_{\text{slabeq}}$ - Thickness of composite slab
Chapter 7 - Design Precast Concrete Girder Bridges

\[ b_{\text{slab}} \] Effective slab width

\[ b_{\text{web eq}} \] Thickness of beam web

\[ A_{\text{slab}} \] Effective area of slab

\[ a \] Depth of equivalent stress block in accordance with 8.8.3.

\[ A_{PS} \] Area of PT in tension zone

\[ d_P \] Distance from extreme compression fiber to the centroid of the prestressing tendons in tension zone

\[ A_S \] Area of reinforcement in tension zone

\[ d_S \] Distance from extreme compression fiber to the centroid of rebar in tension zone

\[ f_y \] Yield strength of rebar

\[ f_{pu} \] Specified tensile strength of prestressing steel (area weighted average of all tendons in tensile zone)

\[ f_{py} \] Yield tensile strength of prestressing steel (area weighted average if all tendons in tensile zone)

\[ f_{ps} \] Average stress in prestressing steel (Clause 8.8.4.2)

\[ k \] PT material constant (Clause 8.8.4.2)

\[ \alpha_1 \] Ratio of averaged stress in a rectangular compression block to the specified concrete strength as specified in Clause 8.8.3.

\[ \beta_1 \] Factor as specified in Clause 8.8.3.

### 7.3.2 Design Process

The derivation of the moment resistance of the section is based on approximate stress distribution specified in Article 8.8.3. The natural relationship between concrete stress and strain is considered satisfied by an equivalent rectangular concrete compressive stress block of \( \alpha_1 \phi_f'c \) over a zone bounded by the edges
of the cross-section and a straight line located parallel to the neutral axis at the
distance \( a = \beta_1 c \) from the extreme compression fiber. The distance \( c \) is measured
perpendicular to the neutral axis. The factor \( \beta_1 \) is taken as 0.97-0.0025\( f'c \) except
that \( \beta_1 \) is not to be taken to be less than 0.67.

The flexural resistance is determined in accordance with Clause 8.8.3. The re-
sistance is evaluated only for bending about horizontal axis 3. Separate capacity
is calculated for positive and negative moment. The capacity is based on bonded
tendons and mild steel located in tension zone as defined in the Bridge Object.
Tendons and mild steel reinforcement located in compression zone are not con-
sidered. It is assumed that all defined tendons in a section, stressed or not, have
\( f_{pe} \) (effective stress after loses) larger than 0.5 \( f_{pu} \) (specified tensile strength). If a
certain tendon should not be considered for the flexural capacity calculation, its
area must be set to zero.

The section properties are calculated for the section before skew, grade, and su-
perelevation are applied. This is consistent with the demands being reported in
the section local axis. The effective width of the flange (slab) in compression is
evaluated per Clause 5.8.2.1.

### 7.3.3 Algorithms

At each section:

- All section properties and demands are converted from CSiBridge model
  units to N, mm.
- The slab effective width is evaluated based on Clause 5.8.2.1
- \( \alpha_1 \) and \( \beta_1 \) stress block factors are evaluated in accordance with 8.8.3
  based on section \( f'c \)
  \[
  \alpha_1 = 0.85 - 0.0015 f'c \geq 0.67
  \]
  \[
  \beta_1 = 0.97 - 0.0025 f'c \geq 0.67
  \]
- The tendon and rebar location, area and material are read. Only bonded
tendons are processed; unbonded tendons are ignored.
Tendons and rebars are split into two groups depending on what sign of moment they resist—negative or positive. A tendon or rebar is considered to resist a positive moment when it is located outside of the top fiber compression stress block and it is considered to resist a negative moment when it is located outside of the bottom fiber compression stress block. The compression stress block extends over a zone bounded by the edges of the cross-section and a straight line located parallel to the neutral axis at the distance \( a = \beta_1 c \) from the extreme compression fiber. The distance \( c \) is measured perpendicular to the neutral axis.

For each tendon group, an area weighted average of the following values is determined:
- sum of tendon areas \( \text{A}_p \)
- center of gravity of tendons \( d_p \)
- specified tensile strength of prestressing steel \( f_{pu} \)
- constant \( k \)

\[
k = 2 \left( 1.04 - \frac{f_{ps}}{f_{pu}} \right)
\]

For each rebar group the following values are determined:
- sum of tension rebar areas \( \text{A}_s \)
- distance from extreme compression fiber to the centroid of tension rebars \( d_s \)

- Positive moment resistance – first it is assumed that the equivalent compression stress block is within the top slab. Distance \( c \) between the neutral axis and the compressive face is calculated in accordance with Clause C8.8.4.1.

\[
c = \frac{\varphi_p \text{A}_p f_{pu} + \varphi_s \text{A}_s f_s}{\alpha_1 \varphi_c f_{c} \beta_1 b_{slab} + k \text{A}_p \frac{f_{pu}}{d_p}}
\]

The distance \( c \) is compared to the slab thickness. If the distance to the neutral axis \( c \) is larger than the composite slab thickness, the distance \( c \) is re-evaluated. For this calculation, the beam flange width and area are converted to their
equivalents in slab concrete by multiplying the beam flange width by the modular ratio between the precast girder concrete and the slab concrete. The web width in the equation for \( c \) is substituted for the effective converted girder flange width. The distance \( c \) is recalculated in accordance with Clause C8.8.4.1.

\[
c = \frac{\varphi_p A_{PS} f_{pu}}{A_s f_s - \alpha_1 \varphi_c f'_c (b_{slab} - b_{webeq}) t_{slabeq}} + \alpha_1 \varphi_c f'_c c \beta_1 b_{webeq} + k A_{PS} \frac{f_{pu}}{d_p}
\]

If the calculated value of \( c \) exceeds the sum of the deck thickness and the equivalent precast girder flange thickness, the program assumes the neutral axis is below the flange of the precast girder and recalculates \( c \). The term \( 0.85 f'_c (b - b_w) \) in the calculation is broken into two terms, one refers to the contribution of the deck to the composite section flange and the second refers to the contribution of the precast girder flange to the composite girder flange.

- Average stress in prestressing steel \( f_{ps} \) is calculated in accordance with Clause 8.8.4.2

\[
f_{ps} = f_{pu} (1 - k \frac{c}{d_p})
\]

- Factored flexural resistance \( M_r \) is calculated in accordance with Clause C8.8.4.1.

If the section is a T-section, then

\[
M_r = \varphi_p A_{PS} f_{ps} \left( d_p - \frac{c \beta_1}{2} \right) + \varphi_s A_s f_s \left( d_s - \frac{c \beta_1}{2} \right) + \alpha_1 \varphi_c f'_c (b_{slab} - b_{webeq}) t_{slabeq} \left( \frac{c \beta_1}{2} - \frac{t_{slabeq}}{2} \right)
\]

else
\[ M_x = \varphi_p A_{ps} f_{ps} \left( d_p - \frac{c_1}{2} \right) + \varphi_s A_s f_y \left( d_s - \frac{c_1}{2} \right) \]

- Extreme moment M3 demands are found from the specified demand sets and the controlling demand set name is recorded.

The process for evaluating negative moment resistance is analogous.
This chapter describes the algorithms CSiBridge applies when designing steel I-beam with composite slab superstructures in accordance with CAN/CSA-S6-14.

8.1 Section Properties

8.1.1 Yield Moments

8.1.1.1 Composite Section in Positive Flexure

The depth of web in compression that is used in section classification is derived based on positive yield moment, $M_y$. The positive yield moment is determined by the program using the following user-defined input, which is part of the Ultimate Design Request (see Chapter 4 for more information about Design Request).

$M_{dc} = \text{The user specifies in the Design Request the name of the combo that represents the moment caused by the permanent load applied before the concrete deck has hardened or is made composite.}$

$M_{dc} = \text{The user specifies in the Design Request the name of the combo that represents the moment caused by the remainder of the permanent load (applied to the composite section).}$

The program solves for $M_{yd}$ from the following equation,
and then calculates yield moment based on the following equation

\[ M_y = M_{dnc} + M_{dc} + M_{AD} \]

where

- \( S_{NC} \) = Noncomposite section modulus
- \( S_{LT} \) = Long-term composite section modulus
- \( S_{ST} \) = Short-term composite section modulus

\( M_y \) is taken as the lesser value calculated for the compression flange, \( M_{sc} \), or the tension flange, \( M_{st} \). The positive \( M_y \) is calculated only once based on \( M_{dnc} \) and \( M_{dc} \) demands specified by the user in the Design Request. It should be noted that the \( M_y \) calculated in the procedure described here is used by the program to determine only the depth of web in compression that is used in classification of webs in accordance with CAN/CSA-S6-14 Table 10.3.

Since for Staged and Non-Staged Constructability Design Checks it is difficult to obtain built-up elastic stresses, for the sake of classification of the web it is assumed that the depth of the web in compression for positive bending is based on all stresses being applied to non-composite sections because this produces the greatest depth of web in compression.

### 8.1.1.2 Composite Section in Negative Flexure

For composite sections in negative flexure, the procedure described for positive yield moment is followed, except that the composite section for both short-term and long-term moments consists of the steel section and the longitudinal reinforcement within the tributary width of the concrete deck. Thus, \( S_{ST} \) and \( S_{LT} \) are the same value. Also, \( M_{st} \) is taken with respect to either the tension flange or the longitudinal reinforcement, whichever yields first. Concrete tension capacity is ignored.

For the sake of classification of the web, the depth of the web in compression for negative bending is based on all stresses being applied to the composite section because this produces the greatest depth of web in compression. This assumption applies to all design checks.
8.1.2 Plastic Moments

8.1.2.1 Composite Section in Positive Flexure Class 1 and 2

The positive plastic moment, \( M_{pl,Rd} \), is calculated as the moment of the plastic forces about the plastic neutral axis. Plastic forces in the steel portions of a cross-section are calculated using the yield strengths of the flanges, the web, and reinforcing steel, as appropriate. The plastic force in the effective width of the composite slab that is in compression is based on a rectangular stress block with the magnitude of the compressive stress equal to \( \phi_s \alpha_1 f'_c \), where \( \alpha_1 \) is ratio of average stress in rectangular compression stress block to the specified concrete strength, taken as 0.85 – 0.0015\( f'_c \) but not less than 0.67. The effective slab width is determined in accordance with Clause 5.8.2.1. Effect of concrete in tension is neglected. The position of the plastic neutral axis is determined by the equilibrium condition such that there is no net axial force.

The plastic moment of a composite section in positive flexure is determined as follows:

- Calculate the element forces and use them to determine if the plastic neutral axis is in the web, top flange, or concrete deck.
- Calculate the location of the plastic neutral axis within the element determined in the first step.
- Calculate \( M_{pl,Rd} \).

Equations for the various potential locations of the plastic neutral axis (PNA) are given in Table 8-1.

<table>
<thead>
<tr>
<th>Case</th>
<th>PNA Condition</th>
<th>( \bar{\nu} ) and ( M_p )</th>
</tr>
</thead>
<tbody>
<tr>
<td>I In Web</td>
<td>( P_t + P_w \geq P_c + P_t + P_ds + P_d )</td>
<td>( \bar{\nu} = \frac{D}{2} \left[ \frac{P_r - P_t - P_w - P_ds + P_ds}{P_w} + 1 \right] )</td>
</tr>
<tr>
<td>II In Top Flange</td>
<td>( P_t + P_w + P_s \geq P_t + P_s + P_ds + P_d )</td>
<td>( \bar{\nu} = \frac{L}{2} \left[ \frac{P_r - P_t - P_w - P_ds + P_ds}{P_t} + 1 \right] )</td>
</tr>
</tbody>
</table>
### Table 8-1 Calculation of PNA and $M_p$ for Sections in Positive Flexure

<table>
<thead>
<tr>
<th>Case</th>
<th>Condition</th>
<th>$\bar{Y}$ and $M_p$</th>
</tr>
</thead>
<tbody>
<tr>
<td>III</td>
<td>Concrete Deck Below $P_{rb}$</td>
<td>$P_r + P_w + P_c + \frac{c_b}{t_s} P_r + P_{rb} + P_s$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\bar{Y} = \left(t_s\right) \left[ \frac{P_r + P_w + P_c + P_{rb} - P_{rb}}{P_r} \right]$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$M_p = \left(\frac{\bar{Y}^2 P_r}{2 t_s}\right) + \left[P_r d_r + P_s d_s + P_c d_c + P_{rb} d_{rb} + P_{rb} d_{rb} + P_{rb} d_{rb} + P_{rb} d_{rb}\right]$</td>
</tr>
<tr>
<td>IV</td>
<td>Concrete Deck at $P_{rb}$</td>
<td>$P_r + P_w + P_c + P_{rb} + \frac{c_b}{t_s} P_r + P_s$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\bar{Y} = c_b$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$M_p = \left(\frac{\bar{Y}^2 P_r}{2 t_s}\right) + \left[P_r d_r + P_s d_s + P_c d_c + P_{rb} d_{rb} + P_{rb} d_{rb} + P_{rb} d_{rb} + P_{rb} d_{rb}\right]$</td>
</tr>
<tr>
<td>V</td>
<td>Concrete Deck Above $P_{rb}$ and Below $P_{rt}$</td>
<td>$P_r + P_w + P_c + P_{rb} + \frac{c_s}{t_t} P_r + P_s$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\bar{Y} = \left(t_t\right) \left[ \frac{P_{rb} + P_r + P_w + P_c - P_{rt}}{P_r} \right]$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$M_p = \left(\frac{\bar{Y}^2 P_r}{2 t_t}\right) + \left[P_r d_r + P_s d_s + P_c d_c + P_{rb} d_{rb} + P_{rb} d_{rb} + P_{rb} d_{rb} + P_{rb} d_{rb}\right]$</td>
</tr>
<tr>
<td>VI</td>
<td>Concrete Deck at $P_{rt}$</td>
<td>$P_r + P_w + P_c + P_{rb} + P_s + \frac{c_s}{t_t} P_r$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\bar{Y} = c_s$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$M_p = \left(\frac{\bar{Y}^2 P_r}{2 t_t}\right) + \left[P_r d_r + P_s d_s + P_c d_c + P_{rb} d_{rb} + P_{rb} d_{rb} + P_{rb} d_{rb} + P_{rb} d_{rb}\right]$</td>
</tr>
<tr>
<td>VII</td>
<td>Concrete Deck Above $P_{rt}$</td>
<td>$P_r + P_w + P_c + P_{rb} + P_s + \frac{c_s}{t_t} P_r$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\bar{Y} = \left(t_t\right) \left[ \frac{P_{rb} + P_r + P_w + P_c + P_{rt}}{P_r} \right]$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$M_p = \left(\frac{\bar{Y}^2 P_r}{2 t_t}\right) + \left[P_r d_r + P_s d_s + P_c d_c + P_{rb} d_{rb} + P_{rb} d_{rb} + P_{rb} d_{rb} + P_{rb} d_{rb}\right]$</td>
</tr>
</tbody>
</table>

### Figure 8-1 Plastic Neutral Axis Cases

in which
8.1.2.2 Composite Section in Positive Flexure Class 3

For composite sections in which the depth of the compression portion of the web of the steel section, calculated on the basis of a fully plastic stress distribution, exceeds $850w/\sqrt{F_y}$, the factored moment resistance, $M_r$, of the composite section is calculated on the basis of fully plastic stress blocks, as shown in Figure 8.2, as follows:

$$M_r = C_c e_c + C_r e_r + C_s e_s$$

where

$$C_c = \alpha_1 \phi_s f' c_b t_c$$
$$C_r = \phi_t A_t f_y$$
$$C_s = \phi_s A'_s f_y$$

The area of the steel section in compression, $A'_{sc}$, includes the top flange and a web area of $(850w^2)/\sqrt{F_y}$, and the area of the steel section in tension, $A'_{st}$, is calculated as follows:

$$A'_{st} = \frac{C_c + C_r + C_s}{\phi_s f_y}$$

$$A'_{sc} = \frac{\alpha_1 \phi_s f' c_b t_c}{\phi_f c'_f}$$
8.1.2.3 Composite Section in Negative Flexure

The plastic moment of a composite section in negative flexure is calculated by an analogous procedure. Equations for the two cases most likely to occur in practice are given in Table 8-2. The plastic moment of a non-composite section is calculated by eliminating the terms pertaining to the concrete deck from the equations in Table 8-1.

Table 8-2 Calculation of PNA and $M_p$ for Sections in Negative Flexure

<table>
<thead>
<tr>
<th>Case</th>
<th>PNA Condition</th>
<th>$\bar{Y}$ and $M_p$</th>
</tr>
</thead>
<tbody>
<tr>
<td>I In Web $P_c + P_w \geq P_t + P_{rb} + P_n$</td>
<td>$\bar{Y} = \left(\frac{D}{2}\right) \left[\frac{P_t - P_{pt} - P_{ptb}}{P_t} + 1\right]$</td>
<td>$M_p = \frac{P_n}{2D} \left[\bar{Y}^2 + (D - \bar{Y})^2\right] + \left[P_t d_t + P_c d_c + P_w d_w + P_{rb} d_{rb}\right]$</td>
</tr>
<tr>
<td>II In Top Flange $P_c + P_w + P_t \geq P_{rb} + P_n$</td>
<td>$\bar{Y} = \left(\frac{t_t}{2}\right) \left[\frac{P_t - P_{pt} - P_{ptb}}{P_t} + 1\right]$</td>
<td>$M_p = \frac{P_t}{2t_t} \left[\bar{Y}^2 + (t_t - \bar{Y})^2\right] + \left[P_t d_t + P_c d_c + P_w d_w + P_{rb} d_{rb}\right]$</td>
</tr>
</tbody>
</table>

Figure 8-2 Plastic Neutral Axis Cases

in which
\[ P_{rt} = F_{yrt} A_{rt} \]
\[ P_{rb} = F_{yrb} A_{rb} \]
\[ P_c = F_{yc} b_c d_c \]
\[ P_w = F_{yw} D t_w \]
\[ P_t = F_{yt} b_d t \]

In the equations for \( M_p \) given in Tables 8-1 and 8-2, \( d \) is the distance from an element force to the plastic neutral axis. Element forces act at (a) mid-thickness for the flanges and the concrete deck, (b) mid-depth of the web, and (c) center of reinforcement. All element forces, dimensions, and distances are taken as positive. The condition are checked in the order listed in Tables 8-1 and 8-2.

### 8.1.3 Classification of Cross-Sections

At each section cut the steel beam section is classified in accordance with Clause 10.9.2.. The classification is carried out separately for positive and negative bending for both composite and non-composite sections. The classification of a cross-section depends on the width to thickness ratio of the parts subject to compression. A cross-section is classified according to the highest (least favorable) class of its compression parts.

For calculating the limiting width-to-thickness ratios of the web of monosymmetric sections, \( h \) in Table 10.3 is replaced by \( 2d_c \).

#### 8.1.3.1 Composite Positive Bending

The resistance of the top flange is assumed as not being limited by its local buckling resistance since it is restrained by effective attachment to a concrete flange by shear connectors. The top flange is always classified as Class 1.

When classifying the web, it is first assumed that the section satisfies requirements for Class 1 or 2, and the depth of web in compression is based on the plastic range of the composite section for positive moment. When the web does not satisfy requirements for Class 1 or 2 the section is classified as Class 3. In the next step, the web is verified for Class 3, where the depth of web in compression is based on positive yield moment. See section 8.1.1.1.1 of this manual for derivation of the yield moment for positive bending of a composite section. When the web does not satisfy requirements for Class 3, the section is classified as Class 4.
The bottom flange is always in tension and therefore does not have an effect on the classification of the section.

8.1.3.2 Non-Composite Positive Bending

The top flange is in compression and is not restrained by the composite slab. Its resistance may be limited by its local buckling resistance. The flange is classified in accordance with Clause 10.9.2 as part subject to compression.

When classifying the web, it is first assumed that the section satisfies requirements for Class 1 or 2, and the depth of the web in compression is based on the plastic range of the steel beam section for positive moment. When the web does not satisfy requirements for Class 1 or 2, the section is classified as Class 3. In the next step, the web is verified for Class 3, where the depth of web in compression is based on the neutral axis of the steel beam. When the web does not satisfy requirements for Class 3, the section is classified as Class 4.

The bottom slab classification follows the same procedure outlined in Section 8.1.1.3.1 of this manual.

8.1.3.3 Composite Negative Bending

The top flange is always in tension and therefore does not have an effect on the classification of the section.

When classifying the web, it is first assumed that the section satisfies requirements for Class 1 or 2, and the depth of web in compression is based on the plastic range for negative moment. When the web does not satisfy requirements for Class 1 or 2, the section is classified as Class 3. In the next step, the web is verified for Class 3, where the depth of the web in compression is based on the negative yield moment. See section 8.1.1.1.2 of this manual for derivation of the yield moment for negative bending of a composite section. When the web does not satisfy requirements for Class 3, the section is classified as Class 4.

The bottom flange is in compression and unrestrained. The bottom flange resistance may be limited by its local buckling resistance and is classified in accordance with Clause 10.9.2 as part subject to compression.
8.1.3.4 Non-Composite Negative Bending

The classification of top and bottom flanges follows the same procedure as outlined in Section 8.1.1.3.3 of this manual.

When classifying the web, it is first assumed that the section satisfies requirements for Class 1 or 2, and the depth of the web in compression is based on the plastic range of the steel beam for negative moment. When the web does not satisfy requirements for Class 1 or 2, the section is classified as Class 3. In the next step, the web is verified for Class 3, where the depth of the web in compression is based on the position of the neutral axis of the steel beam. When the web does not satisfy requirements for Class 3, the section is classified as Class 4.

8.1.4 Class 4 Sections

Sections classified as Class 4 are flagged as invalid and no results are reported.

8.1.5 Unbraced Length \(L\) and Section Transitions

The program assumes that the top flange is continuously braced for all Design Requests, except Constructability. For more information on flange lateral bracing in the Constructability Design Requests, see section 8.6.3 of this manual.

The unbraced length \(L\) for the bottom flange is equal to the distance between the nearest downstation and the upstation qualifying cross diaphragms or span supports, as defined in the Bridge Object. Some of the diaphragm types available in CSiBridge may not necessarily provide restraint to the bottom flange. The program assumes that the following diaphragm qualifies as providing lateral restraint to the bottom flange: single beam and all types of chord and brace except V braces without bottom beams.

The program calculates demands and capacities pertaining to a given section cut at a given station without considering the section transition within the unbraced length. It does not search for the highest demands versus the smallest resistance within the unbraced length as the code suggests. It is the responsibility of the user to pay special attention to section transition within unbraced lengths and to follow the guidelines in the code.

8.1.6 Laterally Unbraced Flanges

The program calculates buckling verification at every section cut.

From Clause 10.10.2.3, the factored moment resistance \(M_r\) is calculated as
(a) \( M_r = 1.15 \phi_s M_p \left[1 - \frac{0.28 M_p}{M_u}\right] \leq \phi_s M_p \), when \( M_u > 0.67 M_p \); or

(b) \( M_r = \phi_s M_u \), when \( M_u \leq 0.67 M_p \)

The critical elastic moment, \( M_u \), of a monosymmetric section is taken as

\[
M_u = \frac{\omega z \pi}{L} \left[ \sqrt{E_s l_y G_s J} \left[ B_1 + \sqrt{1 + B_2 + B_2^2} \right] \right]
\]

Where

\[
B_1 = \frac{\pi}{2} \frac{\beta_x L}{E_s l_y J} \frac{E_s l_y}{G_s J}
\]

where

- \( \beta_x \) = coefficient of monosymmetry

\[
B_2 = \frac{\pi^2 E_s C_w}{L^2 G_s J}
\]

For doubly symmetric sections,

- \( \beta_x = 0.0 \)
- \( B_1 = 0.0 \)
When the design request parameter ‘Method for determining $\omega_2$’ is set to ‘Program Determined’, then for each demand set the applied bending moments $M_a$, $M_b$, $M_c$ and $M_{\text{max}}$ at the unbraced segment are determined by interpolation of demands at nearest section cuts. The designer should be aware that live load moments at neighboring section cuts within the unbraced segment are not necessarily controlled by the same load pattern and as a result the moment gradient calculation may be impacted.

### 8.2 Design Request Parameters

The following Design Request parameters are available for user control:

**Highway class** – Highway Class per clause 1.4.2.2; Default Value = A, Typical value(s): A,B,C,D. The classification is used to determine $F$ and $C_f$ factors

**$\phi$ steel flexure**, resistance factor for steel in flexure, default value = 0.95
\( \phi_{\text{steel shear}} \), resistance factor for steel in shear, default value = 0.95

\( \phi_{\text{concrete}} \), resistance factor for concrete, default value = 0.75

\( \phi_{\text{rebar}} \), resistance factor for reinforcement, default value = 0.9

Use Stage Analysis to determine stresses on composite section, Yes or No

Modular ratio \( n (=E_s/E_c) \) is the multiplier used to determine long-term composite section properties, default value = 3.0.

Omega2 Specification – method to determine coefficient to account for increased moment resistance of a laterally unsupported beam segment when subject to a moment gradient – Program or User Determined

Omega2 – user defined value of \( \omega_2 \) coefficient to account for increased moment resistance of a laterally unsupported beam segment when subject to a moment gradient

8.3 Demand Sets

Demand Set combos (at least one required) are user-defined combination based on CAN/CSA-S6-14 combinations (see Chapter 4 for more information about specifying Demand Sets). The demands from all specified demand combos are enveloped and used to calculate D/C ratios. The way the demands are used depends on if the design parameter “Use Stage Analysis?” is set to Yes or No.

If “Use Stage Analysis? = Yes,” the program reads the stresses on beams and slabs directly from the section cut results. The stresses are calculated based on gross section based on tributary area of the composite slab.

When “Use Stage Analysis? = Yes,” the program assumes that the effects of the staging of loads applied to non-composite versus composite sections and the concrete slab material time dependent properties were captured by using the Nonlinear Staged Construction load case available in CSiBridge.

If “Use Stage Analysis? = No,” the program decomposes load cases present in every demand set combo to three Bridge Design Action categories: non-composite, composite long term, and composite short term. The program uses the load case Bridge Design Action parameter to assign the load cases to the appropriate categories. A default Bridge Design Action parameter is assigned to a load case based on its Design
Chapter 8 - Design Steel I-Beam Bridge with Composite Slab

Type. However, the parameter can be overwritten: click the Analysis > Load Cases > {Type} > New command to display the Load Case Data – {Type} form; click the Design button next to the Load case type drop down list, under the heading Bridge Design Action select the User Defined option and select a value from the list. The assigned Bridge Designed Action values are handled by the program in the following manner:

Table 8-3 Bridge Design Action

<table>
<thead>
<tr>
<th>Bridge Design Action Value specified by the user</th>
<th>Bridge Design Action Category used in the design algorithm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Non-Composite</td>
<td>Non-Composite</td>
</tr>
<tr>
<td>Long-Term Composite</td>
<td>Long-Term Composite</td>
</tr>
<tr>
<td>Short-Term Composite</td>
<td>Short-Term Composite</td>
</tr>
<tr>
<td>Staged</td>
<td>Non-Composite</td>
</tr>
<tr>
<td>Other</td>
<td>Non-Composite</td>
</tr>
</tbody>
</table>

8.3.1 Demand Flange Stresses \( f_{bu} \)

Evaluation of the flange stress, \( f_{bu} \), is dependent on setting the Design Request parameter “Use Stage Analysis?”:

If the “Use Stage Analysis? = No,” then

\[
f_{bu} = \frac{M_{NC}}{S_{steel}} + \frac{M_{LTC}}{S_{LTC}} + \frac{M_{STC}}{S_{STC}}
\]

where \( M_{NC} \) is the demand moment on the non-composite section, \( M_{LTC} \) is the demand moment on the long-term composite section, and \( M_{STC} \) is the demand moment on the short-term composite section.

The short-term section modulus for positive moment is calculated by transforming the concrete deck using the steel-to-concrete modular ratio. The long-term section modulus for positive moment uses a modular ratio factored by \( n \), where \( n \) is specified in the Design Parameters as the “Modular ratio long-term multiplier.” The effect of compression reinforcement is ignored. For negative moment, the concrete deck is assumed cracked and is not included in the section modulus calculations, while tension reinforcement is accounted for.
If “Use Stage Analysis? = Yes,” then the $f_{bu}$ stresses on each flange are read directly from the section cut results. The stresses are calculated based on gross section and tributary width of the composite slab. The program assumes that the effects of the staging of loads applied to non-composite versus composite sections and the concrete slab material time dependent properties were captured by using the Nonlinear Staged Construction load case available in CSiBridge.

In the Strength Design Check, the program verifies the sign of the stress in the composite slab, and if stress is positive (tension), the program assumes that the entire section cut demand moment is carried by the steel section only. This is to reflect the fact that the concrete in the composite slab is cracked and does not contribute to the resistance of the section.

In Constructability checks, the program proceeds based on the status of the concrete slab. When the slab is not present or is non-composite, the $f_{bu}$ stresses on each flange are read directly from the section cut results. When the slab status is composite, the program verifies the sign of the stress in the composite slab, and if stress is positive (tension), the program assumes that the entire section cut demand moment is carried by the steel section only. This is to reflect the fact that the concrete in the composite slab is cracked and does not contribute to the resistance of the section.

Note that the Design Request for staged constructability check (Steel-I Comp Construct Stgd) allows only Nonlinear Staged Construction load cases to be used as Demand Sets. In that case stresses are calculated based on gross section and tributary width of the composite slab.

### 8.4 Ultimate Design Request

The Strength Design Check calculates at every section cut positive bending capacity, negative bending capacity, shear capacity, positive bending shear interaction, and negative bending shear interaction. It then compares the capacities against the envelope of demands specified in the design request.

#### 8.4.1 Bending

##### 8.4.1.1 Positive Bending

The demand over capacity ratio is evaluated as
DoverC = \frac{M_{Ed}}{M_{pl,Rd}}

### 8.4.1.2 Positive Bending Shear Interaction

When subject to the simultaneous action of shear and moment, transversely stiffened webs that depend on tension field action to carry shear, i.e.,

\[ \frac{h}{w} > 502 \sqrt{\frac{k_y}{F_y}} \]

can be checked for compliance with eq. (c) of Clause 10.10.5.2. The demand over capacity ratio is calculated as follows:

\[ \text{DoverC} = 0.727 \frac{M_f}{M_r} + 0.455 \frac{V_f}{V_r} \]

### 8.4.1.3 Negative Bending – Class 1 and 2

The demand over capacity ratio is evaluated as

\[ \text{DoverC} = \frac{M_f}{M_r} \]

For derivation of moment resistance \( M_r \) of laterally unbraced members see section 8.1.1.6 of this manual.

### 8.4.1.4 Negative Bending Shear Interaction – Class 1 and 2

The criteria when section subject to simultaneous action of shear and moment is checked for compliance with eq. (c) of Clause 10.10.5.2 is similar to that used for positive bending (see section 8.4.1.2 of this manual). The demand over capacity ratio is calculated as follows:

\[ \text{DoverC} = 0.727 \frac{M_f}{M_r} + 0.455 \frac{V_f}{V_r} \]

### 8.4.1.5 Negative Bending – Class 3

For derivation of stresses \( f_{bu} \) in flanges, see Section 8.3.1 of this manual. For derivation of the factored moment resistance \( M_f \) of laterally unbraced flanges see Section 8.1.6 of this manual. The demand over capacity ratio is calculated as follows:
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\[
DoverC = \max \left( \frac{f_{buTop}}{\phi_S F_{yTop}}, \frac{f_{buBot}}{\phi_S M_r}, \frac{f_{rebar}}{\phi_S S} \right)
\]

where \( S \) is the elastic section of steel section with respect to the bottom fiber.

8.4.1.6 Negative Bending Shear Interaction – Class 3

The criteria when section subject to simultaneous action of shear and moment is checked for compliance with eq. (c) of Clause 10.10.5.2 is similar to that used for positive bending (see section 8.4.1.2 of this manual). The demand over capacity ratio is calculated as follows:

\[
DoverC = 0.727 \max \left( \frac{f_{buTop}}{\phi_S F_{yTop}}, \frac{f_{buBot}}{\phi_S M_r}, \frac{f_{rebar}}{\phi_S S} \right) + 0.455 \frac{V_f}{V_r}
\]

8.4.2 Shear

When processing the Design Request from the Design module, the program assumes vertical stiffeners are present at supports only, and no intermediate vertical stiffeners are present.

In the Optimization form (Design/Rating > Superstructure Design > Optimize command), the user can specify stiffener locations. The program recalculates the shear resistance based on the defined stiffener layout. It should be noted that stiffeners are not modeled in the Bridge Object, and therefore, adding/modifying stiffeners does not affect the magnitude of the demands.

8.4.2.1 Design Shear Resistance

The factored shear resistance of the web of a flexural member, \( V_r \), is taken as

\[
V_r = \phi_s A_w F_s
\]

where \( A_w \), the shear area, is calculated as web area, and \( F_s \), the ultimate shear stress, is equal to \( F_{cr} + F_t \), where \( F_{cr} \) and \( F_t \) are taken as follows:
(a) when \( \frac{h}{w} \leq 502 \frac{k_v}{F_y} \):

\[
F_{cr} = 0.577F_y
\]

\[
F_t = 0
\]

(b) when \( 502 \frac{k_v}{F_y} < \frac{h}{w} \leq 621 \frac{k_v}{F_y} \):

\[
F_{cr} = \frac{290 \sqrt{F_y k_v}}{h/w}
\]

\[
F_t = \left[ 0.5F_y - 0.866F_{cr} \right] \left[ \frac{1}{\sqrt{1+(a/h)^2}} \right]
\]

(c) when \( \frac{h}{w} > 621 \frac{k_v}{F_y} \):

\[
F_{cr} = \frac{180,000k_v}{(h/w)^2}
\]

\[
F_t = \left[ 0.5F_y - 0.866F_{cr} \right] \left[ \frac{1}{\sqrt{1+(a/h)^2}} \right]
\]

where

\[
k_v = 4 + \frac{5.34}{(a/h)^2} \quad \text{when } a/h < 1
\]

\[
k_v = 5.34 + \frac{4}{(a/h)^2} \quad \text{when } a/h \geq 1
\]
Where \( a \) = spacing of transverse stiffeners. For unstiffened webs, \( a/h \) is considered infinite, so that \( kv = 5.34 \). The demand over capacity ratio is calculated as follows:

\[
\text{DoverC} = \frac{V_f}{V_r}
\]

### 8.5 Service Stress Design Request

The service design check calculates at every section cut the stresses \( f_{bu} \) at the top steel flange of the composite section and the bottom steel flange of the composite section. It then compares them against limits specified in Clause 10.11.4.

#### 8.5.1 Positive Bending

For the derivation of stresses \( f_{bu} \) in flanges, see Section 8.3.1 of this manual.

The demand over capacity ratio is calculated as follows, in accordance with Clause 10.11.4 (a):

\[
\text{DoverC} = \max \left( \frac{f_{buTop}}{0.9F_{yTop}}, \frac{f_{buBot}}{0.9F_{yBot}} \right)
\]

#### 8.5.2 Negative Bending

For the derivation of stresses \( f_{bu} \) in flanges, see Section 8.3.1 of this manual.

The demand over capacity ratio is calculated as follows, in accordance with Clause 10.11.4 (b):

\[
\text{DoverC} = \max \left( \frac{f_{buTop}}{0.9F_{yTop}}, \frac{f_{buBot}}{0.9F_{yBot}} \right)
\]

### 8.6 Constructability Design Request

#### 8.6.1 Staged (Steel-I Comp Construct Stgd)

This request enables the user to verify the superstructure during construction by using the Nonlinear Staged Construction load case. The nonlinear staged analysis allows the user to define multiple snapshots of the structure during construction, when parts of the bridge deck may be at various completion stages. The user controls which stages the program will include in the calculations of the controlling demand over capacity ratios.
For each section cut specified in the Design Request, the constructability design check loops through the Nonlinear Staged Construction load case output steps that correspond to Output Labels specified in the Demand Set. At each step the program determines the status of the concrete slab at the girder section cut. The slab status can be non present, present non-composite, or composite.

The Staged Constructability design check accepts the following Bridge Object Structural Model Options:

- Area Object Model
- Solid Object Model

The Staged Constructability design check cannot be run on Spine models. The section stresses are calculated based on gross section and tributary width of the composite slab.

The non-staged constructability design request (Steel-I Comp Construct NonStgd) calculates stresses based on effective width of slab.

**8.6.2 Non-Staged (Steel-I Comp Construct NonStgd)**

This request enables the user to verify demand over capacity ratios during construction without the need to define and analyze a Nonlinear Staged Construction load case. For each section cut specified in the design request the constructability design check loops through all combos specified in the Demand Set list. At each combo the program assumes the status of the concrete slab as specified by the user in the Slab Status column. The slab status can be non-composite or composite and applies to all the section cuts.

The Non-Staged Constructability design check accepts all Bridge Object Structural Model Options available in the Update Bridge Structural Model form (Bridge > Update > Structural Model Options option).

**8.6.3 Slab Status vs. Unbraced Length**

Based on the slab status the program calculates corresponding positive bending capacity, negative bending capacity, shear capacity, and positive and negative bending versus shear interaction. Next the program compares the capacities against demands specified in the Demand Set by calculating the Demand over Capacity ratio. The controlling Demand Set and the Output Label on a girder-by-girder basis are reported for every section cut.
When the slab status is composite, the program assumes that the top flange is continuously braced. When the slab status is not present or non-composite, the program treats both flanges as discretely braced. It should be noted that the program does not verify the presence of diaphragms at a particular output step. It assumes that any time a steel beam is activated at a given section cut, the unbraced length $L$ for the bottom flange is equal to the distance between the nearest downstation and upstation qualifying cross diaphragms or the span ends as defined in the Bridge Object. The program assumes the same unbraced length $L$ for the top flange. In other words, the unbraced length $L$ is based on the cross diaphragms that qualify as providing restraint to the bottom flange. Some of the diaphragm types available in CSiBridge may not necessarily provide restraint to the top flange. It is the user’s responsibility to provide top flange temporary bracing at the diaphragm locations prior to the slab acting compositely.

8.6.4 Algorithm

When the slab status is composite, the staged and non-staged design checks follow the procedure outlined in section 8.4 of this manual. When the slab status is non-composite, the section resisting demands consist of the steel beam only. Because of the fact that the top flange is not continuously braced, the section is reclassified. In addition to the checks described in section 8.4 of this manual, the top flange is also checked for lateral torsional buckling.

8.7 Section Optimization

After at least one Steel Design Request has been successfully processed, CSiBridge enables the user to open a Steel Section Optimization module. The Optimization module allows interactive modification of steel plate sizes and definition of vertical stiffeners along each girder and span. It recalculates resistance “on the fly” based on the modified section without the need to unlock the model and rerun the analysis. It should be noted that in the optimization process the demands are not recalculated and are based on the current CSiBridge analysis results.

The Optimization form allows simultaneous display of three versions of the section sizes and associated resistance results. The section plate size versions are “As Analyzed,” “As Designed,” and “Current.” The section plots use distinct colors for each version – black for As Analyzed, blue for As Designed, and red for Current. When the Optimization form is initially opened, all three versions are identical and equal to “As Analyzed.”
Two graphs are available to display various forces, moments, stresses, and ratios for the As Analyzed or As Designed versions. The values plotted can be controlled by clicking the “Select Series to Plot” button. The As Analyzed series is plotted as solid lines and the As Designed series as dashed lines.

To modify steel plate sizes or vertical stiffeners, a new form can be displayed by clicking the Modify Section button. After the section modification has been completed, the Current version is shown in red in the elevation and cross-section views. After the resistance has been recalculated successfully by clicking the Recalculate Resistance button, the Current version is designated As Designed and displayed in blue.

After the section optimization has been completed, the As Designed plate sizes and materials can be applied to the analysis bridge object by clicking the OK button. The button opens a new form that can be used to Unlock the existing model (in that case all analysis results will be deleted) or save the file under a new name (New File button). Clicking the Exit button does not apply the new plate sizes to the bridge object and keeps the model locked. The As Designed version of the plate sizes will be available the next time the form is opened, and the Current version is discarded. The previously defined stiffeners can be recalled in the Steel Beam Section Variation form by clicking the Copy/Reset/Recall button in the top menu of the form. The form can be displayed by clicking on the Modify Section button.
This chapter describes the algorithms CSiBridge applies when designing steel U-tub with composite slab superstructures in accordance with the CAN/CSA S14 code (Section 10).

9.1 Section Properties

9.1.1 Yield Moments

9.1.1.1 Composite Section in Positive Flexure

The depth of web in compression that is used in section classification is derived based on positive yield moment, $M_y$. The positive yield moment is determined by the program using the following user-defined input, which is part of the Design Request (see Chapter 4 for more information about Design Requests).

$M_{dc} = \text{The user specifies in the Design Request the name of the combo that represents the moment caused by the permanent load applied before the concrete deck has hardened or is made composite.}$

$M_{dc} = \text{The user specifies in the Design Request the name of the combo that represents the moment caused by the remainder of the permanent load (applied to the composite section).}$
The program solves for $M_{AD}$ from the following equation,

$$F_{sy} = \frac{M_{dnc}}{S_{NC}} + \frac{M_{dc}}{S_{LT}} + \frac{M_{AD}}{S_{ST}}$$

and then calculates yield moment based on the following equation

$$M_y = M_{dnc} + M_{dc} + M_{AD}$$

where

- $S_{NC}$ = Noncomposite section modulus
- $S_{LT}$ = Long-term composite section modulus
- $S_{ST}$ = Short-term composite section modulus

$M_y$ is taken as the lesser value calculated for the compression flange, $M_{yc}$, or the tension flange, $M_{yt}$. The positive $M_y$ is calculated only once based on $M_{dnc}$ and $M_{dc}$ demands specified by the user in the Design Request. It should be noted that the $M_y$ calculated in the procedure described here is used by the program to determine only the depth of web in compression that is used in classification of webs in accordance with CAN/CSA S14 Table 10.3 for positive bending in the Design Check.

Since for Staged and Non-Staged Constructability Design Checks it is difficult to obtain built-up elastic stresses, for the sake of classification of the web, it is assumed that the depth of the web in compression for positive bending is based on all stresses being applied to non-composite sections because this produces the greatest depth of web in compression.

### 9.1.1.2 Composite Section in Negative Flexure

For composite sections in negative flexure, the procedure described for positive yield moment is followed, except that the composite section for both short-term and long-term moments consists of the steel section and the longitudinal reinforcement within the tributary width of the concrete deck. Thus, $S_{ST}$ and $S_{LT}$ are the same value. Also, $M_{yt}$ is taken with respect to either the tension flange or the longitudinal reinforcement, whichever yields first. The concrete tension capacity is ignored.
Chapter 9 - Design Steel U-Tub Bridge with Composite Slab

For the sake of classification of the web, the depth of the web in compression for negative bending is based on all stresses being applied to the composite section because this produces the greatest depth of web in compression. This assumption applies to all design checks.

9.1.2 Plastic Moments

9.1.2.1 Composite Section in Positive Flexure Class 1 and 2

The positive plastic moment, \( M_{pl, Rd} \), is calculated as the moment of the plastic forces about the plastic neutral axis. Plastic forces in the steel portions of a cross-section are calculated using the yield strengths of the flanges, the web, and reinforcing steel, as appropriate. The plastic force in the effective width of the composite slab that is in compression is based on a rectangular stress block with the magnitude of the compressive stress equal to \( \phi_s \alpha_1 f'c \), where \( \alpha_1 \) is ratio of average stress in rectangular compression stress block to the specified concrete strength, taken as 0.85 – 0.0015\( f'c \) but not less than 0.67. The effective slab width is determined in accordance with Clause 5.8.2.1. The effect of concrete in tension is neglected. The effective width of bottom flange in tension is determined in accordance with Clause 10.12.2. The position of the plastic neutral axis is determined by the equilibrium condition such that there is no net axial force.

The plastic moment of a composite section in positive flexure is determined as follows:

- Calculate the element forces and use them to determine if the plastic neutral axis is in the web, top flange, or concrete deck.
- Calculate the location of the plastic neutral axis within the element determined in the first step.
- Calculate \( M_{pl, Ra} \).

Equations for the various potential locations of the plastic neutral axis (PNA) are given in Table 9-1.
### Table 9-1 Calculation of PNA and $M_p$ for Sections in Positive Flexure

<table>
<thead>
<tr>
<th>Case</th>
<th>PNA Condition</th>
<th>$\bar{Y}$ and $M_p$</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>In Web $P_t + P_w \geq P_s + P_r + P_o$</td>
<td>$\bar{Y} = \left(\frac{D}{2}\right) \left(\frac{P_t - P_s - P_r - P_o}{P_s}\right) + 1$</td>
</tr>
<tr>
<td>II</td>
<td>In Top Flange $P_t + P_w \geq P_s + P_r + P_o$</td>
<td>$\bar{Y} = \left(\frac{t_s}{2}\right) \left(\frac{P_t - P_s - P_r - P_o}{P_s}\right) + 1$</td>
</tr>
<tr>
<td>III</td>
<td>Concrete Deck Below $P_{rb}$ $P_t + P_w \geq (\frac{c_{rb}}{t_s}) (P_t + P_r + P_o)$</td>
<td>$\bar{Y} = (t_s) \left(\frac{P_t - P_s - P_r - P_o}{P_t}\right)$</td>
</tr>
<tr>
<td>IV</td>
<td>Concrete Deck at $P_{rb}$ $P_t + P_w + P_r + P_o \geq P_c$</td>
<td>$\bar{Y} = c_{rb}$</td>
</tr>
<tr>
<td>V</td>
<td>Concrete Deck Above $P_{rb}$ and Below $P_{rt}$ $P_t + P_w \geq (\frac{c_{rt}}{t_r}) (P_t + P_o)$</td>
<td>$\bar{Y} = (t_r) \left(\frac{P_r + P_o + P_t - P_s}{P_t}\right)$</td>
</tr>
<tr>
<td>VI</td>
<td>Concrete Deck at $P_{rt}$ $P_t + P_w + P_r + P_o \geq P_c$</td>
<td>$\bar{Y} = c_{rt}$</td>
</tr>
<tr>
<td>VII</td>
<td>Concrete Deck Above $P_{rt}$ $P_t + P_w + P_r + P_o + P_s &lt; P_c$</td>
<td>$\bar{Y} = (t_r) \left(\frac{P_r + P_o + P_t + P_c}{P_t}\right)$</td>
</tr>
</tbody>
</table>
in which

\[ P_{rt} = F_{yr} A_{rt} \]
\[ P_s = \alpha_1 \phi_s f'_{c} b_t t_s \]
\[ P_{rb} = F_{yrb} A_{rb} \]
\[ P_c = 2 F_{yc} b_t c \]
\[ P_w = \frac{(2 F_{yw} D t_w)}{\cos \alpha_{web}} \]
\[ P_t = F_{yt} b_t t \text{ where } b_t \text{ is effective width of bottom flange} \]

### 9.1.2.2 Composite Section in Positive Flexure Class 3

For composite sections in which the depth of the compression portion of the web of the steel section, calculated on the basis of a fully plastic stress distribution, exceeds \(850 w \sqrt{F_y}\), the factored moment resistance, \(M_r\), of the composite section is calculated on the basis of fully plastic stress blocks, as shown in Figure 9-2, as follows,

\[ M_r = C_c e_c + C_r e_r + C_s e_s \]

where

\[ C_c = \alpha_1 \phi_c b_t c' f'_{c} \]
\[ C_r = \phi_r A_r f_y \]
\[ C_s = \phi_s A'_{sc} F_y \]
The area of the steel section in compression, $A'_{sc}$, includes the top flanges and a webs area of \((850w^2)\sqrt{F_y}\), and the area of the steel section in tension, $A'_{st}$, is calculated as follows:

$$A'_{st} = \frac{C_c + C_r + C_y}{\phi_y F_y}$$

**Figure 9-2 Class 3 Section in Positive Moment Regions**

### 9.1.2.3 Composite Section in Negative Flexure

The plastic moment of a composite section in negative flexure is calculated by an analogous procedure. Equations for the two cases most likely to occur in practice are given in Table 9-2. The plastic moment of a noncomposite section is calculated by eliminating the terms pertaining to the concrete deck and longitudinal reinforcement from the equations for composite sections.
Table 9-2 Calculation of PNA and $M_p$ for Sections in Negative Flexure

<table>
<thead>
<tr>
<th>Case</th>
<th>PNA Condition</th>
<th>$\bar{Y}$ and $M_p$</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>In Web $P_c + P_w + P_t + P_{rb} + P_n$</td>
<td>$\bar{Y} = \left( \frac{D}{2} \right) \left[ \frac{P_c - P_t - P_{rb} - P_n}{P_n} + 1 \right]$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$M_p = \frac{P_n}{2D} \left[ \bar{Y}^2 + (D - \bar{Y})^2 \right] + \left[ P_w d_n + P_{rb} d_{rb} + P_t d_t + P_l d_l \right]$</td>
</tr>
<tr>
<td>II</td>
<td>In Top Flange $P_c + P_w + P_t + P_{rb} + P_n$</td>
<td>$\bar{Y} = \left( \frac{t_l}{2} \right) \left[ \frac{P_c - P_t - P_{rb} - P_n}{P_t} + 1 \right]$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$M_p = \frac{P_t}{2t_l} \left[ \bar{Y}^2 + \left( \frac{t_l - \bar{Y}}{2} \right)^2 \right] + \left[ P_w d_n + P_{rb} d_{rb} + P_t d_t + P_l d_l \right]$</td>
</tr>
</tbody>
</table>

![Diagram](image)

**Figure 9-3 Plastic Neutral Axis Cases – Negative Flexure**

in which

- $P_{rt} = F_{yrt} A_{rt}$
- $P_t = 0$
- $P_{rb} = F_{yrb} A_{rb}$
- $P_c = F_{yc} b_c t_c$
- $P_w = (2 F_{yw} D t_w)/\cos \alpha_{web}$
- $P_t = 2 F_{yt} b_t t_t$

In the equations for $M_p$, $d$ is the distance from an element force to the plastic neutral axis. Element forces act at (a) mid-thickness for the flanges and the
9.1.3 Classification of Cross-Sections

At each section cut the steel beam section is classified in accordance with Clause 10.9.2. The classification is carried out separately for positive and negative bending for both composite and non-composite sections. The classification of a cross-section depends on the width to thickness ratio of the parts subject to compression. A cross-section is classified according to the highest (least favorable) class of its compression parts.

For calculating the limiting width-to-thickness ratios of the web of monosymmetric sections, \( h \) in Table 10.3 is replaced by \( 2d_c \).

9.1.3.1 Composite Positive Bending

The resistance of the top flanges is assumed as not being limited by its local buckling resistance since they are restrained by effective attachment to a concrete slab by shear connectors. The top flanges are always classified as Class 1.

When classifying the webs, it is first assumed that the section satisfies requirements for Class 1 or 2, and the depth of web in compression is based on the plastic range of the composite section for positive moment. When the web does not satisfy requirements for Class 1 or 2 the section is classified as Class 3. In the next step, the web is verified for Class 3, where the depth of web in compression is based on positive yield moment. See Section 9.1.1.1 of this manual for derivation of the yield moment for positive bending of a composite section. When the web does not satisfy requirements for Class 3, the section is classified as Class 4.

The bottom flange is always in tension and therefore does not have an effect on the classification of the section.

9.1.3.2 Non-Composite Positive Bending

The top flanges are in compression and are not restrained by the composite slab. Their resistance may be limited by local buckling resistance. The flanges
are classified in accordance with Clause 10.9.2 as a part subject to compression.

When classifying the webs, it is first assumed that the section satisfies requirements for Class 1 or 2, and the depth of the web in compression is based on the plastic range of the steel beam section for positive moment. When the web does not satisfy requirements for Class 1 or 2, the section is classified as Class 3. In the next step, the web is verified for Class 3, where the depth of the web in compression is based on the neutral axis of the steel beam. When the web does not satisfy requirements for Class 3, the section is classified as Class 4.

The bottom slab classification follows the same procedure as is outlined in Section 9.1.3.1 of this manual.

**9.1.3.3 Composite Negative Bending**

The top flange is always in tension and therefore does not have an effect on the classification of the section.

When classifying the web, it is first assumed that the section satisfies requirements for Class 1 or 2, and the depth of web in compression is based on the plastic range for negative moment. When the web does not satisfy requirements for Class 1 or 2, the section is classified as Class 3. In the next step, the web is verified for Class 3, where the depth of the web in compression is based on the negative yield moment. See Section 9.1.1.2 of this manual for derivation of the yield moment for negative bending of a composite section. When the web does not satisfy requirements for Class 3, the section is classified as Class 4.

The bottom flange is in compression and unrestrained. The bottom flange resistance may be limited by its local buckling resistance and is classified in accordance with Clause 10.9.2 as a part subject to compression.

**9.1.3.4 Non-Composite Negative Bending**

The classification of the top and bottom flanges follows the same procedure as outlined in Section 9.1.3.3 of this manual.

When classifying the web, it is first assumed that the section satisfies requirements for Class 1 or 2, and the depth of the web in compression is based
on the plastic range of the steel beam for negative moment. When the web does not satisfy requirements for Class 1 or 2, the section is classified as Class 3. In the next step, the web is verified for Class 3, where the depth of the web in compression is based on the position of the neutral axis of the steel beam. When the web does not satisfy requirements for Class 3, the section is classified as Class 4.

9.1.4 Class 4 Sections
Sections classified as Class 4 are flagged as invalid and no results are reported.

9.1.5 Unbraced Length L and Section Transitions
The program assumes that the top flanges are continuously braced for all Design Requests, except Constructability when slab status is equal to ‘Non-composite. For more information on flange lateral bracing in the Constructability Design Requests, see Section 9.6.3 of this manual.

When slab status is equal to ‘Non-composite’ the unbraced length $L$ for the top flange is equal to the distance between the nearest downstation and the upstation qualifying cross diaphragms or span supports, as defined in the Bridge Object. Some of the diaphragm types available in CSiBridge may not necessarily provide restraint to the top flange. The program assumes that the following diaphragm qualifies as providing lateral restraint to the top flange: single beam and all types of chords and braces, except V braces without bottom beams.

The program calculates demands and capacities pertaining to a given section cut at a given station without considering the section transition within the unbraced length. It does not search for the highest demands versus the smallest resistance within the unbraced length as the code suggests. It is the responsibility of the user to pay special attention to section transition within unbraced lengths and to follow the guidelines in the code.

9.1.6 Open-Top Box Lateral-Torsional Stability
When slab status is equal to ‘Non-composite’ the program calculates buckling verification at every section cut.

From Clause 10.10.2.3. the factored moment resistance $M_r$ is calculated as
(a) \[ M_r = 1.15 \phi_s M_p \left[ 1 - \frac{0.28 M_u}{M_p} \right] \leq \phi_s M_p, \text{ when } M_u > 0.67 M_p; \text{ or} \]

(b) \[ M_r = \phi_s M_u, \text{ when } M_u \leq 0.67 M_p \]

The critical elastic moment, \( M_u \), of a monosymmetric section is taken as

\[
M_u = \frac{6\alpha \pi}{L} \left[ \sqrt{E_s I_y G_s J} \right] \left[ B_1 + \sqrt{1 + B_2 + B_3^2} \right]
\]

where

\[
B_1 = \frac{\beta_s}{2L} \sqrt{\frac{E_s I_y}{G_s J}}
\]

where

\[
\beta_s = \text{ coefficient of monosymmetry}
\]

\[
B_2 = \frac{\pi^2 E_s C_w}{L^2 G_s J}
\]

---

Section Properties 9 - 11
Figure 9-4 Properties of open-top box girder for lateral-torsional stability calculations

\[ A_{fb} = b_b t_b \]
\[ A_{ft} = b_t t_t \]
\[ A_w = d_w t_w \]
\[ d_w = \sqrt{d^2 + s^2} \]
\[ s = \frac{B_1 - B_2}{2} \]
\[ e' = \left[ B_2 d \left( \frac{A_{ft} B_1}{2} + \frac{A_w}{6} \left( B_1 + \frac{B_2}{2} \right) \right) - \frac{d_1 t_w A_{ft}}{6} \right] \frac{1}{I_y} \]
\[ e = -(e' + y_b) + t_b \]

\((e\) is positive only if \(S\) is between \(C\) and the compression flange\)

Torsion constant

\[ J = \frac{1}{3} (2b_1 t_i^2 + 2d_w t_w^3 + b_b t_b^3) \]

Warping constant

\[ C_w = \frac{2}{3} \left[ \frac{K_f^2 A_{fb}}{2} + (K_f^2 + K_2^2 - K_1 K_2) A_w + (4K_2^2 + (K_2 - K_3)^2) \right] \]

Coefficient of monosymmetry

\[ \beta_x = \frac{1}{l_x} \int_A y(x^2 + y^2) dA + 2e \]

where

\[ K_1 = \frac{e^{\text{e}b_z}}{2} \]

\[ K_2 = \frac{dB_z}{2} - s(e' + d + t_b) - K_1 \]

\[ K_3 = \frac{b_t}{2} (e' + d + t_b) \]

\[ k_3 = \frac{\beta_x}{2} \]

\[ \int_A y(x^2 + y^2) dA = y_b A_{fb} \left( \frac{b_b^2}{12} + y_b^2 \right) - y_t A_{ft} \left( \frac{b_t^2}{2} + \frac{b_b^2}{6} + 2y_t^2 \right) + t_w \left[ c^2 (y_b^2 - y_t^2) - \frac{4cs}{3d} (y_b^3 + y_t^3) + \frac{1}{2} \left( 1 + \left( \frac{s}{d} \right)^2 \right) (y_b^4 - y_t^4) \right] \]

\[ c = \frac{b_b}{2} + \frac{y_b s}{d} \]
When the design request parameter ‘Method for determining $\omega_2$’ is set to ‘Program Determined’, then for each demand set the applied bending moments $M_a$, $M_b$, $M_c$ and $M_{\text{max}}$ at the unbraced segment are determined by interpolation of demands at nearest section cuts. The designer should be aware that live load moments at neighboring section cuts within the unbraced segment are not necessarily controlled by the same load pattern and as a result the moment gradient calculation may be impacted.

### 9.1.7 Unstiffened Compression Flanges

The program assumes that the bottom flanges are always unstiffened. The factored moment resistance of unstiffened compression flanges is determined in accordance with Clause 10.13.7.4.2.2 as follows:

$$ R_v = \sqrt{1 - 3 \left(\frac{f_s}{F_y}\right)^2} $$

a) When the torsional shear stress $f_s \leq \frac{0.75F_y}{\sqrt{3}}$, the factored moment resistance, $M_r$, shall be taken as

i) $M_r = \phi_s R_v F_y S'$ for $\frac{b_s}{t} \leq \frac{R_1}{F_y}$

ii) $M_r = \phi_s F_y \left( R_v - 0.4 \right) + 0.4s \sin \left( \frac{c_s \pi}{2} \right) S'$ for $\frac{R_1}{F_y} < \frac{b_s}{t} \leq \frac{R_2}{F_y}$

iii) $M_r = \phi_s F_{cr} S'$ for $\frac{b_s}{t} > \frac{R_2}{F_y}$

b) When $0.75 \frac{F_y}{\sqrt{3}} < f_s \leq \frac{F_y}{\sqrt{3}}$ and $\frac{b_s}{t} \leq \frac{R_1}{F_y}$, the factored moment resistance, $M_r$, shall be taken as $M_r = \phi_s R_v F_y S'$ where
\[
R_1 = \frac{225\sqrt{k_1}}{\sqrt{\frac{1}{2} R_v + \left[ R_v^2 + 4 \left( \frac{f_s}{f_y} \right)^2 \left[ \frac{k_1}{k_s} \right]^2 \right]^{0.5}}}
\]
\[
R_2 = \frac{550\sqrt{k_1}}{\sqrt{\frac{1}{1.2} (R_v - 0.4) + \left[ (R_v - 0.4)^2 + 4 \left( \frac{f_s}{f_y} \right)^2 \left[ \frac{k_1}{k_s} \right]^2 \right]^{0.5}}}
\]
\[
C_s = \frac{R_2 - \frac{p_2}{t} \sqrt{F_y}}{R_2 - R_1}
\]
\[
F_{cr} = \frac{18k_1 \times 10^4}{\left[ \frac{p_2}{t} \right]^2} - \frac{f_s^2 k_1 \left[ \frac{p_2}{t} \right]^2}{18k_s^2 \times 10^4}
\]

Where \( k_1 = 4 \), \( k_s = 5.34 \) and \( b_s = b \) = width of bottom flange between webs.

The coexisting shear stress due to torsion \( f_s \) in the bottom flange is calculated as:

\[
f_s = \frac{T}{2A_o t_b}
\]

Where \( A_o \) is enclosed area within the box section.

### 9.2 Design Request Parameters

The following Design Request parameters are available for user control:

**Highway class** – Highway Class per clause 1.4.2.2; Default Value = A, Typical value(s): A, B, C, D. The classification is used to determine \( F \) and \( C_f \) factors

- \( \phi \) steel flexure, resistance factor for steel in flexure, default value = 0.95
- \( \phi \) steel shear, resistance factor for steel in shear, default value = 0.95
- \( \phi \) concrete, resistance factor for concrete, default value = 0.75
ϕ rebar, resistance factor for reinforcement, default value = 0.9

Use Stage Analysis to determine stresses on composite section, Yes or No

Modular ratio \( n = \frac{E_s}{E_c} \) is the multiplier used to determine long-term composite section properties, default value = 3.0.

Omega2 Specification – method to determine coefficient to account for increased moment resistance of a laterally unsupported beam segment when subject to a moment gradient – Program or User Determined

Omega2 – user defined value of \( \omega_2 \) coefficient to account for increased moment resistance of a laterally unsupported beam segment when subject to a moment gradient

9.3 Demand Sets

Demand Set combos (at least one required) are user-defined combinations based on CAN/CSA S14 combinations (see Chapter 4 for more information about specifying Demand Sets). The demands from all specified demand combos are enveloped and used to calculate D/C ratios. The way the demands are used depends on if the design parameter "Use Stage Analysis?" is set to Yes or No.

If “Use Stage Analysis? = Yes,” the program reads the stresses on beams and slabs directly from the section cut results. The stresses are calculated using the gross section, which is derived from the tributary area of the composite slab and gross section properties of the steel U tub. The use of effective section properties cannot be accommodated with this option. To design sections where gross and effective areas are not equal, the design parameter "Use Stage Analysis?" should be set to No.

When “Use Stage Analysis? = Yes,” the program assumes that the effects of the staging of loads applied to non-composite versus composite sections and the concrete slab material time dependent properties were captured by using the Nonlinear Staged Construction load case available in CSiBridge.

If “Use Stage Analysis? = No,” the program decomposes load cases present in every demand set combo to three Bridge Design Action categories: non-composite, composite long term, and composite short term. The program uses
the load case Bridge Design Action parameter to assign the load cases to the appropriate categories. A default Bridge Design Action parameter is assigned to a load case based on its Design Type. However, the parameter can be overwritten: click the Analysis > Load Cases > {Type} > New command to display the Load Case Data – {Type} form; click the Design button next to the Load case type drop down list, under the heading Bridge Design Action select the User Defined option and select a value from the list. The assigned Bridge Designed Action values are handled by the program in the following manner:

### Table 9-3 Bridge Design Action

<table>
<thead>
<tr>
<th>Bridge Design Action Value specified by the user</th>
<th>Bridge Design Action Category used in the design algorithm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Non-Composite</td>
<td>Non-Composite</td>
</tr>
<tr>
<td>Long-Term Composite</td>
<td>Long-Term Composite</td>
</tr>
<tr>
<td>Short-Term Composite</td>
<td>Short-Term Composite</td>
</tr>
<tr>
<td>Staged</td>
<td>Non-Composite</td>
</tr>
<tr>
<td>Other</td>
<td>Non-Composite</td>
</tr>
</tbody>
</table>

#### 9.3.1 Demand Flange Stresses \( f_{bu} \)

Evaluation of the flange stress, \( f_{bu} \), is dependent on setting the Design Request parameter “Use Stage Analysis?":

If the “Use Stage Analysis? = No,” then

\[
f_{bu} = \frac{M_{NC}}{S_{steel}} + \frac{M_{LTC}}{S_{LTC}} + \frac{M_{STC}}{S_{STC}}
\]

where \( M_{NC} \) is the demand moment on the non-composite section, \( M_{LTC} \) is the demand moment on the long-term composite section, and \( M_{STC} \) is the demand moment on the short-term composite section.

The short-term section modulus for positive moment is calculated by transforming the effective area of concrete deck using the steel-to-concrete modular ratio. The long-term section modulus for positive moment uses a modular ratio factored by \( n \), where \( n \) is specified in the Design Parameters as the “Modular ratio long-term multiplier.” The effect of compression reinforcement is ig-
nored. For negative moment, the concrete deck is assumed cracked and is not included in the section modulus calculations, while tension reinforcement is accounted for.

If “Use Stage Analysis? = Yes,” then the $f_{nu}$ stresses on each flange are read directly from the section cut results. The stresses are calculated based on gross section and tributary width of the composite slab; the use of effective section properties cannot be accommodated with this option. To design sections where gross and effective areas are not equal, the design parameter “Use Stage Analysis?” should be set to No. The program assumes that the effects of shear lag, of the staging of loads applied to non-composite versus composite sections, and the concrete slab material time dependent properties were captured by using the Nonlinear Staged Construction load case available in CSiBridge.

In the Strength Design Check, the program verifies the sign of the stress in the composite slab, and if stress is positive (tension), the program assumes that the entire section cut demand moment is carried by the steel section only. This is to reflect the fact that the concrete in the composite slab is cracked and does not contribute to the resistance of the section.

In Constructability checks, the program proceeds based on the status of the concrete slab. When the slab is not present or is non-composite, the $f_{nu}$ stresses on each flange are read directly from the section cut results. When the slab status is composite, the program verifies the sign of the stress in the composite slab, and if stress is positive (tension), the program assumes that the entire section cut demand moment is carried by the steel section only. This is to reflect the fact that the concrete in the composite slab is cracked and does not contribute to the resistance of the section.

Note that the Design Request for staged constructability check (Steel-U Comp Construct Stgd) allows only Nonlinear Staged Construction load cases to be used as Demand Sets. In that case stresses are calculated based on gross section; the use of effective section properties cannot be accommodated for this type of Design Request. To design sections where gross and effective areas are not equal, the non-staged constructability Design Request (Steel-U Comp Construct NonStgd) can be used.
9.4 **Ultimate Design Request**

The Strength Design Check calculates at every section cut positive bending capacity, negative bending capacity, shear capacity, positive bending shear interaction, and negative bending shear interaction. It then compares the capacities against the envelope of demands specified in the Design Request.

**9.4.1 Bending**

**9.4.1.1 Positive Bending**

The factored moment resistance is based on fully plastic stress distribution while accounting for the effective width of the bottom flange in tension as determined in accordance with Clause 10.12.2 and the yield strength $F_Y$ of the bottom flange being reduced per Clause 10.13.7.4.1 to account for shear stress due to torsion. The demand over capacity ratio is evaluated as

$$DoverC = \frac{M_f}{M_r}$$

**9.4.1.2 Positive Bending Shear Interaction**

When subject to the simultaneous action of shear and moment, transversely stiffened webs that depend on tension field action to carry shear, i.e.,

$$\frac{h}{w} > 502\sqrt{\frac{k_c}{F_y}}$$

are checked for compliance with eq. (c) of Clause 10.10.5.2. The demand over capacity ratio is calculated as follows:

$$DoverC = 0.727 \frac{M_f}{M_r} + 0.455 \frac{V_t + V_{tor}}{V_r}$$

For evaluation of torsional shear $V_{tor}$ see Section 9.4.2 of this manual.

**9.4.1.3 Negative Bending – Class 1 and 2**

The demand over capacity ratio is evaluated as

$$DoverC = \frac{M_f}{M_r}$$
For derivation of moment resistance $M_r$ of unstiffened compression flanges see Section 9.1.7 of this manual.

### 9.4.1.4 Negative Bending Shear Interaction – Class 1 and 2

When a section subject to the simultaneous action of shear and moment is checked for compliance with eq. (c) of Clause 10.10.5.2, the performance criteria are similar to those used for positive bending (see section 9.4.1.2 of this manual). The demand over capacity ratio is calculated as follows:

$$DoverC = 0.727 \frac{M_f}{M_r} + 0.455 \frac{V_f + V_{tor}}{V_r}$$

For evaluation of torsional shear $V_{tor}$ see Section 9.4.2 of this manual.

### 9.4.1.5 Negative Bending – Class 3

For derivation of stresses $f_{bu}$ in flanges, see Section 9.3.1 of this manual. For derivation of moment resistance $M_r$ of unstiffened compression flanges see Section 9.1.7 of this manual. The demand over capacity ratio is calculated as follows:

$$DoverC = \max \left( \frac{f_{buTop}}{\phi_s f_{yTop}}, \frac{f_{buBot}}{\phi_s M_r}, \frac{f_{rebar}}{\phi_s S} \right)$$

where $S$ is the elastic section of a steel section with respect to the bottom fiber.

### 9.4.1.6 Negative Bending Shear Interaction – Class 3

When a section subject to the simultaneous action of shear and moment is checked for compliance with eq. (c) of Clause 10.10.5.2, the performance criteria are similar to those used for positive bending (see Section 9.4.1.2 of this manual). For evaluation of torsional shear $V_{tor}$ see Section 9.4.2 of this manual. The demand over capacity ratio is calculated as follows:

$$DoverC = 0.727 \max \left( \frac{f_{buTop}}{\phi_s f_{yTop}}, \frac{f_{buBot}}{\phi_s M_r} \right) + 0.455 \frac{V_f + V_{tor}}{V_r}$$
9.4.2 Shear

When processing the Design Request from the Design module, the program assumes vertical stiffeners are present at supports only, and no intermediate vertical stiffeners are present.

In the Optimization form (Design/Rating > Superstructure Design > Optimize command), the user can specify stiffener locations. The program recalculates the shear resistance based on the defined stiffener layout. It should be noted that stiffeners are not modeled in the Bridge Object, and therefore, adding/modifying stiffeners does not affect the magnitude of the demands.

9.4.2.1 Design Shear Resistance

The factored shear resistance of the web of a flexural member, $V_r$, is taken as

$$V_r = \phi_s A_w F_s$$

where $A_w$, the shear area, is calculated as web area, and $F_s$, the ultimate shear stress, is equal to $F_{cr} + F_t$, where $F_{cr}$ and $F_t$ are taken as follows:

(a) when $\frac{h}{w} \leq 502 \frac{k_v}{F_y}$,

$$F_{cr} = 0.577 F_y$$

$$F_t = 0$$

(b) when $502 \frac{k_v}{F_y} < \frac{h}{w} \leq 621 \frac{k_v}{F_y}$,

$$F_{cr} = \frac{290 \sqrt{F_y k_v}}{h/w}$$

$$F_t = \left[ 0.5 F_y - 0.866 F_{cr} \right] \frac{1}{\sqrt{1 + (a/h)^2}}$$
(c) when \( \frac{h}{w} > 621 \sqrt[2]{\frac{k_v}{F_y}} \),

\[ F_{cr} = \frac{180000k_y}{(h/w)^2} \]

\[ F_t = \left[ 0.5F_y - 0.866F_{cr} \right] \left[ \frac{1}{\sqrt{1+(a/h)^2}} \right] \]

where,

\[ k_v = \begin{cases} 4 + \frac{5.34}{(a/h)^2} & \text{when } a/h < 1 \\ 5.34 + \frac{4}{(a/h)^2} & \text{when } a/h \geq 1 \end{cases} \]

where \( a = \) spacing of transverse stiffeners. For unstiffened webs, \( a/h \) is considered infinite, so that \( k_v = 5.34 \).

The St. Venant torsional shear stress in the web is determined as:

\[ f_s = \frac{T}{2A_o \ell_w} \]

Where \( A_o \) is enclosed area within the box section. Shear force per web resulting from the torsional stress is evaluated as:

\[ V_{tor} = f_o A_{web} \]

The contribution from the flanges and composite slab is always ignored. The demand to capacity ratio is evaluated as:

\[ DoverC = \frac{V_f + V_{tor}}{V_f} \]
9.5 **Service Stress Design Request**

The service design check calculates at every section cut the stresses $f_{bu}$ at the top steel flange of the composite section and the bottom steel flange of the composite section. It then compares them against limits specified in Clause 10.11.4.

9.5.1 **Positive Bending**

For the derivation of stresses $f_{bu}$ in flanges, see Section 9.3.1 of this manual.

The demand over capacity ratio is calculated as follows, in accordance with Clause 10.11.4 (a):

$$DoverC = \max \left( \frac{f_{bustop}}{0.9 F_{ytop}}, \frac{f_{bu BOT}}{0.9 F_{ybot}} \right)$$

9.5.2 **Negative Bending**

For the derivation of stresses $f_{bu}$ in flanges, see Section 9.3.1 of this manual.

The demand over capacity ratio is calculated as follows, in accordance with Clause 10.11.4 (b):

$$DoverC = \max \left( \frac{f_{bustop}}{0.9 F_{ytop}}, \frac{f_{bu BOT}}{0.9 F_{ybot}} \right)$$

9.6 **Constructability Design Request**

9.6.1 **Staged (Steel-U Comp Construct Stgd)**

This request enables the user to verify the superstructure during construction by using the Nonlinear Staged Construction load case. The nonlinear staged analysis allows the user to define multiple snapshots of the structure during construction, when parts of the bridge deck may be at various completion stages. The user controls which stages the program will include in the calculations of the controlling demand over capacity ratios.
For each section cut specified in the Design Request, the constructability design check loops through the Nonlinear Staged Construction load case output steps that correspond to Output Labels specified in the Demand Set. At each step the program determines the status of the concrete slab at the girder section cut. The slab status can be non-present, present non-composite, or composite.

The Staged Constructability design check accepts the following Bridge Object Structural Model Options:

- Area Object Model
- Solid Object Model

The Staged Constructability design check cannot be run on Spine models. The section stresses are calculated based on gross section and tributary width of the composite slab.

The section stresses are calculated based on gross section; the use of effective section properties cannot be accommodated for this design request. To design sections where gross and effective areas are not equal, the non-staged constructability design request (Steel-U Comp Construct NonStgd) can be used.

### 9.6.2 Non-Staged (Steel-U Comp Construct NonStgd)

This request enables the user to verify demand over capacity ratios during construction without the need to define and analyze a Nonlinear Staged Construction load case. For each section cut specified in the design request the constructability design check loops through all combos specified in the Demand Set list. At each combo the program assumes the status of the concrete slab as specified by the user in the Slab Status column. The slab status can be non-composite or composite and applies to all the section cuts.

The Non-Staged Constructability design check accepts all Bridge Object Structural Model Options available in the Update Bridge Structural Model form (Bridge > Update > Structural Model Options option).

### 9.6.3 Slab Status vs. Unbraced Length

Based on the slab status the program calculates corresponding positive bending capacity, negative bending capacity, shear capacity, and positive and negative bending versus shear interaction. Next the program compares the capacities
against demands specified in the Demand Set by calculating the Demand over Capacity ratio. The controlling Demand Set and the Output Label on a girder-by-girder basis are reported for every section cut.

When the slab status is composite, the program assumes that the top flange is continuously braced. When the slab status is not present or non-composite, the program treats the top flanges as discretely braced. It should be noted that the program does not verify the presence of diaphragms at a particular output step. It assumes that any time a steel beam is activated at a given section cut, the unbraced length \( L \) for the top flanges is equal to the distance between the nearest downstation and upstation qualifying cross diaphragms or the span ends as defined in the Bridge Object. Some of the diaphragm types available in CSiBridge may not necessarily provide restraint to the top flange. It is the user’s responsibility to provide top flange temporary bracing at the diaphragm locations prior to the slab acting compositely.

9.6.4 Algorithm

When the slab status is composite, the staged and non-staged design checks follow the procedure outlined in Section 9.4 of this manual. When the slab status is non-composite, the section resisting demands consists of the steel beam only. Because of the fact that the top flange is not continuously braced, the section is reclassified. In addition to the checks described in Section 9.4 of this manual, the top flange is also checked for lateral torsional buckling.

9.7 Section Optimization

After at least one Steel Design Request has been successfully processed, CSiBridge enables the user to open a Steel Section Optimization module. The Optimization module allows interactive modification of steel plate sizes and definition of vertical stiffeners along each girder and span. It recalculates resistance “on the fly” based on the modified section without the need to unlock the model and rerun the analysis. It should be noted that in the optimization process the demands are not recalculated and are based on the current CSiBridge analysis results.

The Optimization form allows simultaneous display of three versions of the section sizes and associated resistance results. The section plate size versions are “As Analyzed,” “As Designed,” and “Current.” The section plots use
distinct colors for each version – black for As Analyzed, blue for As Designed, and red for Current. When the Optimization form is initially opened, all three versions are identical and equal to “As Analyzed.”

Two graphs are available to display various forces, moments, stresses, and ratios for the As Analyzed or As Designed versions. The values plotted can be controlled by clicking the “Select Series to Plot” button. The As Analyzed series is plotted as solid lines and the As Designed series as dashed lines.

To modify steel plate sizes or vertical stiffeners, a new form can be displayed by clicking the Modify Section button. After the section modification has been completed, the Current version is shown in red in the elevation and cross-section views. After the resistance has been recalculated successfully by clicking the Recalculate Resistance button, the Current version is designated As Designed and displayed in blue.

After the section optimization has been completed, the As Designed plate sizes and materials can be applied to the analysis bridge object by clicking the OK button. The button opens a new form that can be used to Unlock the existing model (in that case all analysis results will be deleted) or save the file under a new name (New File button). Clicking the Exit button does not apply the new plate sizes to the bridge object and keeps the model locked. The As Designed version of the plate sizes will be available the next time the form is opened, and the Current version is discarded. The previously defined stiffeners can be recalled in the Steel Beam Section Variation form by clicking the Copy/Reset/Recall button in the top menu of the form. The form can be displayed by clicking on the Modify Section button.
Chapter 10
Run a Bridge Design Request

This chapter identifies the steps involved in running a Bridge Design Request. (Chapter 4 explains how to define the Request.) Running the Request applies the following to the specified Bridge Object:

- Program defaults in accordance with the selected code—the Preferences
- Type of design to be performed—the check type (Section 4.2.1)
- Portion of the bridge to be designed—the station ranges (Section 4.1.3)
- Overwrites of the Preferences—the Design Request parameters (Section 4.1.4)
- Load combinations—the demand sets (Chapter 2)
- Live Load Distribution factors, where applicable (Chapter 3)

For this example, the AASHTO LRFD 2007 code is applied to the model of a concrete box-girder bridge shown in Figure 10-1.

It is assumed that the user is familiar with the steps that are necessary to create a CSiBridge model of a concrete box girder bridge. If additional assistance is needed to create the model, a 30-minute Watch and Learn™ video entitled, "Bridge – Bridge Information Modeler" is available at the CSI website www.csiamerica.com. The tutorial video guides the user through the creation of the bridge model referenced in this chapter.
10.1 Description of Example Model

The example bridge is a two-span prestressed concrete box girder bridge with the following features:

Abutments: The abutments are skewed by 15 degrees and connected to the bottom of the box girder only.

Prestress: The concrete box girder bridge is prestressed with four 10-in² tendons (one in each girder) and a jacking force of 2160 kips per tendon.

Bents: The one interior bent has three 5-foot-square columns.

Deck: The concrete box girder has a nominal depth of 5 feet. The deck has a parabolic variation in depth from 5 feet at the abutments to a maximum of 10 feet at the interior bent support.

Spans: The two spans are each approximately 100 feet long.
10.2 Design Preferences

Use the Design/Rating > Superstructure Design > Preferences command to select the AASHTO LRFD 2007 design code. The Bridge Design Preferences form shown in Figure 10-4 displays.

![Bridge Design Preferences form](image)

10.3 Load Combinations

For this example, the default design load combinations were activated using the Design/Rating > Load Combinations > Add Defaults command. After the Bridge Design option has been selected, the Code-Generated Load Combinations for Bridge Design form shown in Figure 10-5 displays. The form is used to
specify the desired limit states. Only the Strength II limit state was selected for this example. Normally, several limit states would be selected.

![Code-Generated Load Combinations for Bridge Design form](image1)

**Figure 10-5 Code-Generated Load Combinations for Bridge Design form**

The defined load combinations for this example are shown in Figure 10-6.

![Define Load Combinations form](image2)

**Figure 10-6 Define Load Combinations form**
Chapter 10 - Run a Bridge Design Request

The Str-II1, Str-II2 and StrIIGroup1 designations for the load combinations are specified by the program and indicate that the limit state for the combinations is Strength Level II.

10.4 Bridge Design Request

After the Design/Rating > Superstructure Design > Design Request command has been used, the Bridge Design Request form shown in Figure 10-7 displays.

The name given to this example Design Request is FLEX_1, the Check Type is for Concrete Box Flexure and the Demand Set, DSet1, specifies the combination as StrII (Strength Level II).

The only Design Request Parameter option for a Concrete Box Flexural check type is for PhiC. A value of 0.9 for PhiC is used.
10.5 Start Design/Check of the Bridge

After an analysis has been run, the bridge model is ready for a design/check. Use the Design/Rating > Superstructure Design > Run Super command to start the design process. Select the design to be run using the Perform Bridge Design form shown in Figure 10-8:

The user may select the desired Design Request(s) and click on the Design Now button. A plot of the bridge model, similar to that shown in Figure 10-9, will display.

If several Design Requests have been run, the individual Design Requests can be selected from the Design Check options drop-down list. This plot is described further in Chapter 11.

Figure 10-8 Perform Bridge Design - Superstructure

Figure 10-9 Plot of flexure check results
Chapter 11
Display Bridge Design Results

Bridge design results can be displayed on screen and as printed output. The on-screen display can depict the bridge response graphically as a plot or in data tables. The Advanced Report Writer can be used to create the printed output, which can include the graphical display as well as the database tables.

This chapter displays the results for the example used in Chapter 10. The model is a concrete box girder bridge and the code applied is AASHTO LRFD 2007. Creation of the model is shown in a 30-minute Watch and Learn™ video on the CSI website, www.csiamerica.com.

11.1 Display Results as a Plot

To view the forces, stresses, and design results graphically, click the Home > Display > Show Bridge Superstructure Design Results command, which will display the Bridge Object Response Display form shown in Figure 11-1.

The plot shows the design results for the FLEX_1 Design Request created using the process described in the preceding chapters. The demand moments are enveloped and shown in the blue region, and the negative capacity moments are shown with a brown line. If the demand moments do not exceed the capacity moments, the superstructure may be deemed adequate in response to the flexure Design Request. Move the mouse pointer onto the demand or capacity plot to view the values for each nodal point. Move the pointer to the capacity moment at station 1200
536981.722 kip-in is shown. A verification calculation that shows agreement with this CSiBridge result is provided in Section 11.4.

11.1.1 Additional Display Examples

Use the Home > Display > Show Bridge Forces/Stresses command to select, on the example form shown in Figure 11-2, the location along the top or bottom portions of a beam or slab for which stresses are to be displayed. Figures 11-3 through 11-9 illustrate the left, middle, and right portions as they apply to Multicell Concrete Box Sections. Location 1, as an example, refers to the top left selection option while location 5 would refer to the bottom center selection option. Locations 1, 2, and 3 refer to the top left, top center, and top right selection option while locations 4, 5, and 6 refer to the bottom left, bottom center, and bottom right selection options.

Figure 11-1 Plot of flexure check results for the example bridge design model

11 - 2 Display Results as a Plot
Figure 11-2 Select the location on the beam or slab for which results are to be displayed

Figure 11-3 Bridge Concrete Box Deck Section - External Girders Vertical
Figure 11-4 Bridge Concrete Box Deck Section - External Girders Sloped

Figure 11-5 Bridge Concrete Box Deck Section - External Girders Clipped

Display Results as a Plot
Chapter 11 - Display Bridge Design Results

Figure 11-6 Bridge Concrete Box Deck Section - External Girders and Radius

Figure 11-7 Bridge Concrete Box Deck Section - External Girders Sloped Max

Display Results as a Plot 11-5
Figure 11-8 Bridge Concrete Box Deck Section - Advanced

Figure 11-9 Bridge Concrete Box Deck Section - AASHTO - PCI - ASBI Standard

11 - 6 Display Results as a Plot
11.2 Display Data Tables

To view design results on screen in tables, click the Home > Display > Show Tables command, which will display the Choose Tables for Display form shown in Figure 11-10. Use the options on that form to select which data results are to be viewed. Multiple selections may be made.

When all selections have been made, click the OK button and a database table similar to that shown in Figure 11-11 will display. Note the drop-down list in the upper right-hand corner of the table. That drop-down list will include the various data tables that match the selections made on the Choose Tables for Display form. Select from that list to change to a different database table.
Figure 11-11 Design database table for AASHTO LRFD 2007 flexure check

The scroll bar along the bottom of the form can be used to scroll to the right to view additional data columns.

11.3 Advanced Report Writer

The File > Report > Create Report command is a single button click output option but it may not be suitable for bridge structures because of the size of the document that is generated. Instead, the Advanced Report Writer feature within CSiBridge is a simple and easy way to produce a custom output report.

To create a custom report that includes input and output, first export the files using one of the File > Export commands: Access; Excel; or Text. When this command is executed, a form similar to that shown in Figure 11-12 displays.
Chapter 11 - Display Bridge Design Results

Figure 11-12 Choose Tables for Export to Access form

This important step allows control over the size of the report to be generated. Export only those tables to be included in the final report. However, it is possible to export larger quantities of data and then use the Advanced Report Writer to select only specific data sets for individual reports, thus creating multiple smaller reports. For this example, only the Bridge Data (input) and Concrete Box Flexure design (output) are exported.

After the data tables have been exported and saved to an appropriate location, click the File > Report > Advanced Report Writer command to display a form similar to that show in Figure 11-13. Click the appropriate button (e.g., Find existing DB File, Convert Excel File, Convert Text File) and locate the exported data tables. The tables within that Database, Excel, or Text file will be listed in the List of Tables in Current Database File display box.
Select the tables to be included in the report from that display box. The selected items will then display in the Items Included in Report display box. Use the various options on the form to control the order in which the selected tables appear in the report as well as the headers (i.e., Section names), page breaks, pictures, and blanks required for final output in .rft, .txt, or .html format.

After the tables have been selected and the headers, pictures, and other formatting items have been addressed, click the **Create Report** button to generate the report. The program will request a filename and the path to be used to store the report. Figure 11-14 shows an example of the printed output generated by the Report Writer.
11.4 Verification

As a verification check of the design results, the output at station 1200 is examined. The following output for negative bending has been pulled from the ConBoxFlexure data table, a portion of which is shown in Figure 11-10:

- Demand moment, “DemandMax” (kip-in) = \(-245973.481\)
- Resisting moment, “ResistingNeg” (kip-in) = 536981.722
- Total area of prestressing steel, “AreaPTTop” (in²) = 20.0
- Top \(k\) factor, “kFactorTop” = 0.2644444
- Neutral axis depth, \(c\), “CDistForNeg” (in) = 5.1286
- Effective stress in prestressing, \(f_p\), “EqFpsForNeg” (kip/in²) = 266.7879

A hand calculation that verifies the results follows:

- For top \(k\) factor, from eq. 5.7.3.1.1-2,

\[
k = 2 \left( 1.04 - \frac{f_{ps}}{f_{pu}} \right) = 2 \left( 1.04 - \frac{245.1}{270} \right) = 0.26444 \text{ (Results match)}
\]
For neutral axis depth, from (AASHTO LRFD eq. 5.7.3.1.1-4),

\[
\begin{align*}
    c &= \frac{A_{PT} f_{PU} - 0.85 f'_c \left( b_{slab} - b_{webeq} \right) t_{slabeq}}{0.85 f'_c \beta b_{webeq} + k A_{PT} \frac{f_{PU}}{Y_{PT}}} \quad \text{for a T-section} \\
    c &= \frac{A_{PT} f_{PU}}{0.85 f'_c \beta b_{webeq} + k A_{PT} \frac{f_{PU}}{Y_{PT}}} \quad \text{when not a T-section} \\
    c &= \frac{20.0(270)}{0.85(4)(0.85)(360) + 0.26444(20)(\frac{270}{114})} = 5.1286 \text{ (Results match)}
\end{align*}
\]

For effective stress in prestressing, from (AASHTO LRFD eq. 5.7.3.1.1-1),

\[
\begin{align*}
    f_{PS} &= f_{PU} \left( 1 - k \frac{c}{Y_{PT}} \right) = 270 \left( 1 - 0.26444 \frac{5.1286}{144} \right) = 266.788 \text{ (Results match)}
\end{align*}
\]

For resisting moment, from (AASHTO LRFD eq. 5.7.3.2.2-1),

\[
\begin{align*}
    M_N &= A_{PT} f_{PS} \left( Y_{PT} - \frac{c \beta_1}{2} \right) + 0.85 f'_c \left( b_{SLAB} - b_{webeq} \right) t_{slabeq} \left( \frac{c \beta_1}{2} - \frac{t_{slabeq}}{2} \right) \\
    M_N &= A_{PT} f_{PS} \left( Y_{PT} - \frac{c \beta_1}{2} \right), \text{ when the box section is not a T-section} \\
    M_N &= 20.0(266.788) \left( 144 - \frac{5.1286(0.85)}{2} \right) = 596646.5 \text{ kip-in} \\
    M_R &= \phi M_N = 0.85(596646.5) = 536981.8 \text{ kip-in (Results match)}
\end{align*}
\]

The preceding calculations are a check of the flexure design output. Other design results for concrete box stress, concrete box shear, and concrete box principal have not been included. The user is encouraged to perform a similar check of these designs and to review Chapters 5, 6, and 7 for detailed descriptions of the design algorithms.
Bibliography

ACI, 2007. Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary (ACI 318R-08), American Concrete Institute, P.O. Box 9094, Farmington Hills, Michigan.


