

CT-Steel Method to Solution

CT-Steel Beta Version 3.0

Document Revisions:

Changes in blue: Draft 8/1/2024.

Changes in orange: Draft 12/13/2024 (Revised for Version 3)

This document is currently under development. Please contact Christopher.Patria@ct.gov for questions or issues.

Section 1 Force Effects

1.1 Force Effect Interpolation

CT-Steel will use linear interpolation to populate Force Effects at stations where the user did not input a force effect entry.

CT-Steel will not extrapolate force effects that are missing beyond the extreme stations provided by the user. For an example, if the user provided Force Effects entries for stations 6 ft to 126 ft, CT-Steel will not extrapolate missing force effect stations down-station of 6 ft or up-station of 126 ft. Generally, CT-Steel will encounter a runtime error if a force effect is not applied at the start station of the first span and end station of the last span for each load case.

The LRE should evaluate the "density" of stations to provide to CT-Steel to achieve the desired degree of accuracy.

1.2 Flange Lateral Bending

CT-Steel will amplify flange lateral bending moments of compression flanges to account for second order effects in accordance with the BDS.

1.3 Handling of Live Load Cases

Exclusive of the fatigue evaluation, CT-Steel handles and rates each live load case independently. A live load case is assigned to a vehicle by the user in the live load case sheet. This live load case will not interact with any other live load cases assigned to that vehicle. Each live load case will be rated with all applicable rating procedures. Any rating function which uses a live loading case will use the force effects provided under that the live load case. For example, when determining the moment gradient modifier, the live load moments at various stations along the unbraced range are pulled from the same live load case.

The user can use this functionality to consider concurrent live loading when computing the moment gradient modifier in lieu of using the extreme moment envelopes.

1.4 Adjacent Vehicles

Adjacent vehicle live load force effects are entered by the user. The adjacent vehicle live load will be summed with the dead load force effects, applied on the short-term section, throughout calculations.

In the SpecCheck files, the factored adjacent live load will be combined with the dead load force effects.

For adjacent vehicles which have been assigned a negative live load factor, CT-Steel will sum the adjacent vehicle force effect with the rating vehicle force effect. Note, in this case, it will not be summed with the dead load force effect as described above and the absolute value of the adjacent vehicle load factor will be taken.

No adjustments will be applied to the adjacent vehicle force effects which decrease the extreme rating vehicle effect, i.e., the full adjacent vehicle force will be taken in this case.

1.5 Handling of Fatigue Live Loads

When determining the stress range, CT-Steel will select the extreme positive and extreme negative moment effect at each station from any live load case which have been assigned to the same Vehicle on the live load cases tab.

1.6 Dead Load Combinations

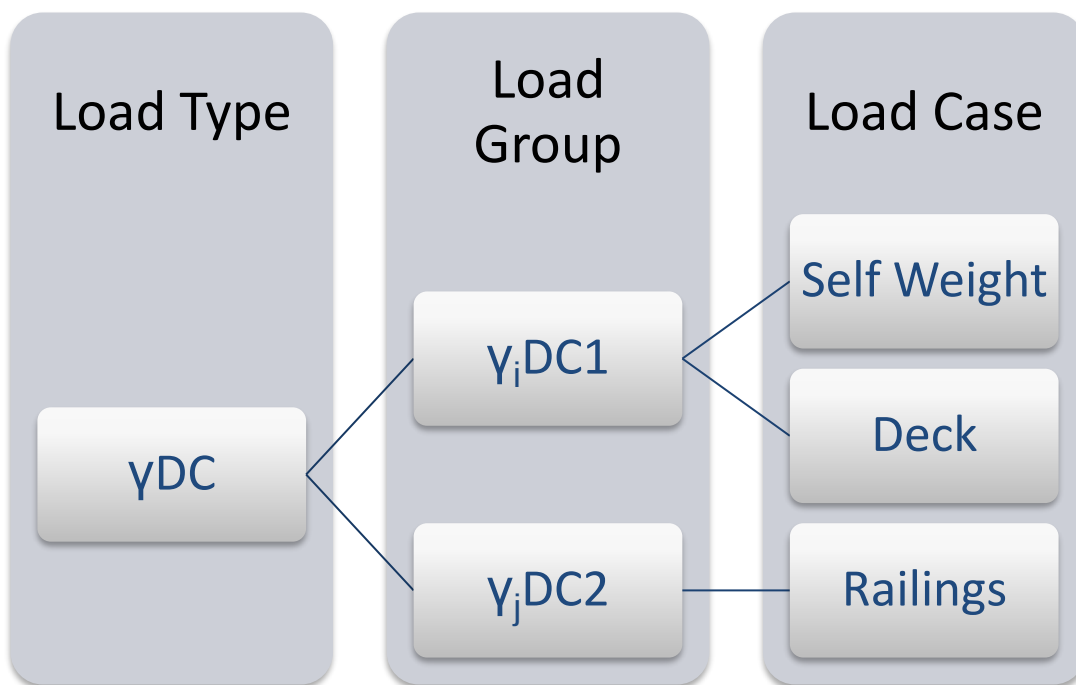
1.6.1 *Min & Max Load Factors*

CT-Steel will use the minimum and maximum load factors provided by the user to produce the extreme dead load force effect for the rating calculation under consideration.

Case	Dead Load Factor
$\frac{DL}{LL} < 0$	$\gamma_{DL,min}$
$\frac{DL}{LL} > 0$	$\gamma_{DL,max}$

1.6.2 Factoring of Dead Load Cases

CT-Steel will group all dead load cases with the same dead load type and stage and apply the same dead load factor (i.e., the minimum or maximum load factor). The below schematic illustrates that the load cases are summed to a load group. The load groups are then each applied the min or max load factor.



Section 2 Analysis Sections

2.1 Points of Interests (POI)

CT-Steel will perform rating calculations at the stations following Stations, on the Left Section and Right Section.

- 10th Points along the span
- Point Brace Point
- Start and End Station of the Following Range Inputs
- Web
- Top or Bottom Flange
- Top of Bottom Cover Plate
- Top of Bottom Side Plate
- Section Loss to any of the above
- Effective Flange
- Composite
- Slab Reinforcement
- Haunch
- Transverse Stiffeners
- Longitudinal Stiffeners
- Discrete Bracing
- Lateral (Continuous) Bracing
- Horizontal Alignment Curvature
- Fatigue Details
- Control Option Ranges
- Capacity Overrides
- Hinges & Simple Support Locations
- Mid-Point between Brace Points
- Quarter Points between Brace Points*
- User Defined Stations

Note the Left station at the beginning of the girder and Right station at the end of the girder are omitted. Similarly, the left and right stations are omitted at Gap locations in the span.

* New in V2.10 for AISC Cb calculations.

2.2 Station Tolerance

CT-Steel will round the stations inputs to the "dimTolerance" control option. Any section change occurring within that tolerance will be taken at the same cross-section.

2.3 Stages

Various CT-Steel member inputs require the User to define the stage at which that element is activated. Elastic stresses are computed only on the active elements in each stage, i.e., elements with an active stage less than or equal to the loading stage.

The live load stage is automatically set after the maximum stage an element is assigned, e.g., deck slab hardening stage, cover plate stage and the maximum dead load stage assigned. Reinforcing steel is automatically applied to the same stage as the deck hardening stage.

2.4 Section Symmetry

Double and Single Symmetry of the section is determined by comparing the flange area and flange lateral moment of inertia. A 0.1% tolerance is applied when checking for equality. The flange, cover plate, and side plates at the final stage are considered in the computation.

2.5 Longitudinal Stiffeners

2.5.1 Section Properties

~~The current version of~~ CT-Steel ~~does not~~ will consider longitudinal stiffeners when determining the elastic or plastic section properties of the section ~~if the user enables this option in the longitudinal stiffener definition input~~

2.5.2 Longitudinal Stiffener Depth

If multiple longitudinal stiffeners existing within the web depth, CT-Steel will iterate through each stiffener to select the stiffener which produces the largest resistance, e.g., largest value for the web-bend buckling coefficient or web-load shedding factor.

2.5.3 Transverse Stiffener Spacing

CT-Steel will disregard longitudinal stiffeners at stations where the transverse stiffener spacing is greater than 2D.

2.5.4 Stiffener Rigidity

Longitudinal Stiffeners shall be prevented from Locally Buckling, BDS 6.10.11.3.2, and Laterally Buckling, BDS 6.10.11.3.3. Longitudinal Stiffeners not meeting these requirements will not be considered effective.

The Control Option "TreatAsLongStiffened" will disable CT-Steels checking of the stiffener for local buckling and lateral buckling as discussed in the above paragraph and assume that the longitudinal stiffener is adequately proportioned to be considered as a longitudinal stiffener in all rating calculations. The intention of this control option is allow the LRE to employ reasonable judgement to longitudinal stiffeners not wholly complying to the latest edition of the BDS. Currently, there is no mechanism in CT-Steel to check to see if a longitudinal stiffener is only slightly non-conforming.

2.5.5 Stiffener Effectiveness Factor, R_b

The user can enter a longitudinal stiffener effectiveness factor for each longitudinal stiffener definition. This factor is used to determine the web load shedding factor used in the rating calculations. The effectiveness factor is used as an interpolation between the an unstiffened web and stiffened web. Note that, if the stiffener is already considered ineffective, as discussed in other sections, this factor will have no effect on the evaluation.

$$R_b = R_{b,u}(1 - \eta) + \eta * R_{b,s}$$

Where:

- R_b = Web loading shedding factor used in rating calculations
- $R_{b,u}$ = Web load shedding factor based on unstiffened web
- $R_{b,s}$ = Web load shedding factor based on stiffened web
- η = Effectiveness factor entered by the user.

2.6 Prismatic Unbraced Lengths

CT-Steel will determine if a section change exists if the major axis moment of inertia, sectional area, position of the neutral axis, or flange lateral moment of inertia changes at any Point of Interest within the unbraced length, considering the structural steel elements at the short-term/live load stage.

The below was removed in version 3.

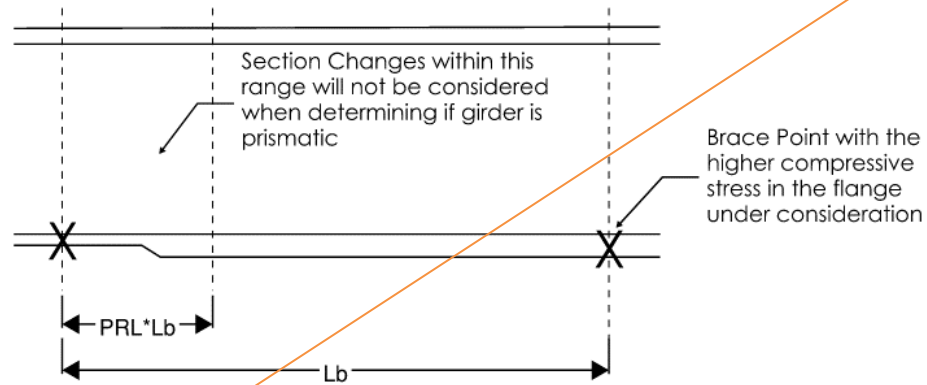
How strict the change of dimension is applied is controlled by the ~~isPrismaticTolerance~~ control option. If any section meets the following inequality, the unbraced range will be considered non-prismatic.

$$\frac{|d_{\perp} - d_{\perp}|}{d_{\perp}} * 100\% \geq T$$

Where:

- $d_{\perp}$ = ~~Dimension of any element of the first cross-section of the unbraced range beyond the PrismaticRangeLimit.~~
- $d_{\perp}$ = ~~Dimension of the corresponding element at any other cross-section beyond the PrismaticRangeLimit.~~
- T = ~~isPrismaticTolerance as percentage.~~

~~Sections at the end of the unbraced range with the smallest compression stress within the PrismaticRangeLimit is not considered when determining if the member is prismatic.~~



Where:

X = Brace Point

PRL = Prismatic Range Limit control option

L_b = Unbraced Length

2.7 Unbraced Lengths

Locations where the top flange transitions from continuous braced to discretely braced, a top flange brace point will be added at that transitions point by the Engine during runtime.

Section 3 Iterative Rating

3.1 General

For any capacity related calculation which is a function of a live load force effect, the live load force effect will be multiplied by the rating factor.

During the initial iteration (itera = 0) the rating factor is taken as 1 when applied to the live load force effect in capacity related calculations and a rating factor is computed. In subsequent iterations, the previously computed rating factor is used to multiply the live load force effect. The iterations will stop once two consecutive iterations compute the same rating factor (percent error $\leq 0.001\%$) or the max number of iterations is exceeded set by the control option.

If the max number of iterations is exceeded CT-Steel will return the minimum rating factor of the last two iterations. In all cases, the specCheck output will contain data based on the last iteration, except for the rating factor field. These specCheck rows are identified by a negative iteration value.

3.1.1 Convergence Strategy

3.1.1.1 Oscillation

Most ratings converge within 4 iterations or will never converge and oscillate between two ratings. For example, an iteration with a trial RF = 13.6 returns a RF = 56.4 and the subsequent iteration with a trial RF = 56.4 returns a RF = 13.6. If Oscillation is detected at the 4 iteration, CT-Steel will split the difference between the previous two rating factors to disrupt the oscillation, and if rating continues to oscillate the minimum the iterations will end and the smallest of the last two iterations will be outputted, and the iteration marked as -2.

Section 4 Moment Resistance

4.1 Compactness

CT-Steel will consider the section compact if all the following criteria are satisfied.

- Curved = FALSE (User Input on Bridge worksheet)
- Kinked = FALSE (User Input on Bridge worksheet)
- Flange and cover plate material strength ≤ 70 ksi
- Web proportion limits satisfied*
- Web compact limits satisfied

*This criteria is optionally considered by a hidden Control Option. Contact CTDOT if the user would like to disable consideration of this criteria.

4.2 Composite Sections in Position Flexure

CT-Steel will use the plastic moment resistance if all the following criteria are satisfied.

- Section is Compact, see 4.1
- allowPlastic Control Option = TRUE

The user should use the allow plastic control option to disable consideration of the plastic moment capacity for any other code, e.g., holes in the tension flange, criteria or other reason.

4.3 Composite Sections in Negative Flexure and Non-Composite Sections

CT-Steel will use Appendix A6 if all of the following criteria are satisfied.

- Flange, cover plate, and side plate material strength ≤ 70 ksi
- Compression to Tension Flange lateral moment of inertia ratio limit per A6
- Web is nonslender
- allowA6 Control Option = TRUE

The user should use the allowA6 control option to disable consideration of the Appendix A6 for any other code criteria, e.g., curved girder, kinked girder, high-skewed framing, non-contiguous bracing, or other reasons.

4.4 Discretely Braced Compression Flanges

This also includes built up members with angle flanges, cover plates, and side plates.

4.4.1 *Lateral Torsional Buckling*

4.4.1.1 *Flange Radius of Gyration, r_t*

AASHTO's equation for the radius of gyration of the compression is only valid for welded and rolled I-Sections without cover plates, CT-Steel uses the following modified equation to compute the radius of gyration for the compression flange for sections with angle flanges, cover plates, and side plates. Note this equation simplifies to the AASHTO equation for regular rectangular flanges. This method is consistent with AISC and the definition of the term.

$$r_t = \sqrt{\frac{I_f + I_p + I_s}{A_f + A_p + A_s + \frac{1}{3}A_{wc}}}$$

Derivation of this equation:

$$r_t = \sqrt{\frac{I_f}{A_f + \frac{1}{3}A_{DC}}} = \sqrt{\frac{t_f \times b_f^3}{12 \left(t_f \times b_f + \frac{1}{3}D_c \times t_w \right)}} = \sqrt{\frac{b_f^2}{12 \left(1 + \frac{1}{3} \frac{D_c \times t_w}{t_f \times b_f} \right)}} = \frac{b_f}{\sqrt{12 + \frac{1}{3} \frac{D_c \times t_w}{t_f \times b_f}}}$$

4.4.1.2 Compressive Stress for LTB

The compressive stress for LTB will be taken at the governing section for the flange element, i.e., flange, cover plate, side plate, at first yield of the flange under consideration.

~~CT Steel computes the total compression force of all the compression flange elements summed for all dead load stages and divides the total compression force by the total flange area to determine the total dead load stress in the flange. This needed to capture the effects of existing structures reinforced under load. If the section has not been reinforced under load this procedure simplifies to the AASHTO equations.~~

$$f = \frac{\sum f_{i,max} \times A_i}{A_E}$$

Where:

f = Compression stress used in LTB evaluation

$f_{i,max}$ = ~~Maximum factored compress of the compression flange elements at i stage of construction.~~

A_i = Total flange area at i stage of construction.

A_E = Total flange area at final dead load stage.

4.4.1.3 Elastic Section Modulus to the Compression Flange for LTB

The elastic section modulus to the compression flange will be taken at the flange element at first yield.

~~CT Steel will use an equivalent Elastic Section Modulus to the Compression Flange, S_{xe} in LTB calculations to be consistent with the method used to determine the stress in the compression flange. MAD and F_{yf} are substituted with the following.~~

$$P_{fa} = \sum f_{(i,max)a} \times A_i$$

$$M_{AD} = (P_{yf} - P_{fa}) \frac{S_{ST}}{A_E}$$

$$F_{yf} = \frac{P_{yf}}{A_E}$$

Where:

~~M_{AD}~~ = Compression stress used in LTB evaluation

~~F_{yf}~~ = Maximum factored compress of the compression flange elements at i stage of construction.

~~A_t~~ =
$$P_{fd} = \sum f_{(t, \max) d} \times A_t$$

~~A_t~~ = Total flange area at final dead load stage.

4.4.2 Local Buckling

The following elements are rated in the local buckling evaluation.

- Horizontal legs of flanges
- Cover Plates

Local buckling of cover plates are checked according to AISC table B4.1b.3. Stresses in each element shall be rated against the capacity derived from the b/t of those elements.

Additionally, slender flanges webs should not be computed with BDS Eq. 6.10.8.2.2-1. BDS proportions restrict a flange from being designed as slender. However, in rating situations can occur where slender compression flanges exist. Since neither the BDS, nor the MBE provide a methodology for determining the limiting stress of a slender flange, AISC standard specifications must be applied.

To smooth the transition from compact, non-compact resistance, computed using 6.10.8 provisions, to the slender resistance computed by AISC, k_c must be taken equal to 0.35 when using AISC equation F5-9. See BDS C6.10.8.2.2.

4.5 Moment Gradient Modifier, C_b

4.5.1 Non-Prismatic Members

The moment gradient modifier is set equal to 1 for non-prismatic sections. Note that the CT-Steel considers the section non-prismatic even if the section change to a smaller flange is within 20% of the unbraced range.

4.5.2 Unbraced Cantilever

Sections applied the Unbraced Cantilever Control option will limited to a maximum value of $C_b = 1.0$.

4.5.3 AISC LTB

Option removed in version 3.

~~When the C_b method control option is set to the AISC method. The Kirby and Nethercot equation will be used as modified by the R_m factor for singly symmetric sections in reverse curvature bending. If the top flange is continuously braced and unbraced range is subject to reverse curvature bending the Yura equations will be used.~~

~~The moment gradient modifier will be limited to a maximum value of 3 when the AISC method for computing the moment gradient factor is enabled.~~

~~The moment gradient modifier will be computed using the moment gradient instead of the flange stress gradient.~~

4.6 A6 LTB Resistance and Demands

~~The Lateral Torsional Buckling resistance for the section under consideration are rated with the moment demands computed by multiplying the maximum compressive stress in the unbraced range by the section modulus to the compression, S_{xc} , of the section under consideration. Each section within the unbraced is rated and reported.~~

4.7 LTB Parameters for Nonprismatic Unbraced Length

4.7.1 Method A

4.7.1.1 Unbraced Cantilevers

C_{be} will be taken limited to a maximum value for 1.0 for unbraced cantilevers.

4.7.1.2 Splice Plates

Splice plates are currently not entered into CT-Steel, therefore, the contribution is not considered. The user should

4.7.1.3

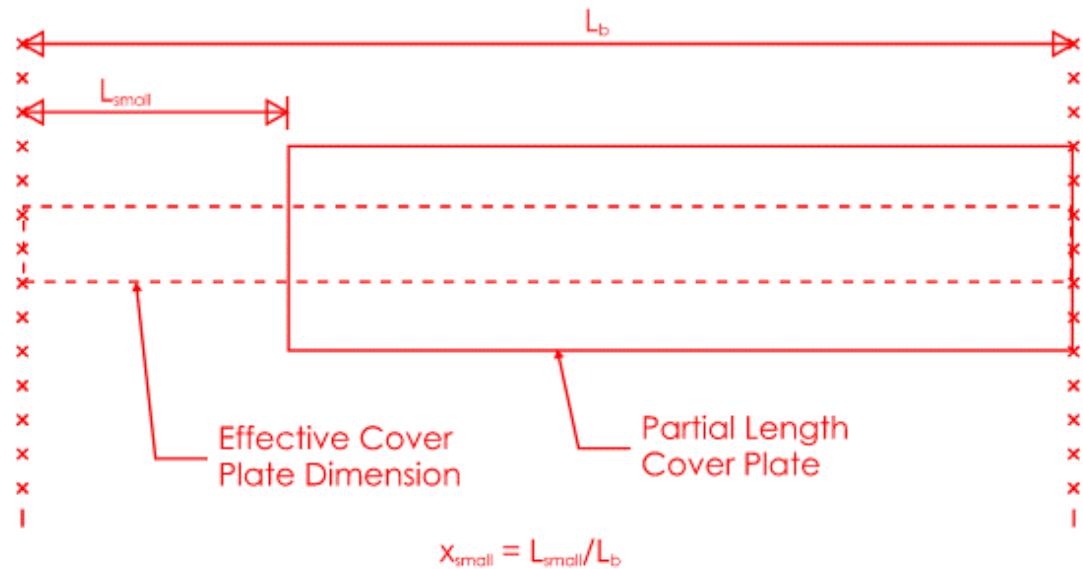
4.7.2 Method B

4.7.2.1 Effective Section

The effective section is determined in accordance with Section D6.6.3 of the 10th edition ballot item. CT-Steel handles the following situations

Partial Length Cover or Side Plates:

If cover plates are terminated within the unbraced length, the small dimension is taken as zero and the portion of the small dimension within the unbraced length is taken as the length without of the cover plate, as illustrated in the below graphic.



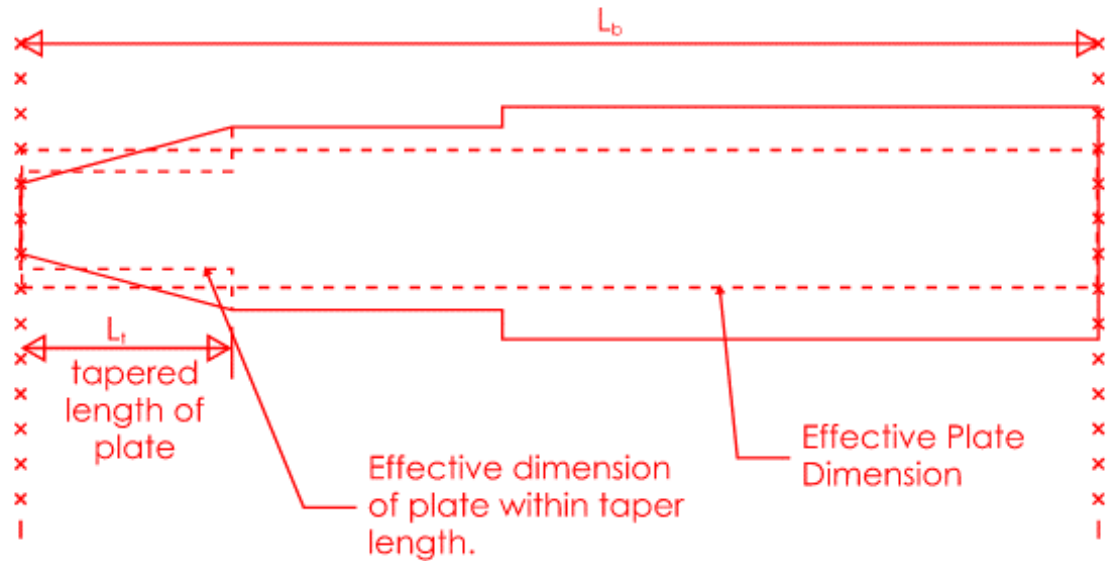
Vertical Offsets

For the purposes of determine elastic section properties, the vertical offset of elements in the cross-section, e.g., flanges, cover plates, side plates, and haunch heights are taken by apply the general form of the effective dimension equations in BDS D6.6.3-1, with the exponent taken as 2, and each section vertical offset measure from the nearest edge of the web.

Elements with varying dimensions:

For elements of a varying dimension, such as a tapered cover plate, tapered dimensions are converted to an effective constant dimension for the tapered length prior to computing the effective dimension within the unbraced length, as

illustrated in the below graphic.



Slab and Reinforcing Steel:

Slab and reinforcing steel used to determine effective section modulus are taken at the governing section.

Longitudinal Stiffeners:

Longitudinal stiffeners are taken at the location of the governing section. The depth of the longitudinal stiffener on the web as expressed in the below equation.

$$d_{s,eff} = d_{s,gov} \frac{D_{eff}}{D_{gov}}$$

Where:

- $d_{s,eff}$ = Effective depth of the longitudinal stiffener
- $d_{s,gov}$ = Depth of the longitudinal stiffener at the governing section
- D_{eff} = Effective Web Depth
- D_{gov} = Web depth of the governing section

The effective sections can be view in the “MethodBEffectiveSections” SpecCheck article. Note that all flanges are converted to an double angle flange when computing the effective section. The width dimension in the specCheck article is of one leg width. Therefore, the width of plate flanges will be shown as half the width in the SpecCheck article.

4.7.3 Method C

The inputted value for the elastic-lateral-torsional buckling load ratio directly taken in 6.10.8.2.3c-2 or A6.3.3.3-2, as-applicable. It likely that differing Method's or values for the elastic-lateral- torsional buckling load ratio will need to be entered using the control option ranges.

4.7.4 Method P

Method P shown in the specCheck indicates the section was evaluated using the prismatic procedures.

4.7.5 Method Z (Nonconforming Non-Prismatic Unbraced Lengths)

Denoted as Method Z in CT-Steel, is the handling of non-prismatic unbraced lengths which do not meet the Design Specifications. For unbraced lengths which have section changes in the top or bottom flange that result in a change in flange lateral moment of inertia (include flanges, cover plates, and side plates) greater than 2 times for any section change within 25% of the unbraced length, CT-Steel will use the following method.

Note: Users can force Method Z by control option.

CT-Steel will compute γ_E for the unbraced length as the section which produces the smallest elastic later-torsional buckling load ratio, as calculated below.

$$\gamma_E = \frac{C_{be} F_E S_{xc}}{M_{u \max}}$$

Where:

γ_E = Elastic lateral-torsional buckling load ratio calculated at a section within the unbraced length.

C_{be} = Taken equal to 1.0.

F_E = Elastic lateral-torsional buckling stress, computed as

$$\frac{\pi^2 E}{\left(\frac{kL_b}{r_t}\right)^2}$$

Where r_t is computed at the section under consideration within the unbraced in accordance with BDS 6.10.8.2.3b-3.

S_{xc} = Elastic section modulus about the major axis of the section to the compression flange taken as M_{yc}/F_{yc} , calculated at the section under consideration.

$M_{u \max}$ = Maximum factored major-axis bending moment within the unbraced length under consideration, including the end cross-sections, calculated from the moment envelope values that produces the largest flexural compression in the flange under consideration.

Section 5 Shear Resistance

5.1 Stiffener Range

The program will evaluate shear at all POIs. CT-Steel will use the stiffener properties defined at each POI to determine if the shear resistance. The "Range" stiffener will be used at all locations within the stiffener input, except for POIs within D0 of the end of the stiffener input. In that case, the more conservative stiffener will be used between the "Range" stiffener and "End" stiffener.

5.2 Stiffened Panels

For a panel to be considered stiffened the section shall satisfy the Criteria for Stiffened Panel. Otherwise, the section shall be evaluated as unstiffened.

Note that if a stiffener definition is not defined, CT-Steel will consider the Criteria for Stiffened Panel satisfied, the criteria for stiffener spacing is satisfied, and Criteria to Consider Post-Buckling Tension Field Action considered is unsatisfied.

5.2.1 *Criteria for Stiffened Panel*

CT-Steel will consider the section stiffened based on the following criteria.

- max of $6t_w$ from either tension flange
- BDS Equations 6.10.11.1.3-1 & 6.10.11.1.3-2 are satisfied. Minimum moment of inertia to develop stiffened web. Or TreatAsStiffened control option = TRUE.
- Stiffener spacing does not exceed $3D$ for webs without longitudinal stiffeners.
- Stiffener spacing does not exceed $1.5D$ for webs with longitudinal stiffeners, or for an End Panel. (BDS 6.10.9.1).

5.2.2 *Post Buckling Tension Field Action*

For panel sections to develop tension field action the panel section shall satisfy Criteria to Consider Post-Buckling Tension Field Action.

5.2.2.1 *Criteria to Consider Post-Buckling Tension Field Action*

- All of Criteria for Stiffened Panel, listed above, are satisfied
- BDS Equations 6.10.11.1.3-7 & 6.10.11.1.3-8 are satisfied
- BDS Equation 6.10.11.1.3-11 is satisfied if Longitudinal Stiffeners are present
- Panel is not considered an End Panel.

5.2.2.2 *Stiffeners Adjacent to Post buckling Tension-Field Action*

The following approach is used to determine the post-buckling tension-field action:

$$V_{n(max)} = (V_n - V_{cr})\rho_w + V_{cr}$$

Where:

$V_{n(max)}$ = The nominal Shear-yielding or shear-buckling plus partial postbuckling tension-field action plus shear resistance of interior web panels limited by the rigidity of the transverse stiffener (kip)

ρ_w = Ratio of maximum postbuckling resistance to the full postbuckling resistance

= If $I_{t2} > I_{t1}$, then:

$$0.0 \leq \frac{I_t - I_{t1}}{I_{t2} - I_{t1}} \leq 1.0$$

Otherwise:

$$1.0$$

V_n = Shear-yielding or shear-buckling plus postbuckling tension-field action resistance of the web panel under consideration determined as specified in BDS Article 6.10.9.3.2 (kip)

V_{cr} = Shear-yield or shear-buckling resistance of the web panel under consideration (kip)

$$= \frac{CV_p}{V_n}$$

C = Ratio of the shear-buckling resistance to the shear yield strength determined by BDS Eq. 6.10.9.3.2-4, 6.10.9.3.2-5, or 6.10.9.3.2-6, as applicable

V_p = Plastic shear force (kip)

$$= 0.58F_{yw}Dt_w$$

F_{yw} = Specified minimum yield strength of the web (ksi)

D = Web depth (in.)

t_w = Web thickness (in.)

I_t = Moment of inertia of the transverse stiffener taken about the edge in contact with the web for single stiffeners and about the mid-thickness of the web for stiffener pairs (in.⁴)

I_{t1} = Minimum moment of inertia of the transverse stiffener required for the development of the web shear-buckling resistance as defined by BDS Eq. 6.10.11.1.3-3 (in.⁴)

I_{t2} = Minimum moment of inertia of the transverse stiffener required for the development of the full web buckling plus postbuckling tension-field action resistance as defined by BDS Eq. 6.10.11.1.3-4 (in.⁴)

5.2.3 Definition of an End Panel

Panel sections are considered an *End Panel* when the section is within less than the stiffener spacing away from:

- a defined a simple support, or
- an in-span hinge