sCheck

A Steel Section Check and Design Program

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Introduction

sCheck is a versatile steel section check and design Windows program based on ANSI/AISC 360-22 (16th edition) LRFD, 360-16 (15th edition) LRFD, and ANSI/AISC 360-10 (14th edition) LRFD.

The program includes the following features:

- Check the capacity of any of the standard AISC shapes (W, M, S, HP, C, MC, L, WT, MT, ST, 2L, HSS, PIPE) against a set of load effects.
- Design and select optimal standard AISC shapes against a set of load effects.
- Consider moment magnification for non-sway condition.
- Auto generate (and batch generate) detailed calculation procedures in Word and PDF formats.

Graphical User Interface (GUI)

sCheck has a simple and user-friendly user interface (Figure 0.1). There are two modes in sCheck: Section Check (Figure 0.2) and Section Design (Figure 0.3).

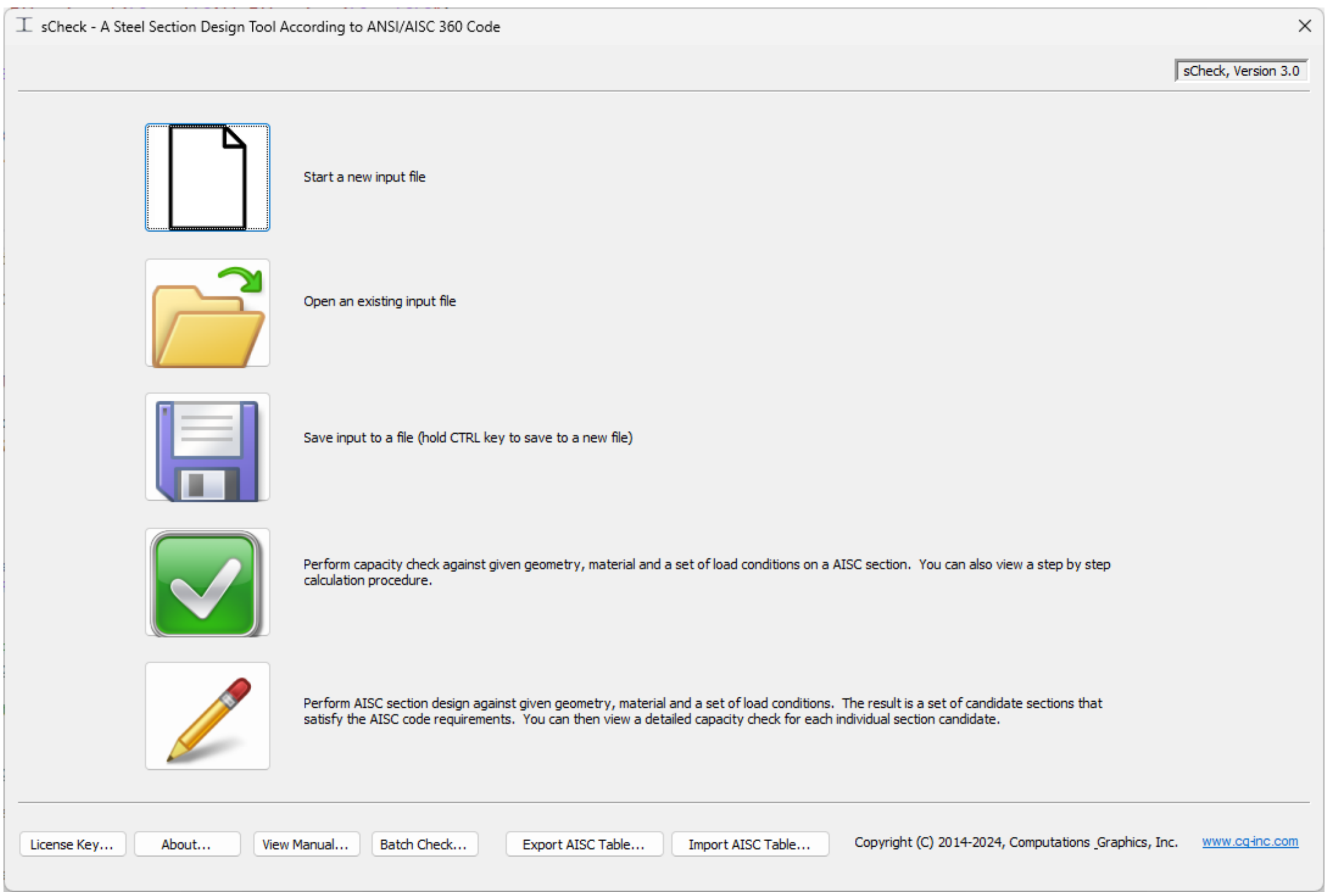


Figure 0.1

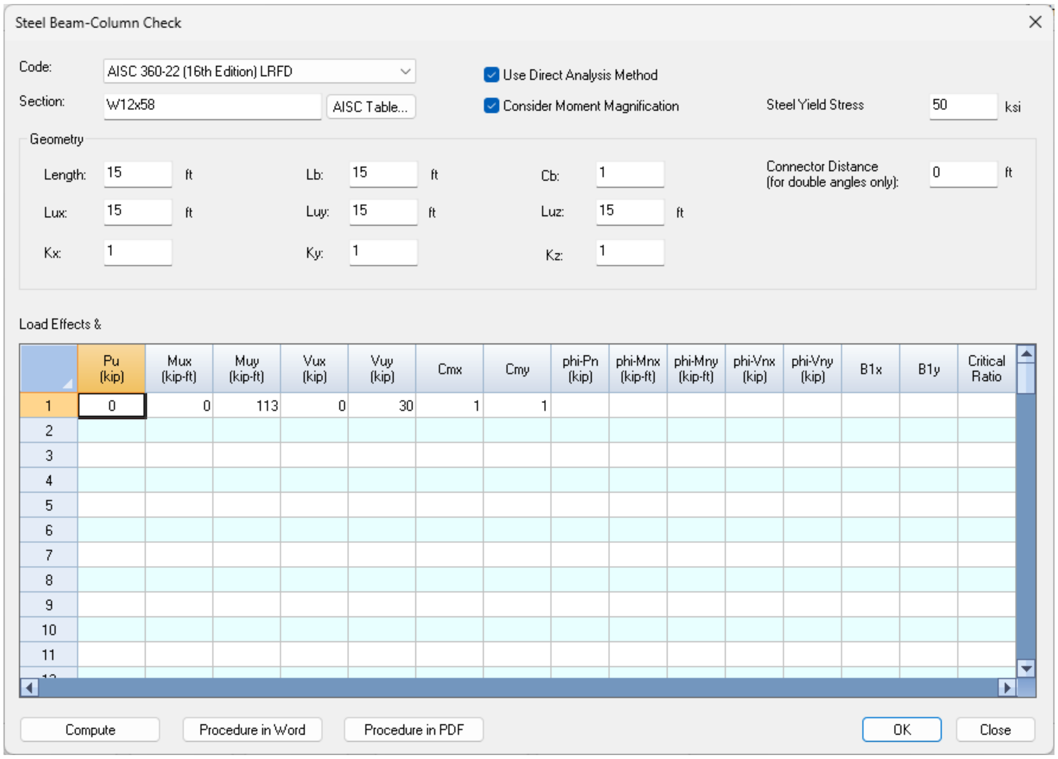


Figure 0.2

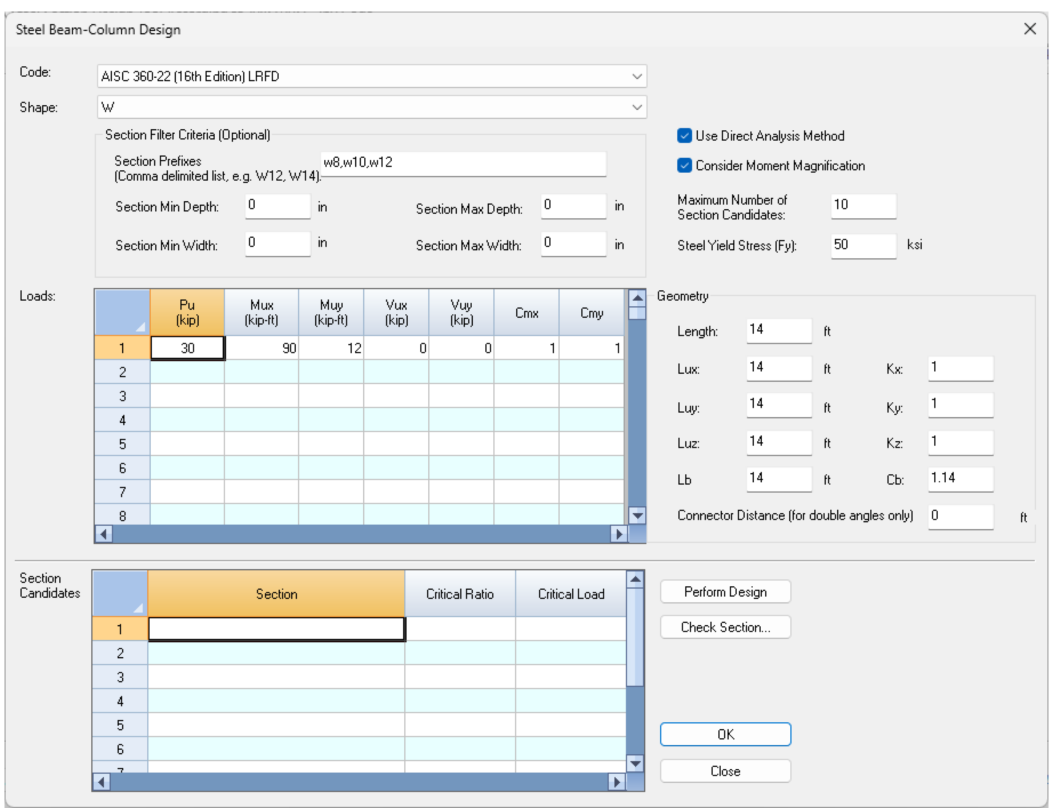


Figure 0.3

Operating System Requirements

x64-based Windows 7, 8, 10, 11

ARM64-based Windows 11

Chapter 1: Section Orientations

The orientations of section local X and Y axes of various AISC shapes are shown below.

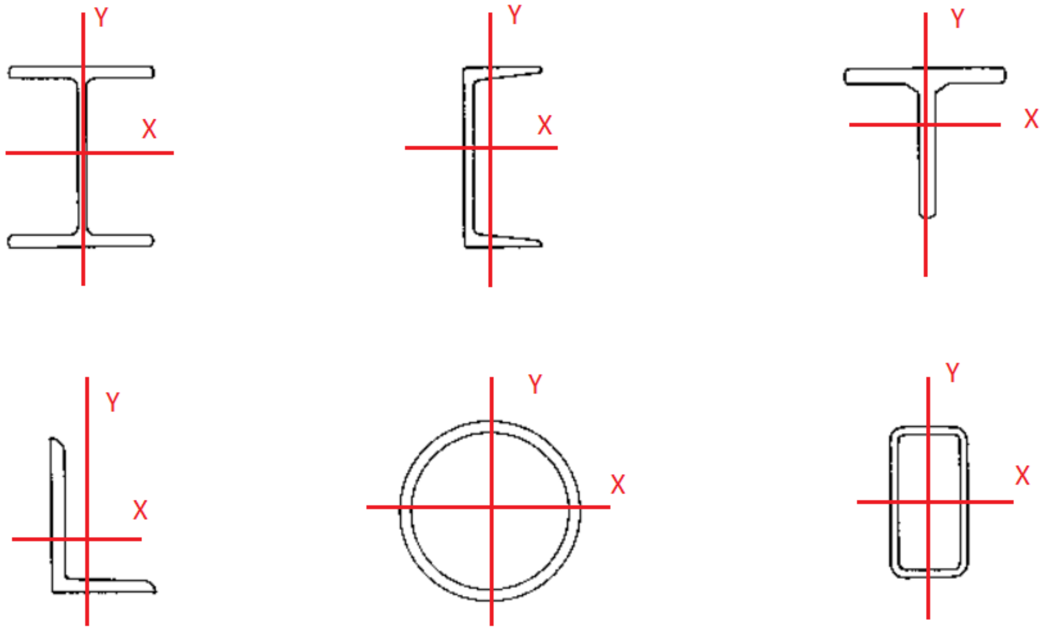


Figure 1.1

Chapter 2:   Member Internal Forces and Moments

1. Axial force P acts perpendicular to the section.   Moments M_x and M_y act about section local X and Y axes respectively.   They have the following sign conventions.

Axial Force P: positive for compression; negative for tension

Moment M_x: Positive when section top most fiber is under compression.

Moment M_y: Positive when section rightmost fiber is under compression.

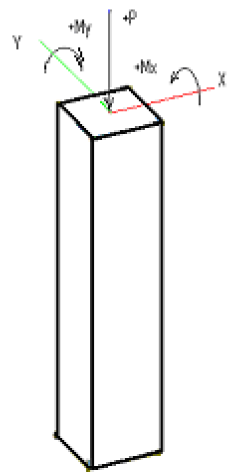


Figure 2.1

2.   All moments are referenced about the geometric centroid of the gross section.

3.   Unlimited number of factored load sets can be input via spreadsheet.   Loads are the required strength computed by the code-specified factored load combinations using either hands or analysis program such as Real3D-Analysis.   It is assumed that an overall 2nd order P-Delta (P-Δ) analysis has been performed on a sway structure.   If desired, the program uses moment magnification procedure to calculate the P-delta (P-δ) effect, which accounts for slenderness of columns in non-sway structure or for slenderness along the lengths of columns in sway structure.

4.   Critical ratio is computed for each section based on the magnified factored loads and the capacity of the section.   Critical ratio equal or less than 1.0 means the design strength is greater than the required strength and the section is adequate.   Critical ratio greater than 1.0 means the design strength is less than the required strength and the section is inadequate.

Chapter 3: Section Check

The Section Check input and output are shown below, followed by a detailed calculation procedure that sCheck produces.

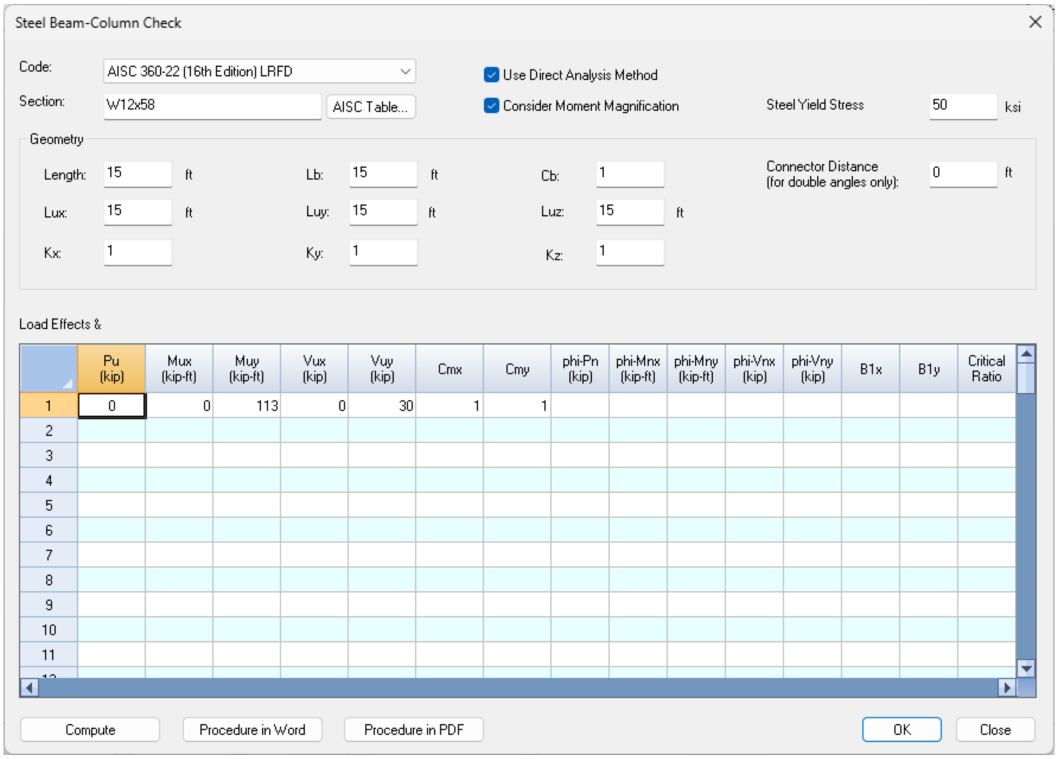


Figure 3.1

Lux, Luy and Luz are unbraced lengths in local x, y and z directions.
Kx, Ky and Kz are unbraced length factors in local x, y and z directions.

Lb is the unbraced lateral length.
Cb is the lateral-torsional buckling modification factor for non-uniform moment diagrams. It should be greater or equal to 1.0. You can use 1.0 for Cb conservatively.

Connector Distance is used for double angles only.

Pu, Mux, Muy, Vux, Vuy are required axial, major moment, minor moment, major shear and minor shear. For Pu, the compressive force is positive while tensile force is negative. Moment Mux is positive when section top most fiber is under compression. Moment Muy is positive when section rightmost fiber is under compression. Moment magnification may be optionally considered to account for the P-delta (P-δ) effect.

If direct analysis method is chosen, the program will account for stiffness reduction when calculating the moment magnification factor B_{1x} and B_{1y} for P- δ effects. It is assumed that a P- Δ (only) second-order analysis is performed for the load effects (P_u, M_{ux}, M_{uy}, V_{ux}, and V_{uy}).

Cmx, Cmy are coefficients accounting for non-uniform moments when computing moment magnification. You can use 1.0 for Cmx and Cmy conservatively. If 0 is entered for Cmx or Cmy, 1.0 is used in the computation instead.

Results include axial capacity (phi-Pn), moment capacity (phi-Mnx, phi-Mny), shear capacity (phi-Vnx, phi-Vny), moment magnification factors (B1x, B1y), and critical ratio. The section is deemed safe to resist a load if the critical ratio is less than 1.0, otherwise, the section is deemed unsafe. Please note that for a single angle, the moment capacities are given about the principal w-w and z-z axes and the input moments Mux and Muy are transformed in the principal axes before flexural-axial interaction ratio is checked.

Chapter 4: Section Design

The Section Design input and output are shown below:

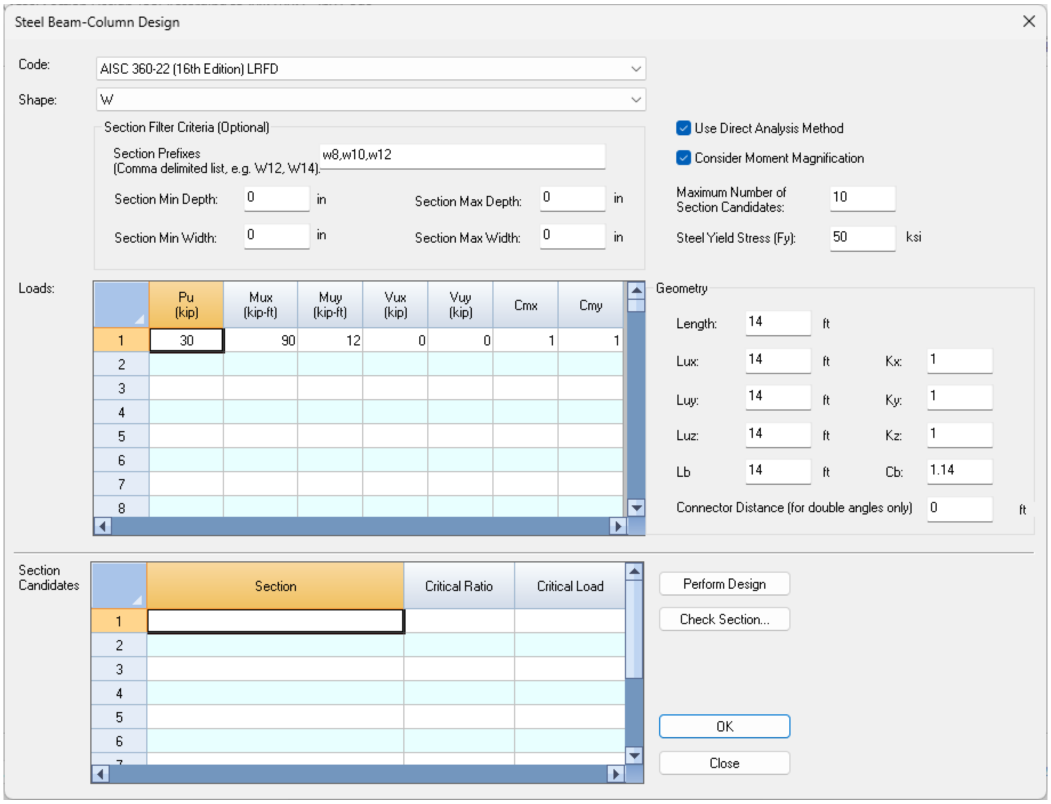


Figure 4.1

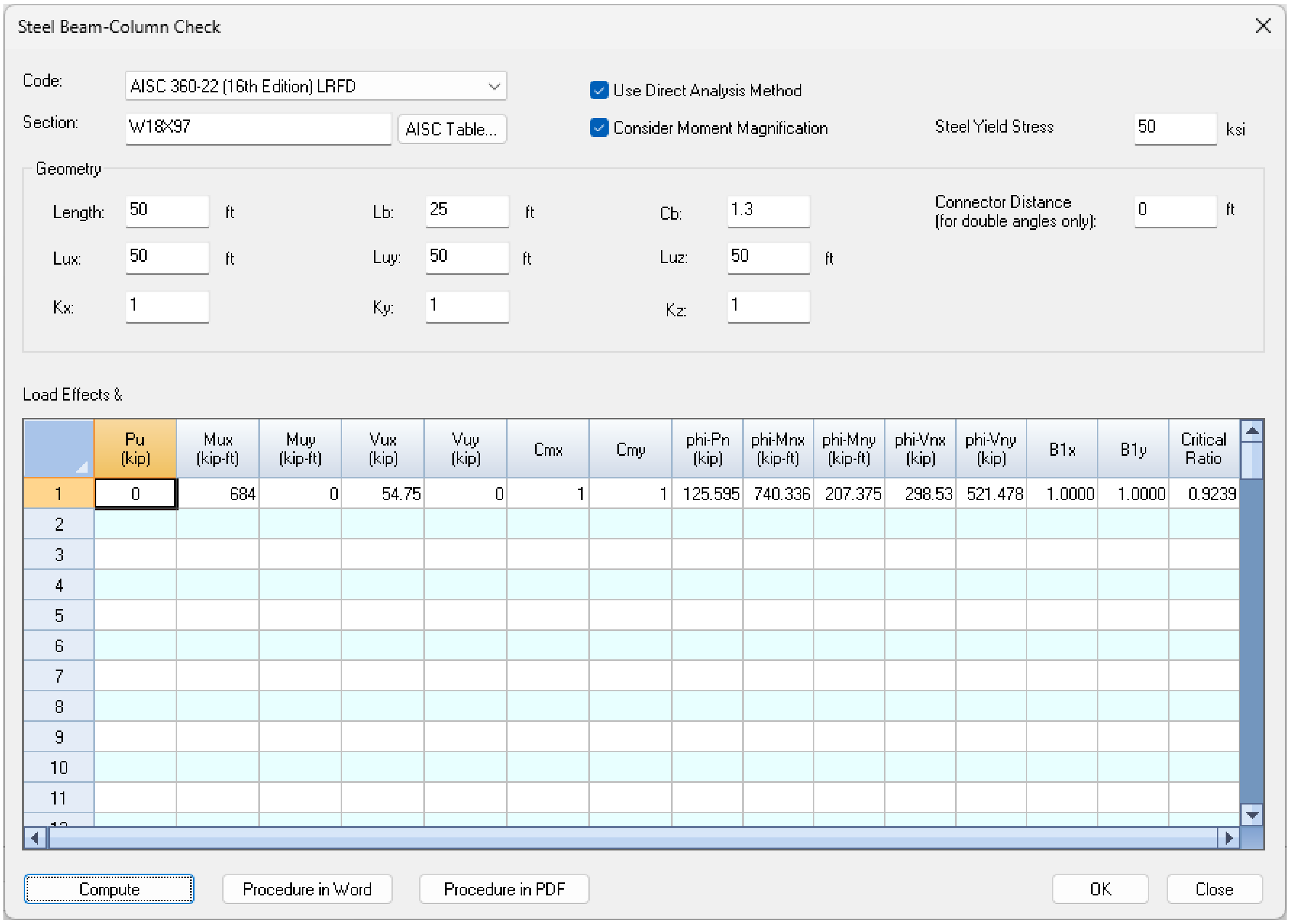
For Section Filter Criteria, you can use either Section Prefixes or section dimension limits (but not both). The section prefixes is a comma delimited list such as W12, W14. If section prefixes is used, the section dimension limits will be ignored. If a section dimension limit is zero, then that limit criteria is ignored.

By default, a maximum of ten section candidates will be provided after a successful design. You can then view the detailed check for each of the section candidate.

Chapter 5: Verification Examples – AISC 360-22 (16th edition) LRFD

Example 1 – W Shape Beam (Flexure in Major Axis, Lp < Lb < Lr)

[Ref 2, pp435]
Check the flexural capacity of W18x97 beam.
Fy = 50 ksi, L = 50 ft, Lb = 25 ft. Cb = 1.3.

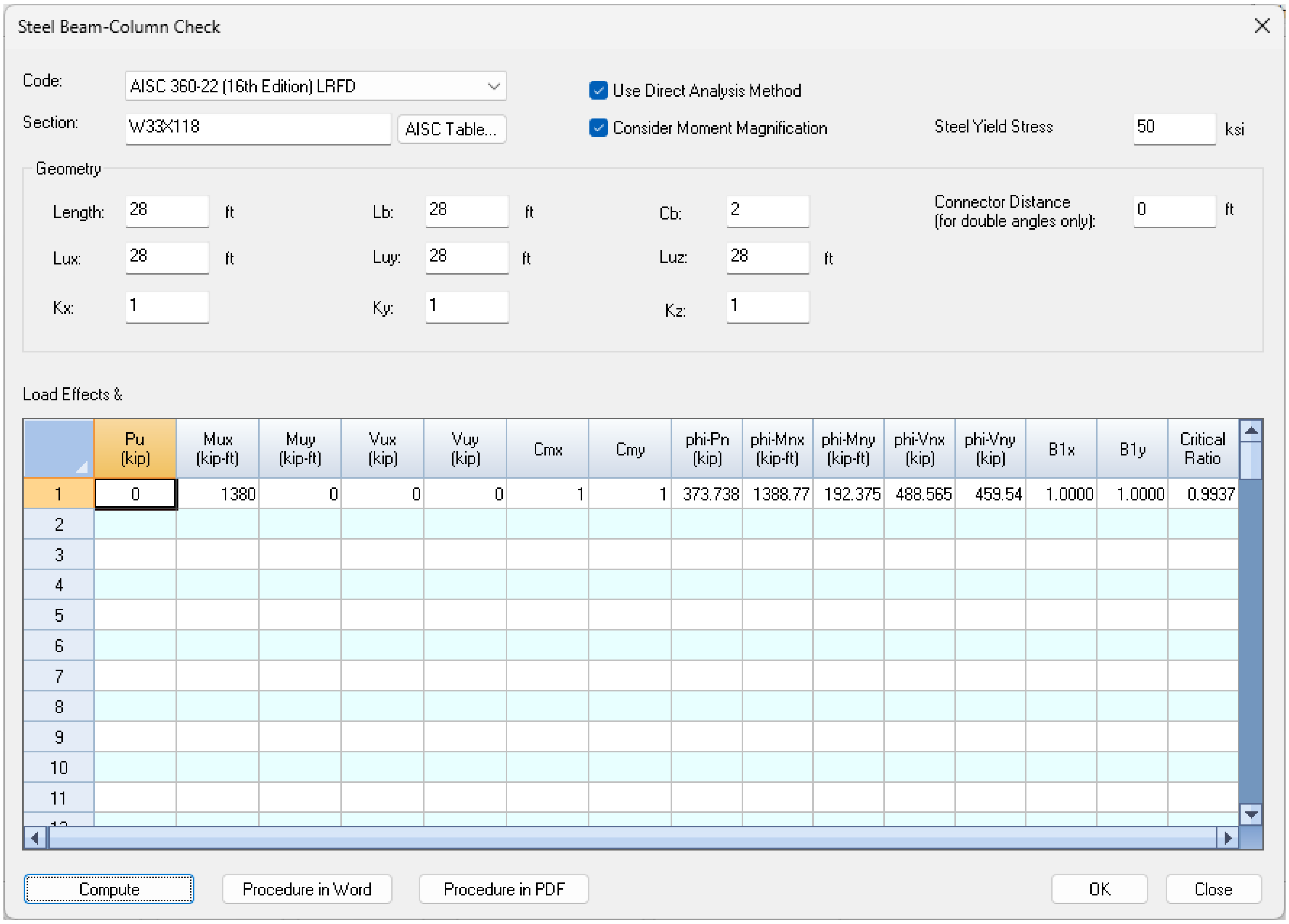


Result Comparison

The reference gives $_{\varphi}$Mnx = 740 ft-kips while sCheck gives $_{\varphi}$Mnx = 740.336 ft-kips. They are practically identical.

Example 2 – W Shape Beam (Flexure in Major Axis, Lb > Lr)

[Ref 2, pp440]
Check the flexural capacity of W33x118 beam.
Fy = 50 ksi, L = 28 ft, Lb = 28ft, Cb = 2.0.

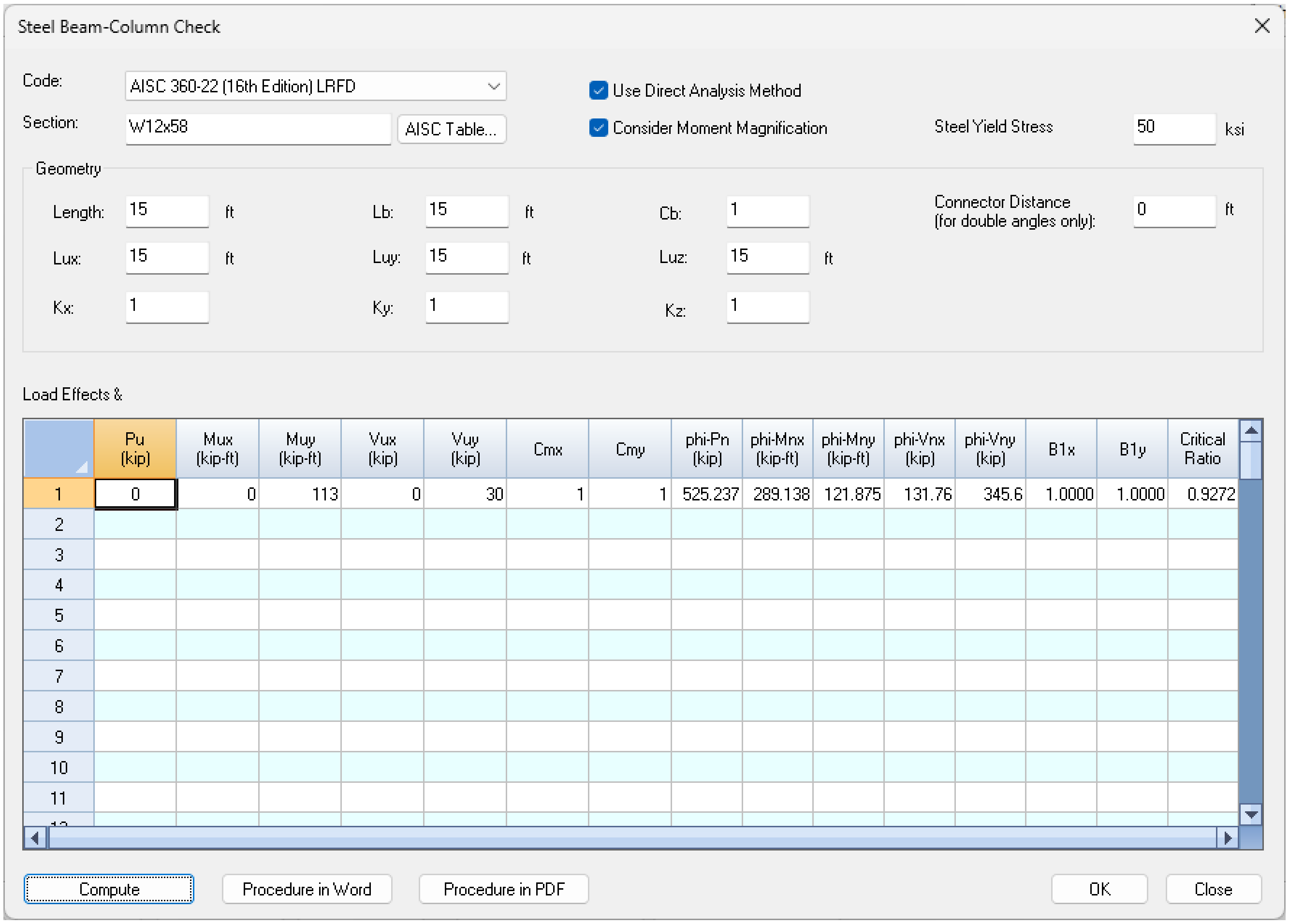


Result Comparison

The reference gives $_{\varphi}$Mnx = 1390 ft-kips while sCheck gives $_{\varphi}$Mnx = 1388.77 ft-kips. They are practically identical.

Example 3 – W Shape Beam (Flexure in Minor Axis)

[Ref 6, Example F.5]
Check the flexural capacity of W12x58 beam in minor axis.   Fy = 50 ksi, L = 15 ft.

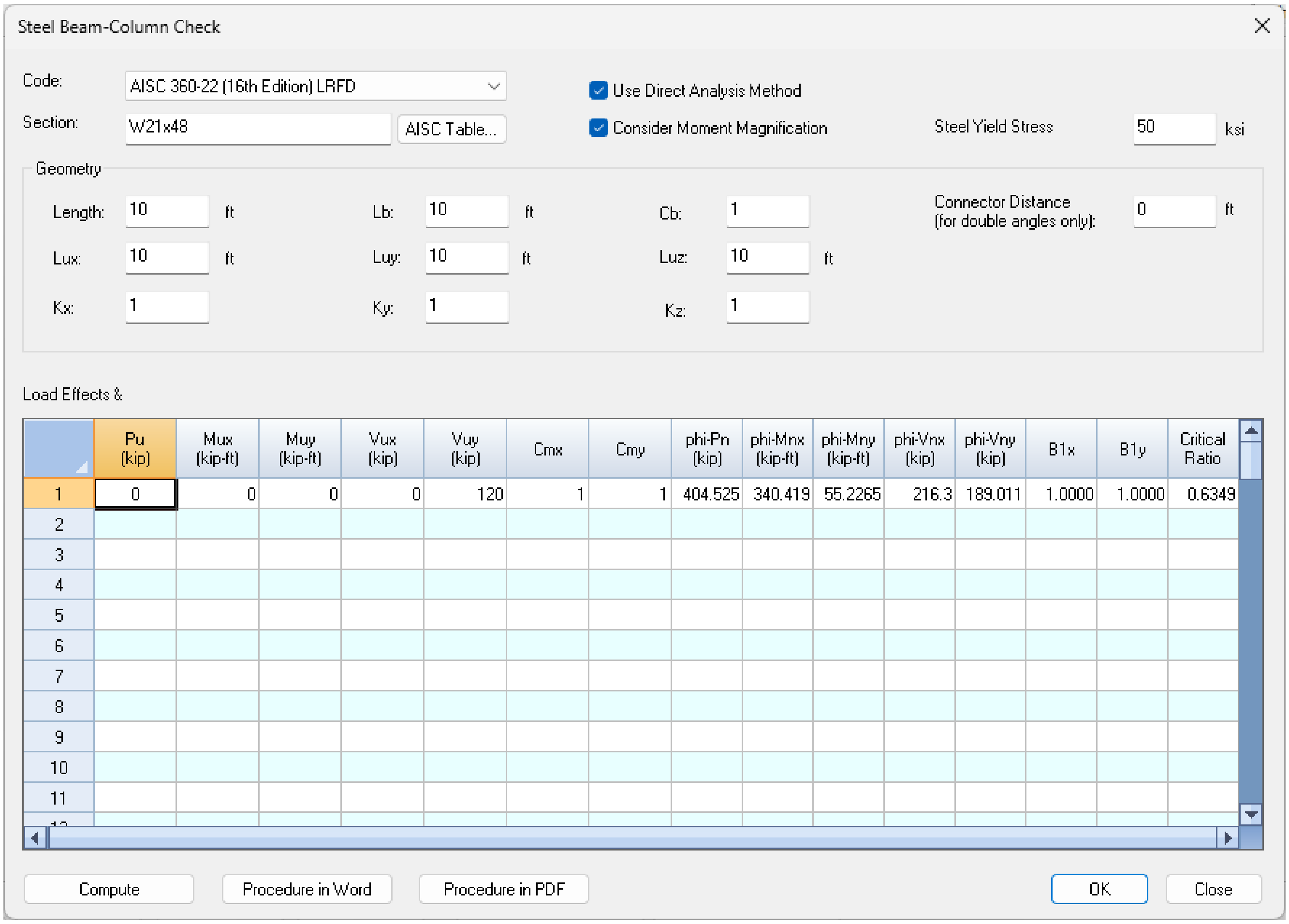


Result Comparison

The reference gives $_{\varphi}$Mny = 122 ft-kips while sCheck gives $_{\varphi}$Mny = 121.875 ft-kips.   They are practically identical.

Example 4 – W Shape Beam (Shear in Minor Axis)

[Ref 6, Example G.6]
Check the shear capacity of W21x48 beam in minor axis.
Fy = 50 ksi

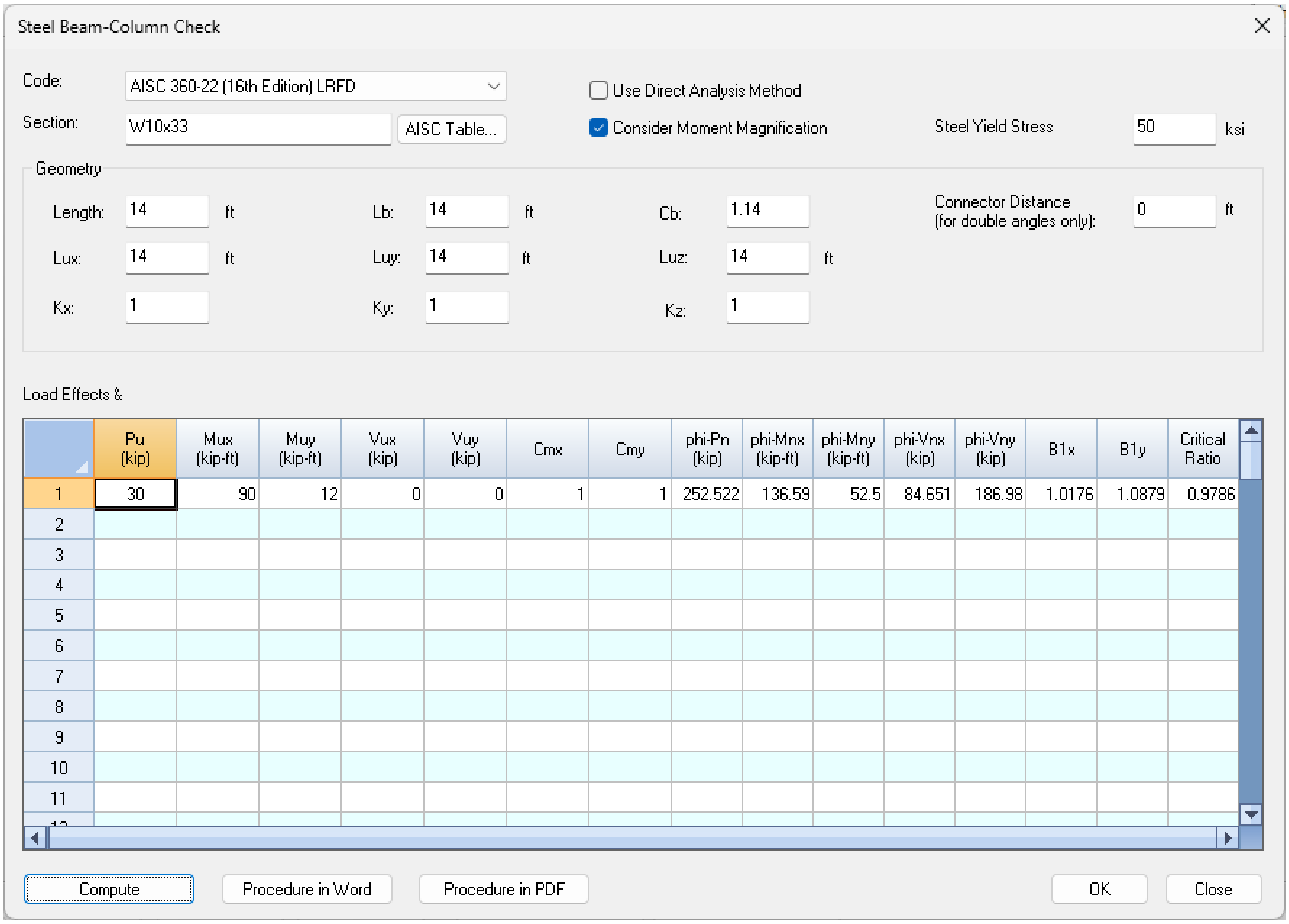


Result Comparison

The reference gives $_{\varphi}$Vny = 189 ft-kips while sCheck gives $_{\varphi}$Vny = 189.011 ft-kips. They are practically identical.

Example 5 – W Shape Column (Combined Axial Compression and Flexure)

[Ref 6, Example H.4]
Check the capacity of W10x33 column in combined axial compression and flexure.
Fy = 50 ksi, L = 14 ft, Kx = Ky = Kz = 1.0, Lux = Luy = Lb = 14 ft, Cb = 1.14.

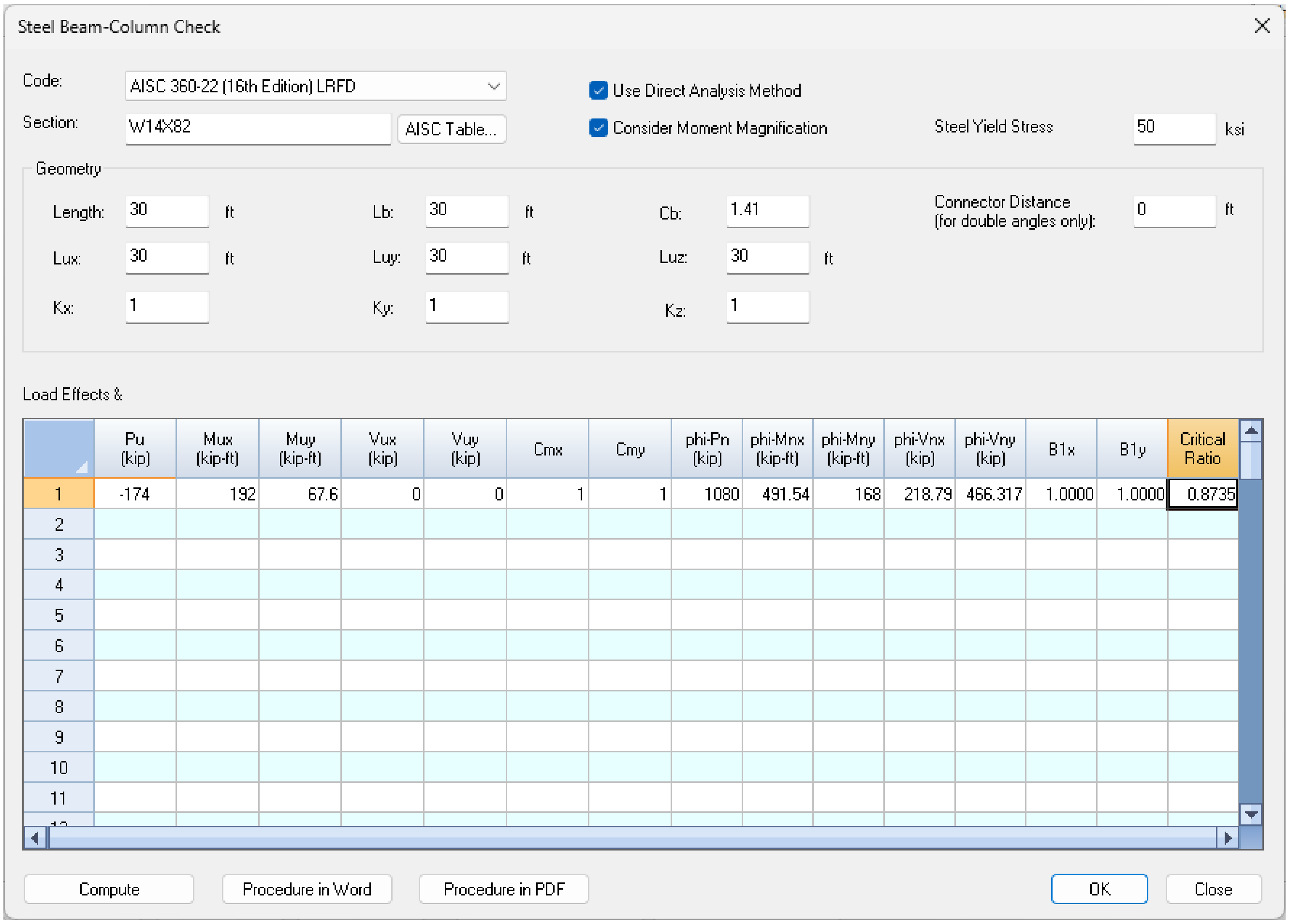


Result Comparison

The reference gives $_{\varphi}$Pn = 253 kips, $_{\varphi}$Mnx = 137 ft-kips, $_{\varphi}$Mny = 52.5 ft-kips while sCheck gives $_{\varphi}$Pn = 252.522 kips, $_{\varphi}$Mnx = 136.59 ft-kips, $_{\varphi}$Mny = 52.5 ft-kips.   They are practically identical.

Example 6 – W Shape Column (Combined Axial Tension and Flexure)

[Ref 6, Example H.3]
Check the capacity of W14x82 column in combined axial tension and flexure.
Fy = 50 ksi, L = 30 ft, Kx = Ky = Kz = 1.0, Lux = Luy = Lb = 30 ft, Cb = 1.41.
Note: tension load must be entered as negative value.

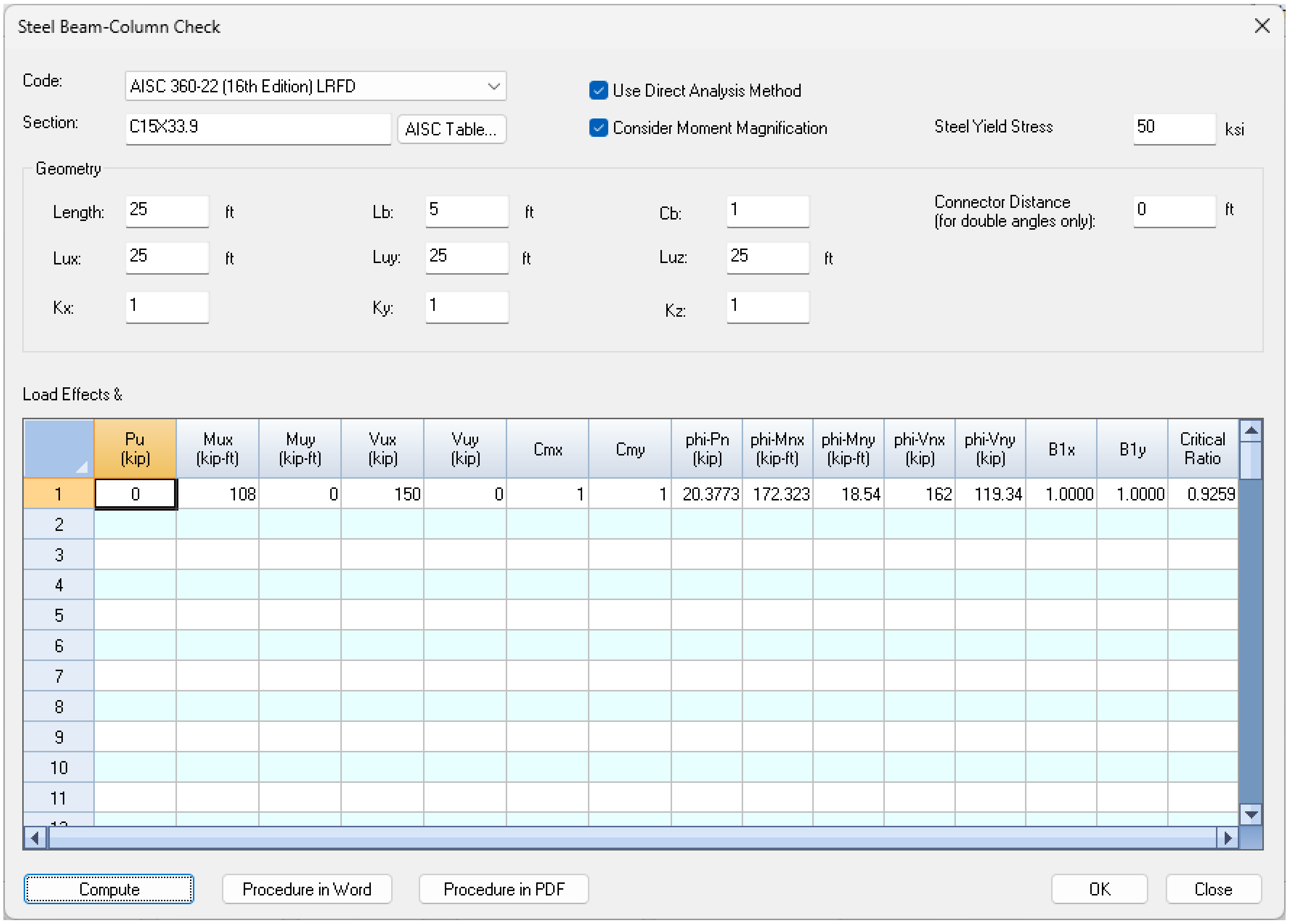


Result Comparison

The reference gives $_{\varphi}$Pn = 1080 kips, $_{\varphi}$Mnx = 492 ft-kips, $_{\varphi}$Mny = 168 ft-kips while sCheck gives $_{\varphi}$Pn = 1080 kips, $_{\varphi}$Mnx = 491.54 ft-kips, $_{\varphi}$Mny =168 ft-kips. They are practically identical. Please note that absolute value of axial load should be used for axial-flexural interaction formula.

Example 7 – C Shape Beam (Flexure and Shear)

[Ref 6, Example F.2-2A and G.2B]
Check the capacity of C15x33.9 column in flexure and shear.
Fy = 50 ksi, L = 25 ft, Lb = 5 ft, Cb = 1.0.

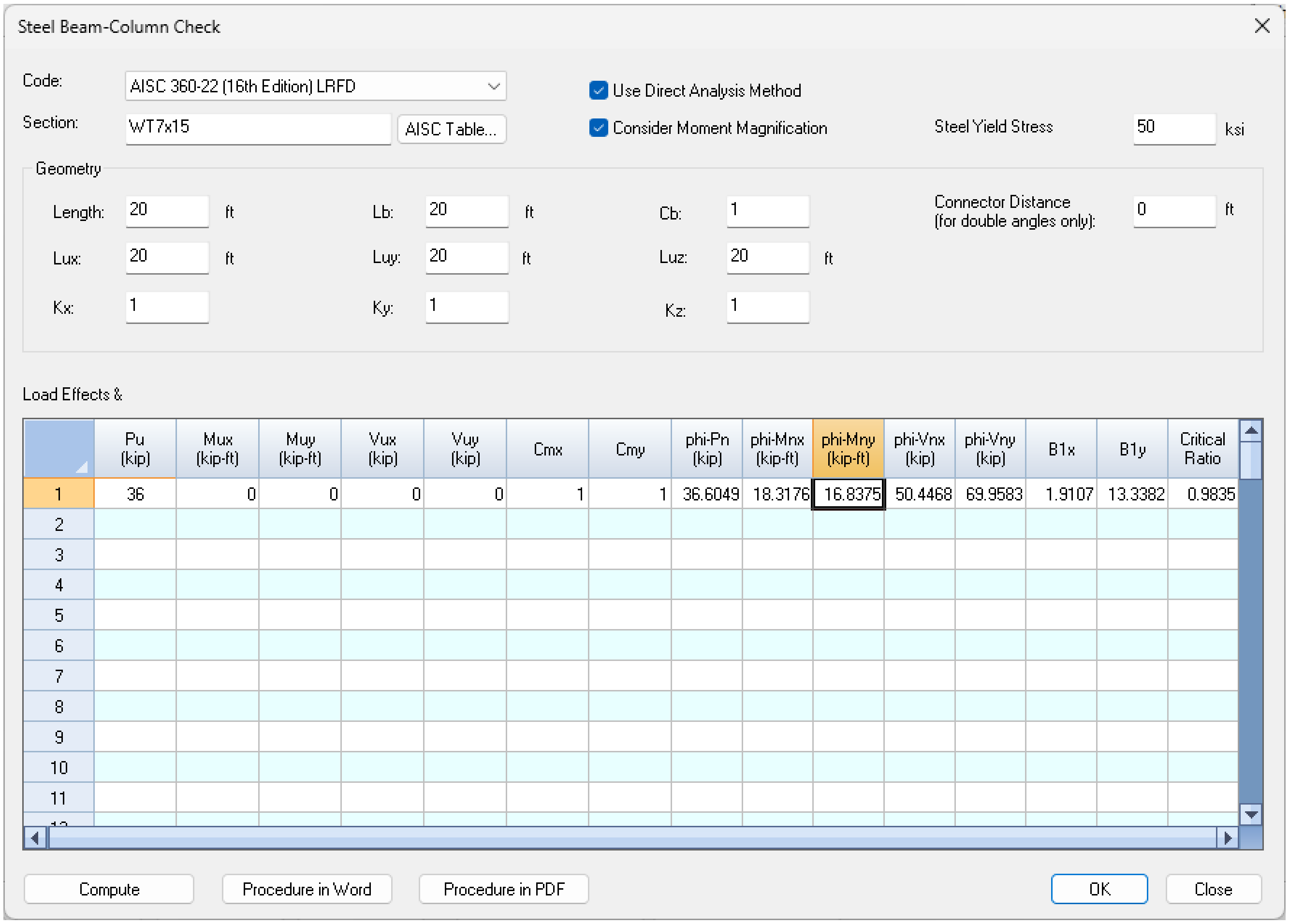


Result Comparison

The reference gives $_{\varphi}$Mnx = 173 ft-kips, $_{\varphi}$Vnx = 162 kips while sCheck gives $_{\varphi}$Mnx = 172.323 ft-kips, $_{\varphi}$Vnx =162 kips. They are practically identical.

Example 8 – WT Shape Column (Axial Compression)

[Ref 6, Example E.8]
Check the capacity of WT7x15 column in axial compression.
Fy = 50 ksi, L = 20 ft, Kx = Ky = Kz = 1.0, Lu = 20 ft, Cb = 1.41.

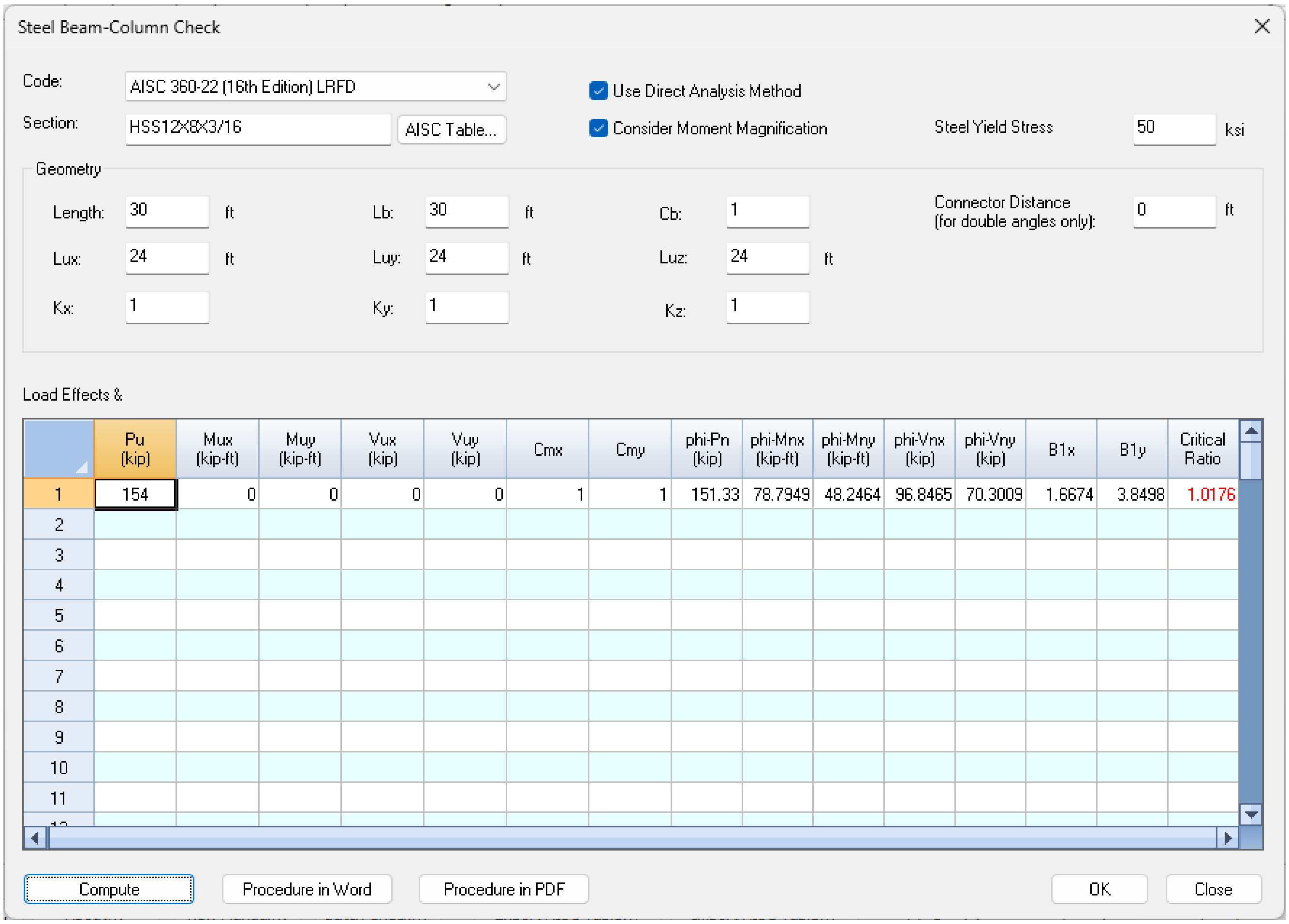


Result Comparison

The reference gives $_{\varphi}$Pn = 36.6 kips while sCheck gives $_{\varphi}$Pn = 36.6049 kips. They are practically identical.

Example 9 – HSS Shape Column (Axial Compression)

[Ref 6, Example E.10]
Check the capacity of HSS12x8x3/16 column in axial compression.
Fy = 50 ksi, L = 30 ft, Kx = Ky = 0.8, Kz = 1.0, Lu = 30 ft, Cb = 1.0.

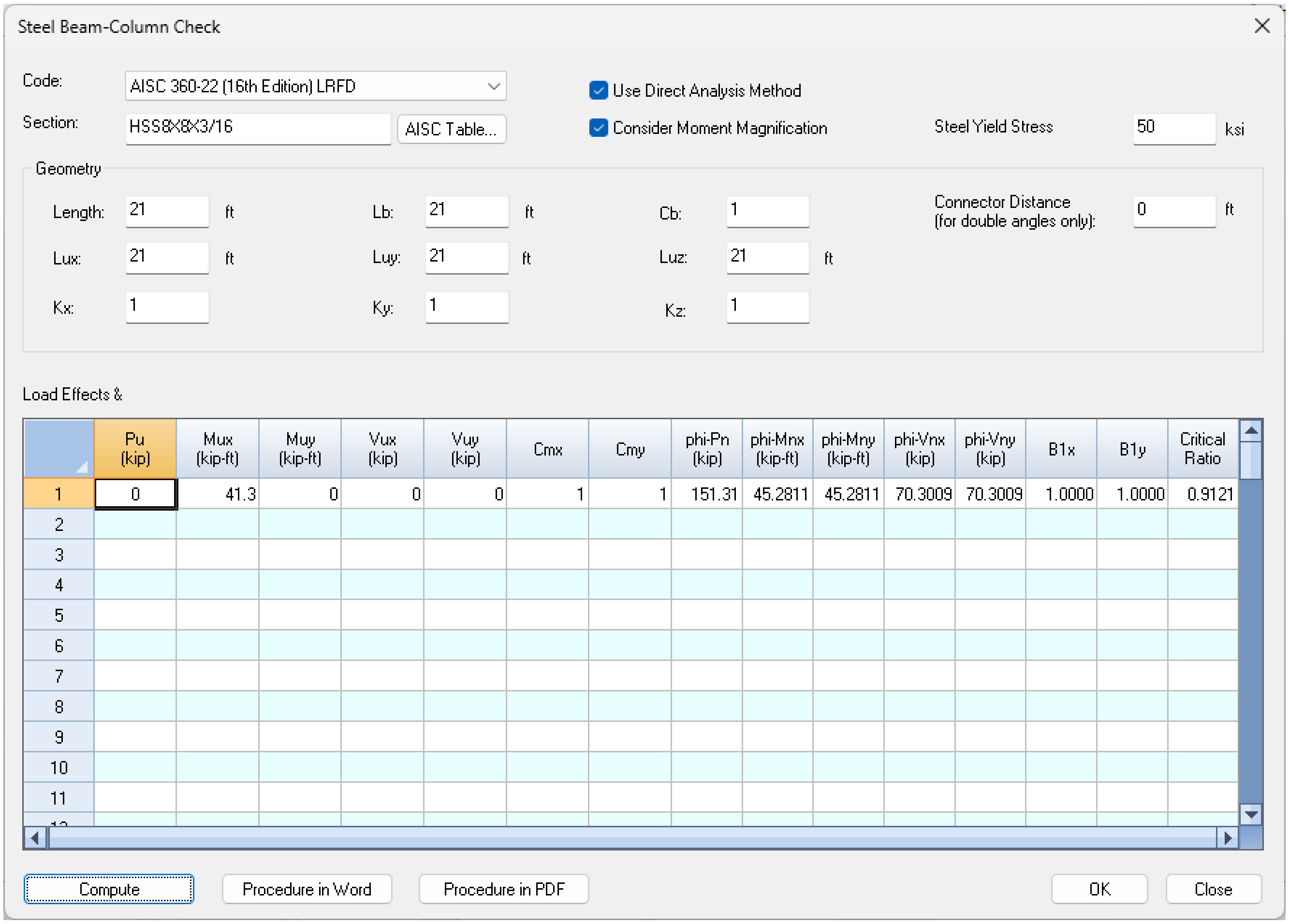


Result Comparison

The reference gives $_{\varphi}$Pn = 151 kips while sCheck gives $_{\varphi}$Pn = 151.33 kips. They are practically identical. The reference gives $_{\varphi}$Pn = 177 kips while sCheck gives $_{\varphi}$Pn = 177.832 kips for Lc = 18 ft. The reference gives $_{\varphi}$Pn = 74.2 kips while sCheck gives $_{\varphi}$Pn = 74.3864 kips for Lc = 40 ft.

Example 10 – Rectangular HSS Shape Beam (Flexure)

[Ref 6, Example F.8B]
Check the capacity of HSS8x8x3/16 column in flexure.
Fy = 50 ksi, L = 21 ft, Lb = 21 ft, Cb = 1.0.


Steel Beam-Column Check

Code: AISC 360-22 (16th Edition) LRFD
Section: HSS8X8X3/16 AISC Table...
Use Direct Analysis Method
Consider Moment Magnification
Steel Yield Stress 50 ksi

Geometry
Length: 21 ft Lb: 21 ft Cb: 1 Connector Distance (for double angles only): 0 ft
Lux: 21 ft Luy: 21 ft Luz: 21 ft
Kx: 1 Ky: 1 Kz: 1

Load Effects &

	Pu (kip)	Mux (kip-ft)	Muy (kip-ft)	Vux (kip)	Vuy (kip)	Cmx	Cmy	phi-Pn (kip)	phi-Mnx (kip-ft)	phi-Mny (kip-ft)	phi-Vnx (kip)	phi-Vny (kip)	B1x	B1y	Critical Ratio
1	0	41.3	0	0	0	1	1	151.31	45.2811	45.2811	70.3009	70.3009	1.0000	1.0000	0.9121
2															
3															
4															
5															
6															
7															
8															
9															
10															
11															

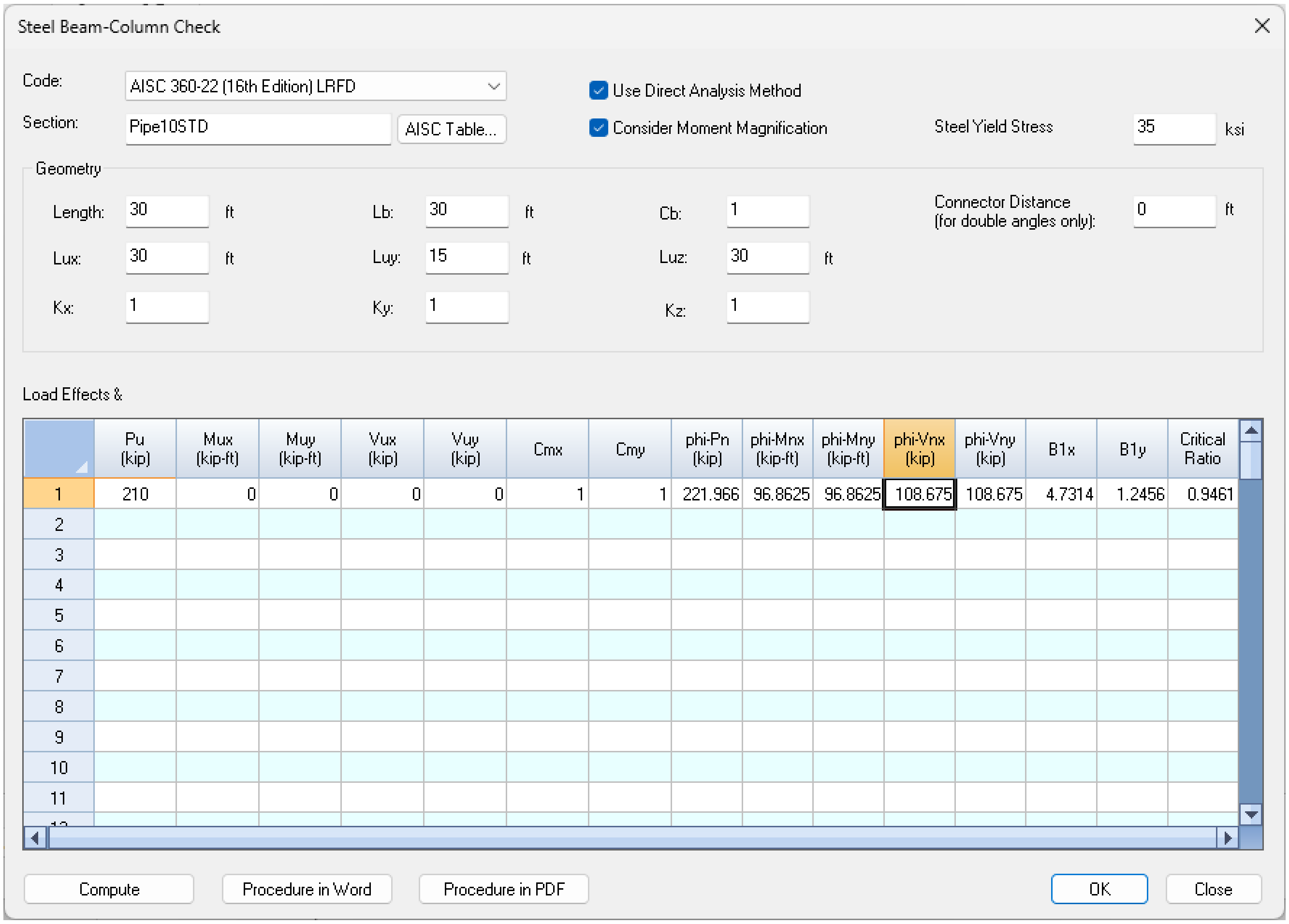
Compute Procedure in Word Procedure in PDF OK Close

Result Comparison

The reference gives $_{\varphi}$Mnx = 45.4 ft-kips while sCheck gives $_{\varphi}$Mnx = 45.2811 ft-kips. They are practically identical.

Example 11 – Pipe Shape Column (Compression)

[Ref 6, Example E.11]
Check the capacity of Pipe10STD column in flexure.
Fy = 35 ksi, L = 30 ft

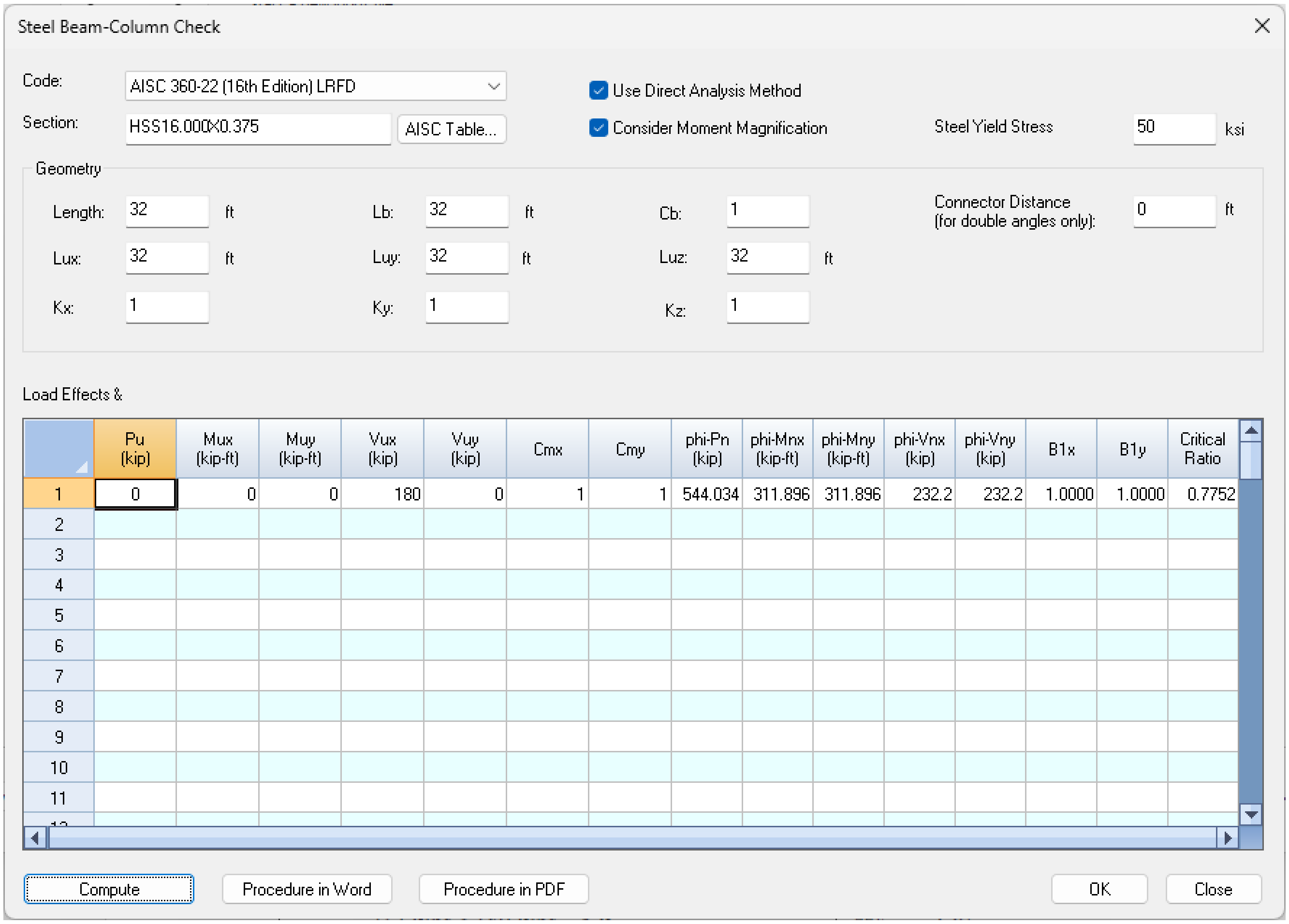


Result Comparison

The reference gives $_{\varphi}$Pn = 222 kips while sCheck gives $_{\varphi}$Pn = 221.966 kips. They are practically identical.

Example 12 – Round HSS Shape Beam (Shear)

[Ref 6, Example G.5]
Check the capacity of HSS16.000X0.375 column in shear.
Fy = 50 ksi, L = 32 ft

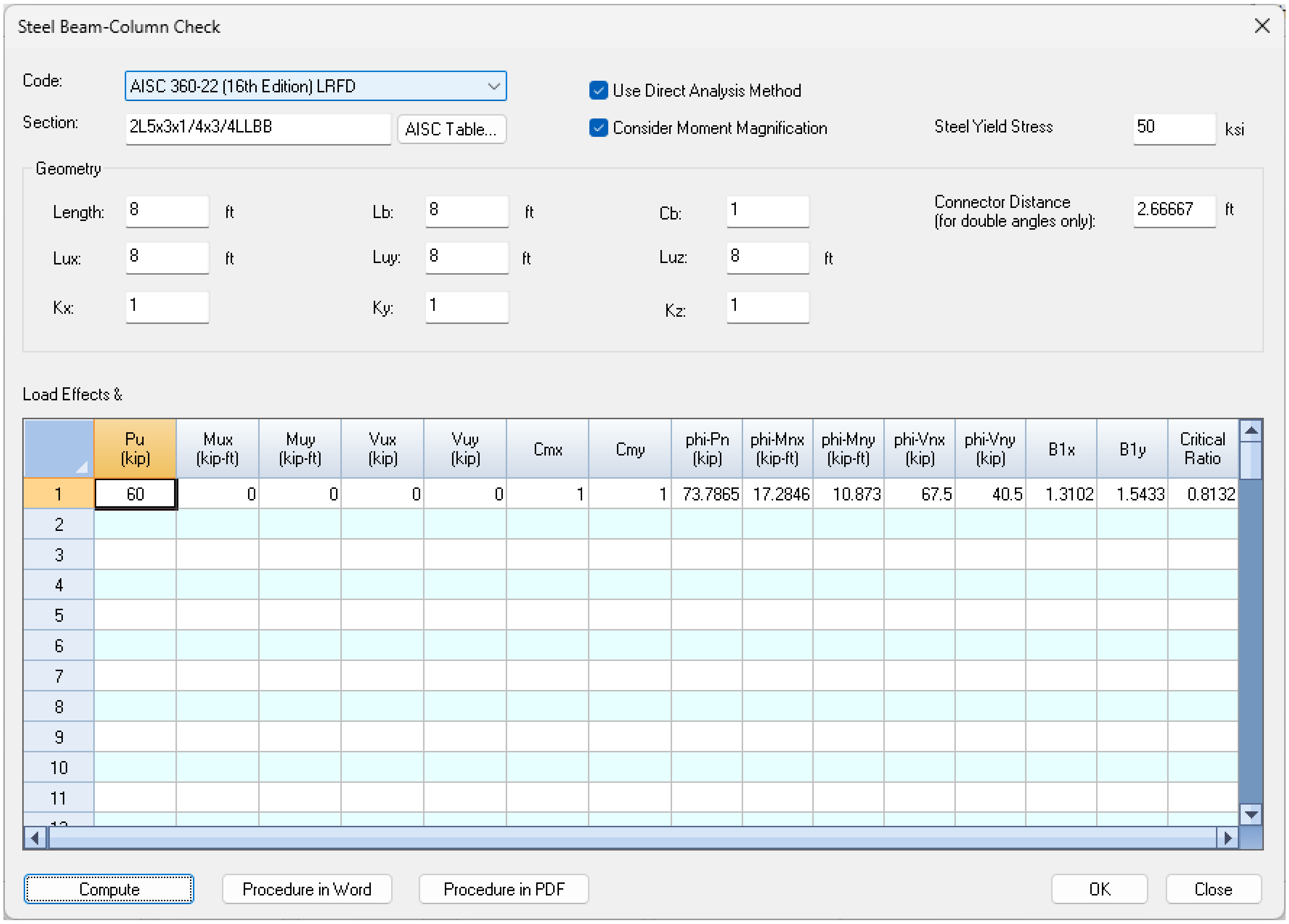


Result Comparison

The reference gives $_{\varphi}$Vnx = 232 kips while sCheck gives $_{\varphi}$Vnx = 232.2 kips. They are practically identical.

Example 13 – Double Angle Shape Beam (Axial Compression)

[Ref 6, Example E.6]
Check the capacity of 2L5x3x1/4x3/4LLBB column in axial compression.
Fy = 50 ksi, L = 8 ft, Kx = Ky = Kz = 1.0, Lux = Luy = Luz = 8 ft.
Connector distance = 32 in = 2.66667 ft.

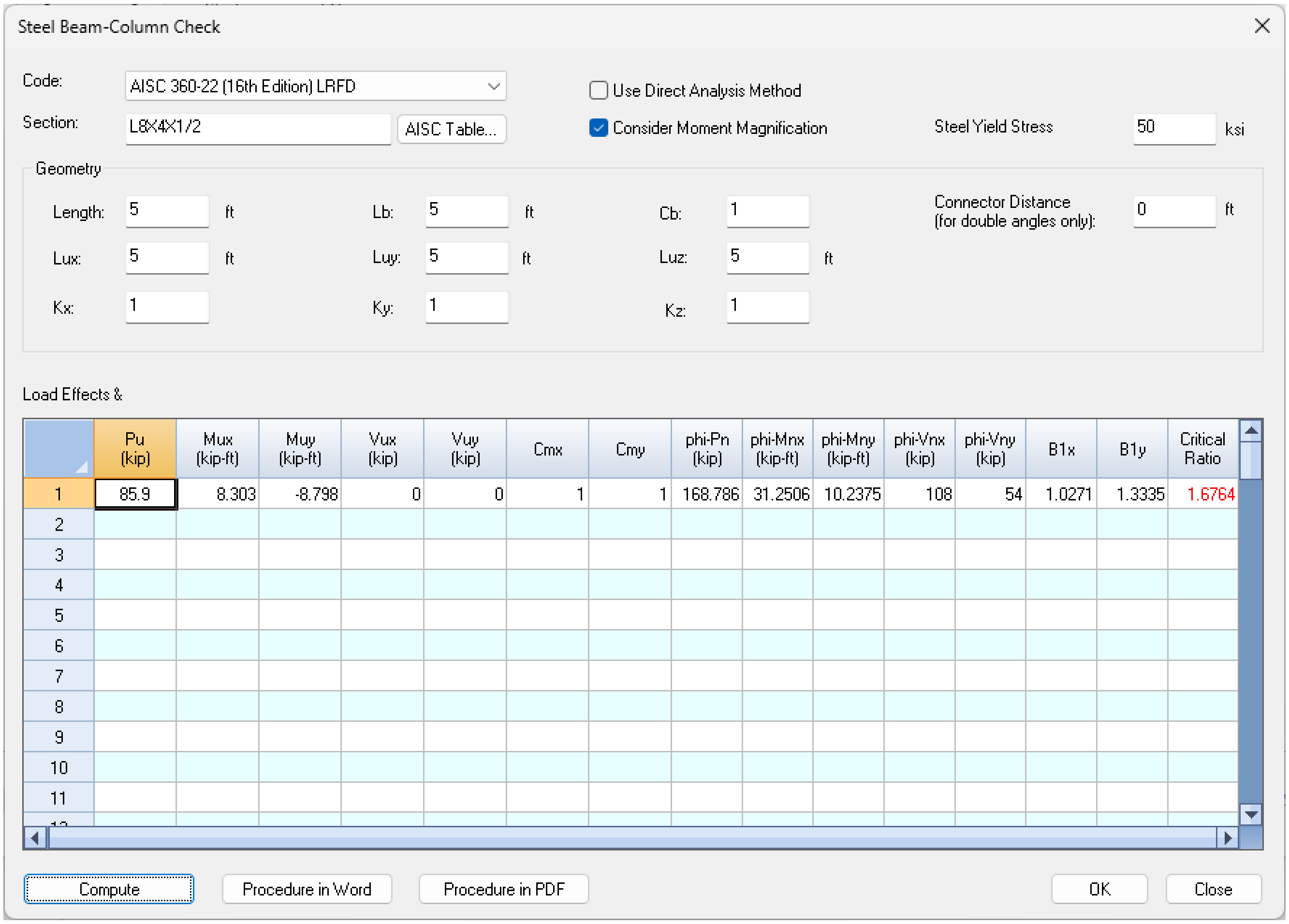


Result Comparison

The reference gives $_{\varphi}Pn$ = 73.8 kips while sCheck gives $_{\varphi}Pn$ = 73.7865 kips. *Please note that L_{cy}/r_y is modified based on AISC Specification Section E6.*

Example 14 – Single Angle Shape Column (Axial Compression and Flexure)

[Ref 6, Example E.14B]
Check the capacity of L8X4X1/2 column in axial compression and flexure.
Fy = 50 ksi, L = 5 ft, Kx = Ky = Kz = 1.0, Lux = Luy = Luz = 5 ft.



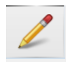
Result Comparison

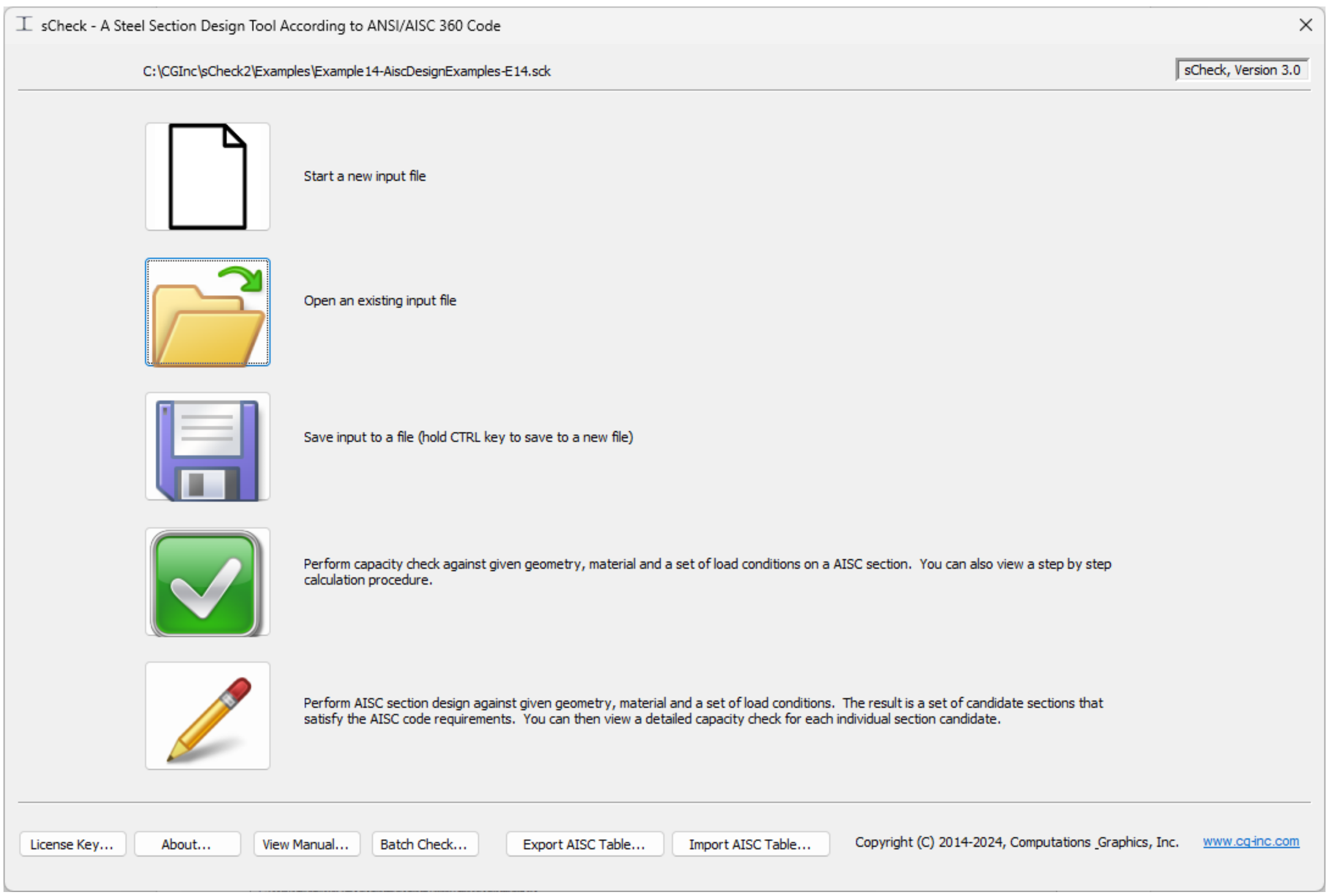
The reference gives $_{\varphi}$Pn = 168 kips, $_{\varphi}$Mnx ($_{\varphi}M_{nw}$) = 375 in-kips = 31.25 ft-kips, $_{\varphi}$Mny ($_{\varphi}M_{nz}$) = 123 in-kips = 10.25 ft-kips while sCheck gives $_{\varphi}$Pn =168.786 kips, $_{\varphi}$Mnx ($_{\varphi}M_{nw}$) = 31.2506 ft-kips, $_{\varphi}$Mny ($_{\varphi}M_{nz}$) = 10.2375 ft-kips. They are practically identical. Please note that for a single angle, the moment capacities are given about the principal w-w and z-z axes and the input moments Mux and Muy are transformed in the principal axes before flexural-axial interaction ratio is checked. In a conservative approach, the program always uses the absolute value for each term in Eq. H2-1, thus we have different critical ratio between sCheck and the reference.

Example 15 – W Shape Beam-Column Design – A Tutorial

[Ref 6, Example H.4]
Design a W8, W10 or W12 shape pinned beam-column that will resist the following conditions.
Fy = 50 ksi, L = 14 ft, Kx = Ky = Kz = 1.0, Lux = Luy = Lb = 14 ft, Cb = 1.14.
Pn = 30 kips, Mux = 90 ft-kips, Muy = 12 ft-kips, Cmx = 1.0, Cmy = 1.0.
The required moments do not include second-order effects.

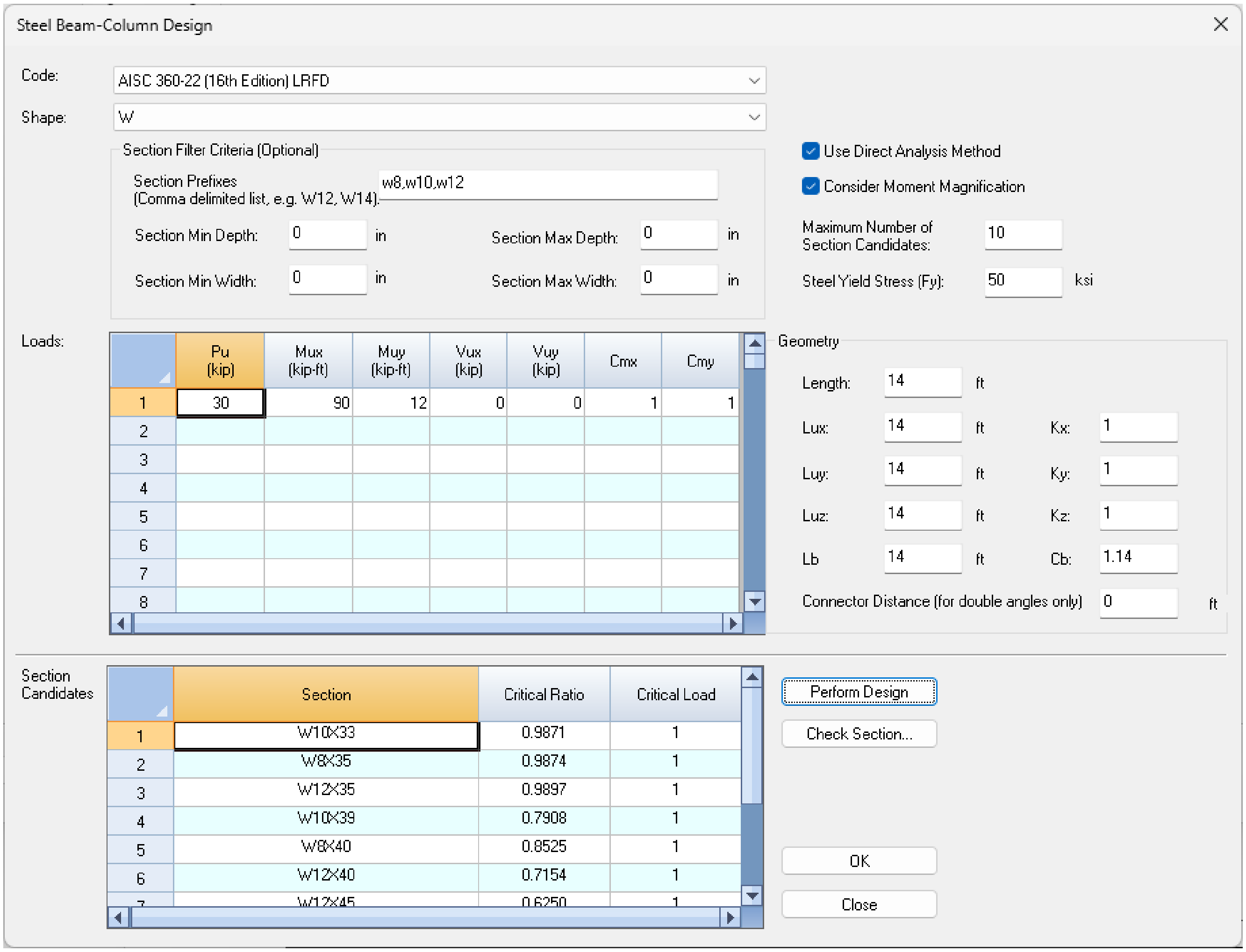
Step 1:

Click on the design button  from the sCheck main screen.

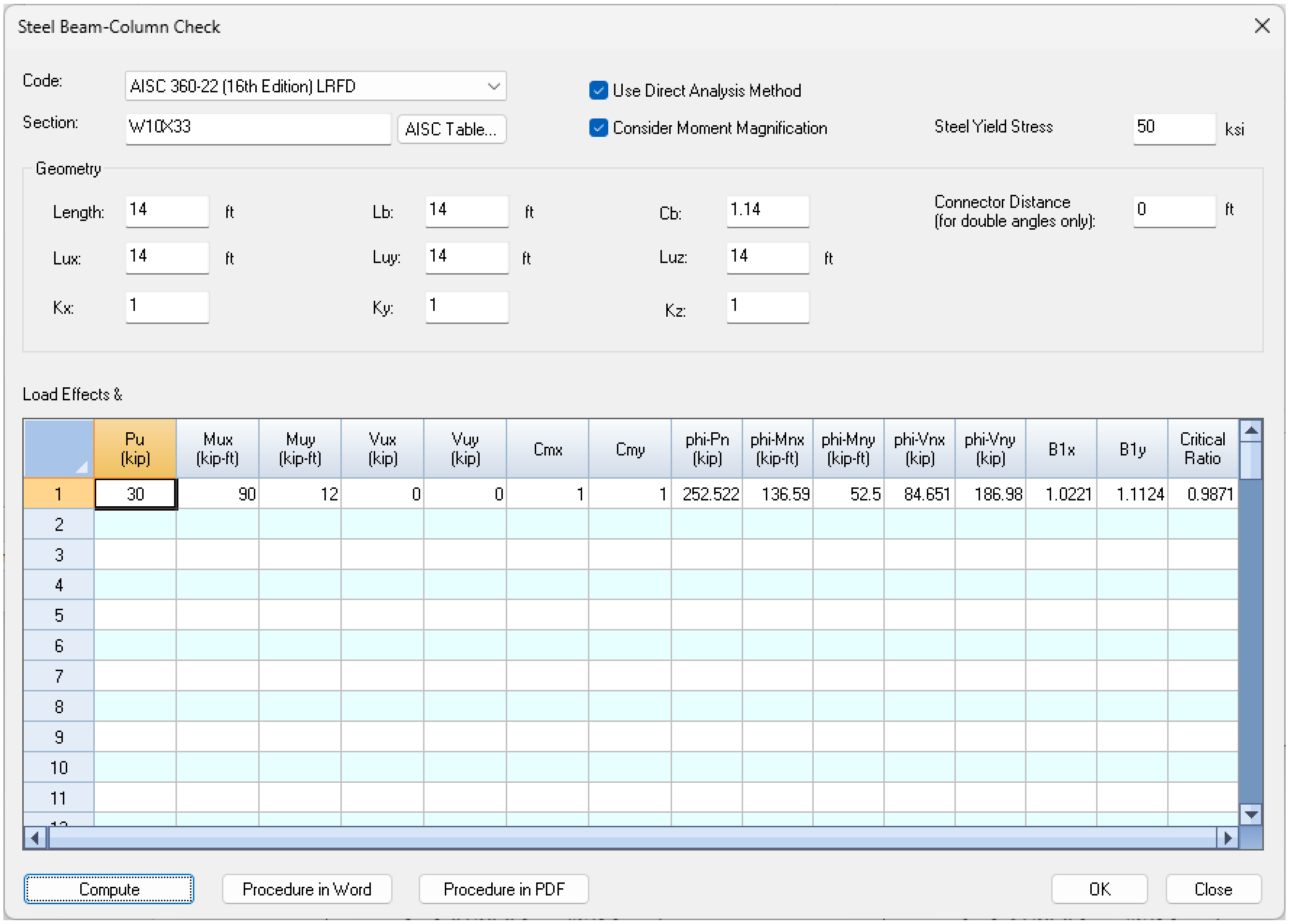


Step 2:

Enter the geometry, load condition and design criteria on the “Steel Beam-Column Design” dialog as following and click on “Perform Design”. By default, 10 section candidates will be chosen by the program. The sections are ordered in terms of section weight by linear foot.



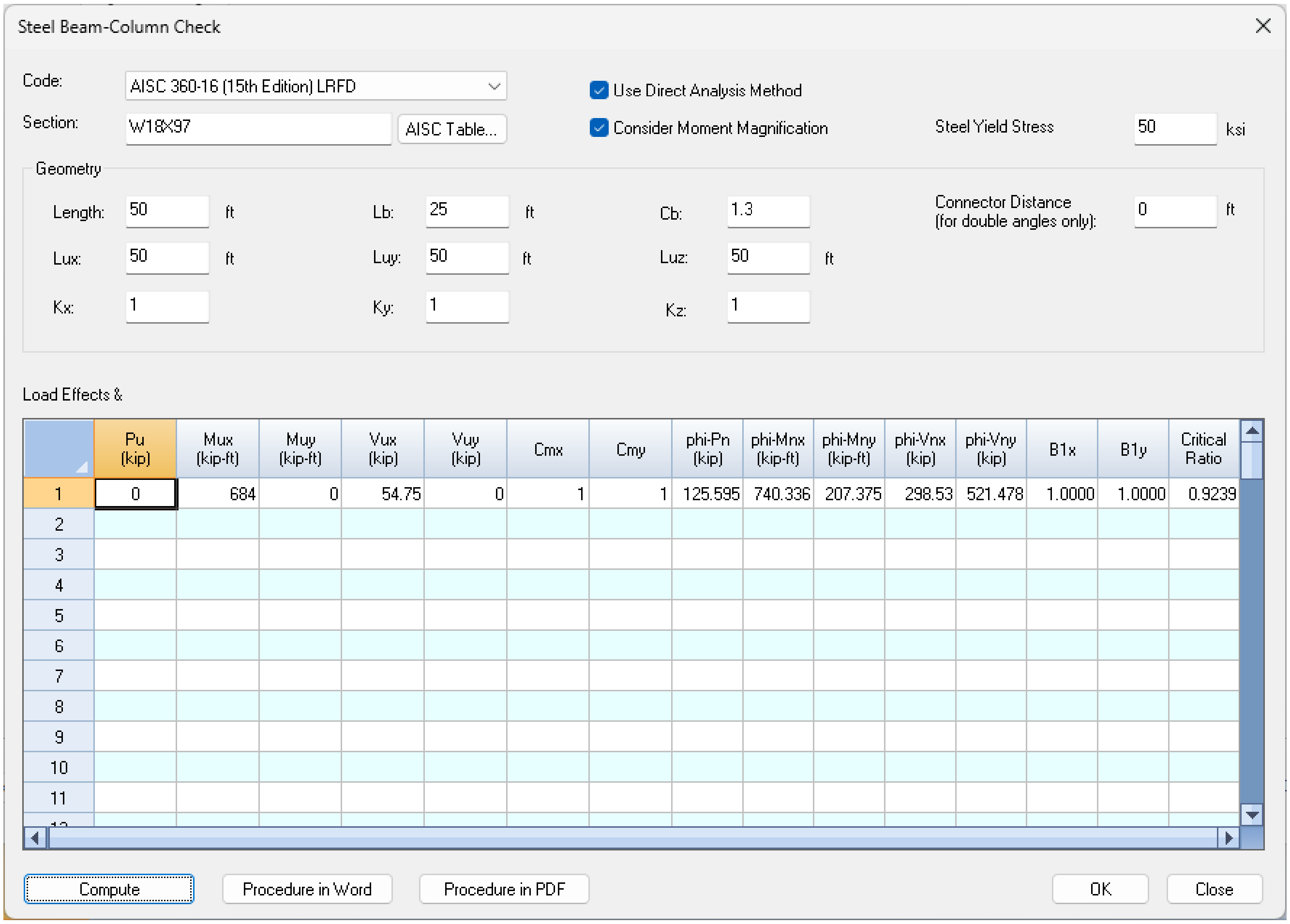
Step 3:
The first section candidate is W10x33, which is the one given by the reference. We can view the detailed check on a section candidate by first selecting the section candidate, then clicking on “Check Section” button on the “Steel Beam-Column Design” dialog above. The following is the “Steel Beam-Column Check” dialog. From here we are able to view very detailed calculation procedure by clicking on “Procedure in Word” or “Procedure in PDF”.



Chapter 6: *Verification Examples – AISC 360-16 (15th edition) LRFD*

Example 1 – W Shape Beam (Flexure in Major Axis, Lp < Lb < Lr)

[Ref 2, pp435]
Check the flexural capacity of W18x97 beam.
Fy = 50 ksi, L = 50 ft, Lb = 25 ft. Cb = 1.3.

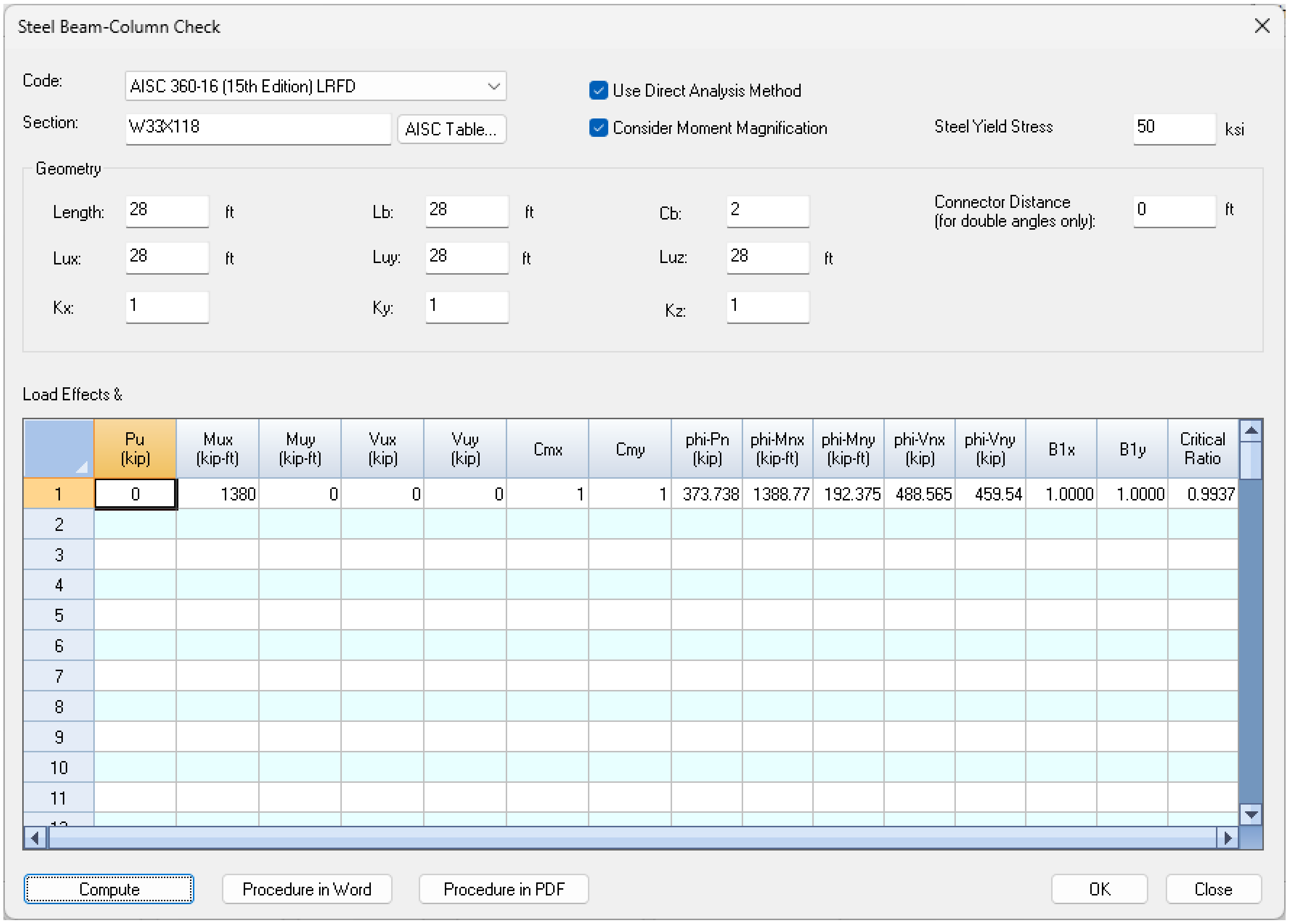


Result Comparison

The reference gives $_{\varphi}$Mnx = 740 ft-kips while sCheck gives $_{\varphi}$Mnx = 740.336 ft-kips. They are practically identical.

Example 2 – W Shape Beam (Flexure in Major Axis, Lb > Lr)

[Ref 2, pp440]
Check the flexural capacity of W33x118 beam.
Fy = 50 ksi, L = 28 ft, Lb = 28ft, Cb = 2.0.

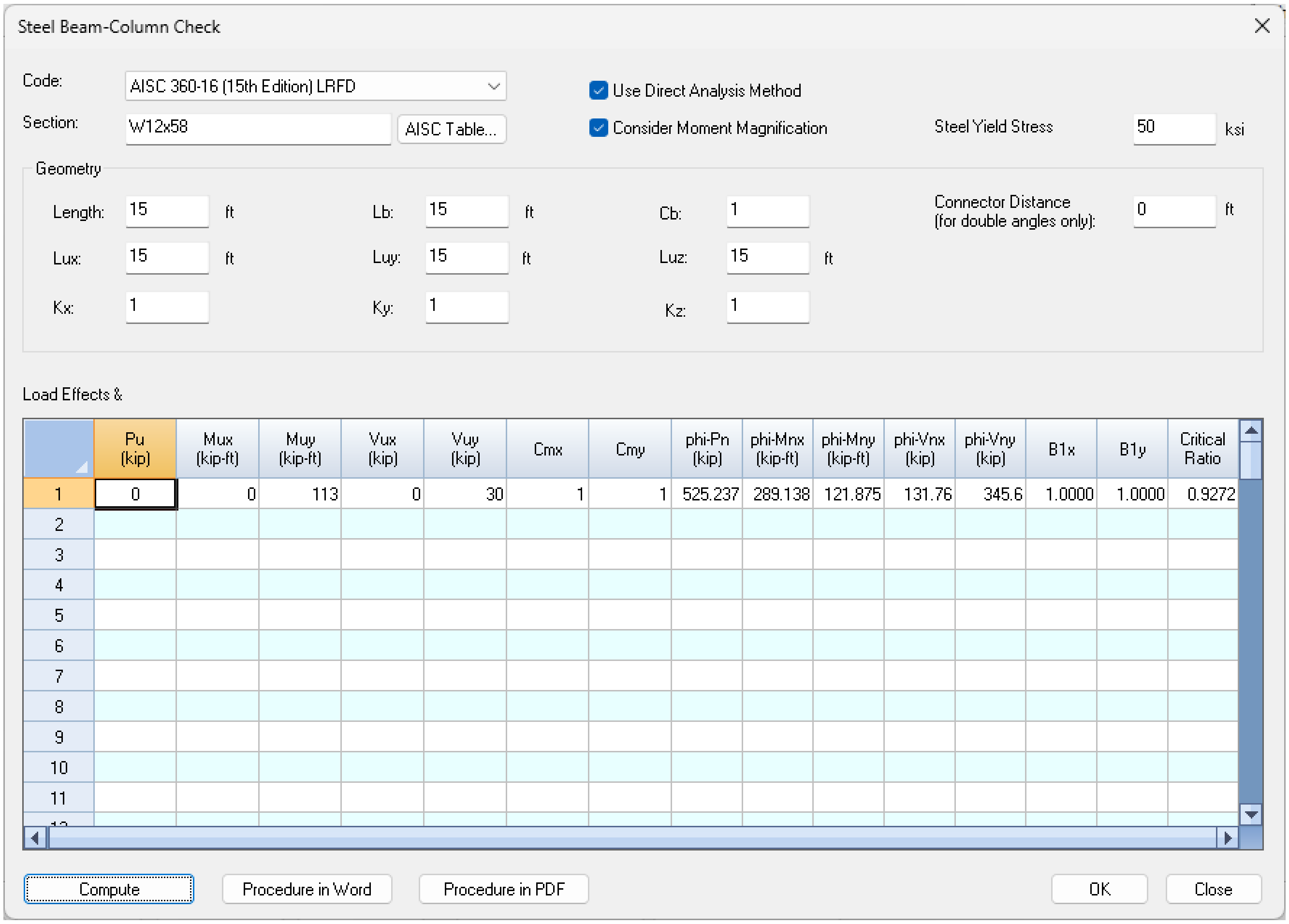


Result Comparison

The reference gives $_{\varphi}$Mnx = 1390 ft-kips while sCheck gives $_{\varphi}$Mnx = 1388.77 ft-kips. They are practically identical.

Example 3 – W Shape Beam (Flexure in Minor Axis)

[Ref 4, Example F.5]
Check the flexural capacity of W12x58 beam in minor axis. Fy = 50 ksi, L = 15 ft.

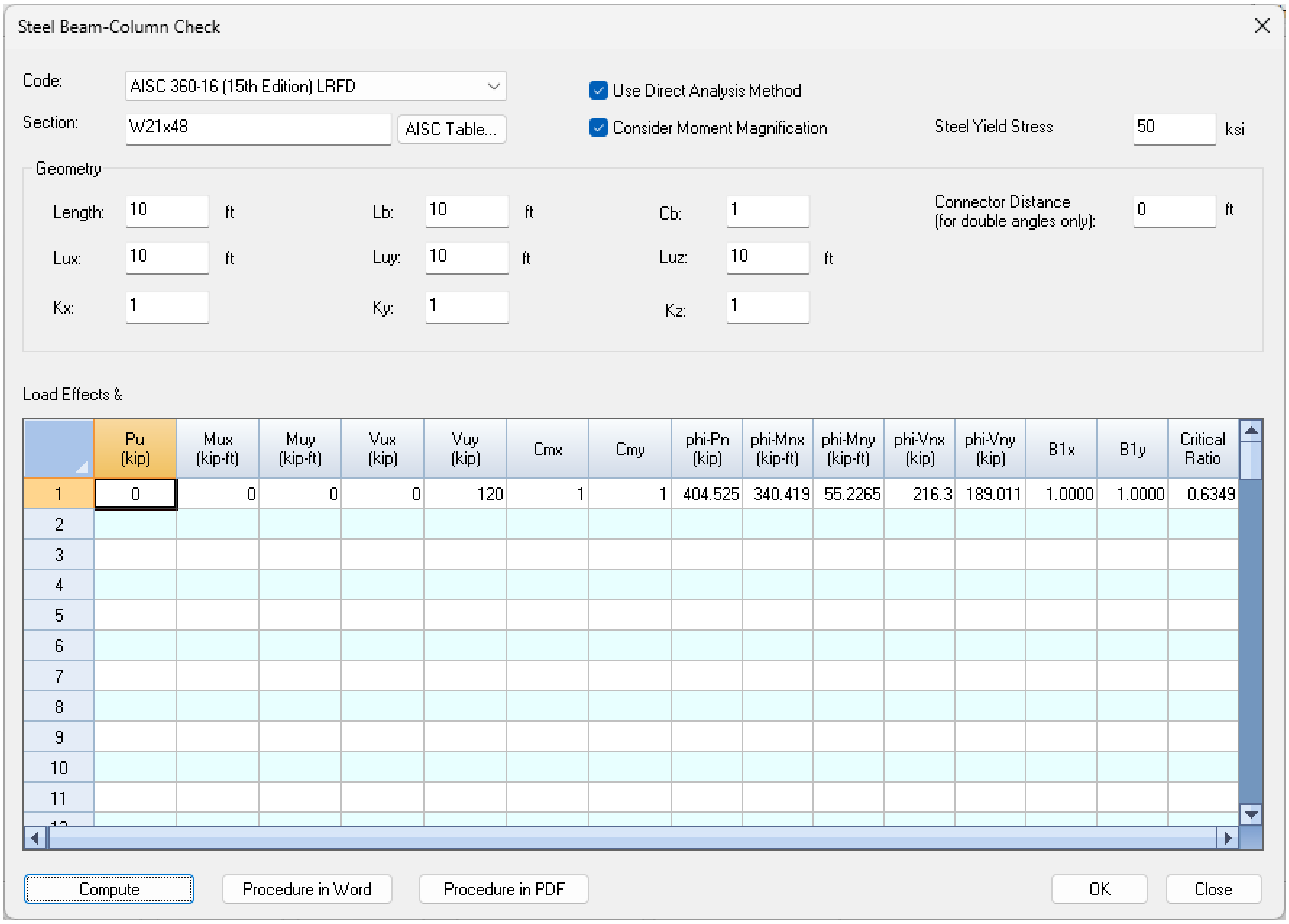


Result Comparison

The reference gives $_{\varphi}$Mny = 122 ft-kips while sCheck gives $_{\varphi}$Mny = 121.875 ft-kips. They are practically identical.

Example 4 – W Shape Beam (Shear in Minor Axis)

[Ref 4, Example G.6]
Check the shear capacity of W21x48 beam in minor axis.
Fy = 50 ksi

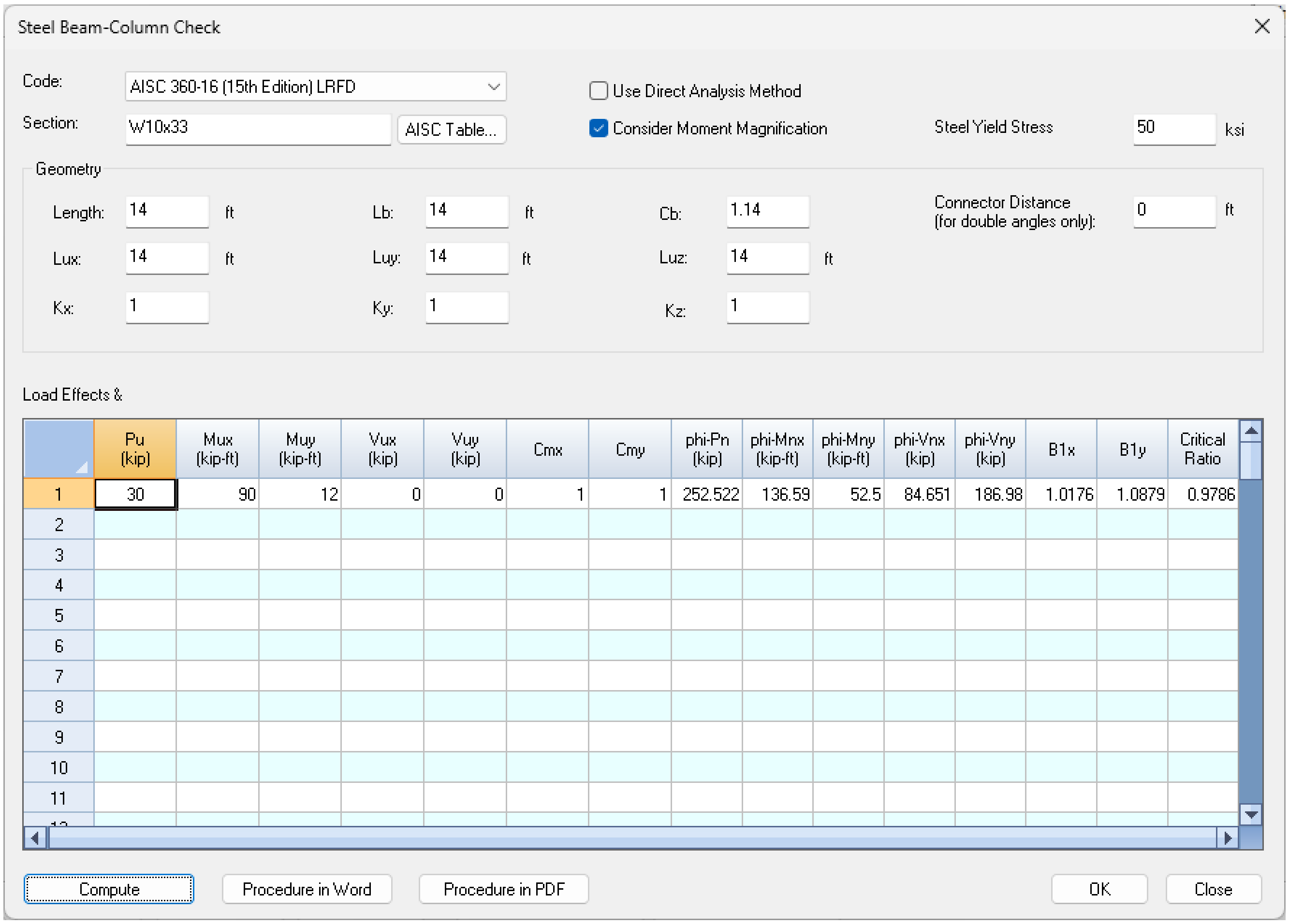


Result Comparison

The reference gives $_{\varphi}$Vny = 189 ft-kips while sCheck gives $_{\varphi}$Vny = 189.011 ft-kips. They are practically identical.

Example 5 – W Shape Column (Combined Axial Compression and Flexure)

[Ref 4, Example H.4]
Check the capacity of W10x33 column in combined axial compression and flexure.
Fy = 50 ksi, L = 14 ft, Kx = Ky = Kz = 1.0, Lux = Luy = Lb = 14 ft, Cb = 1.14.

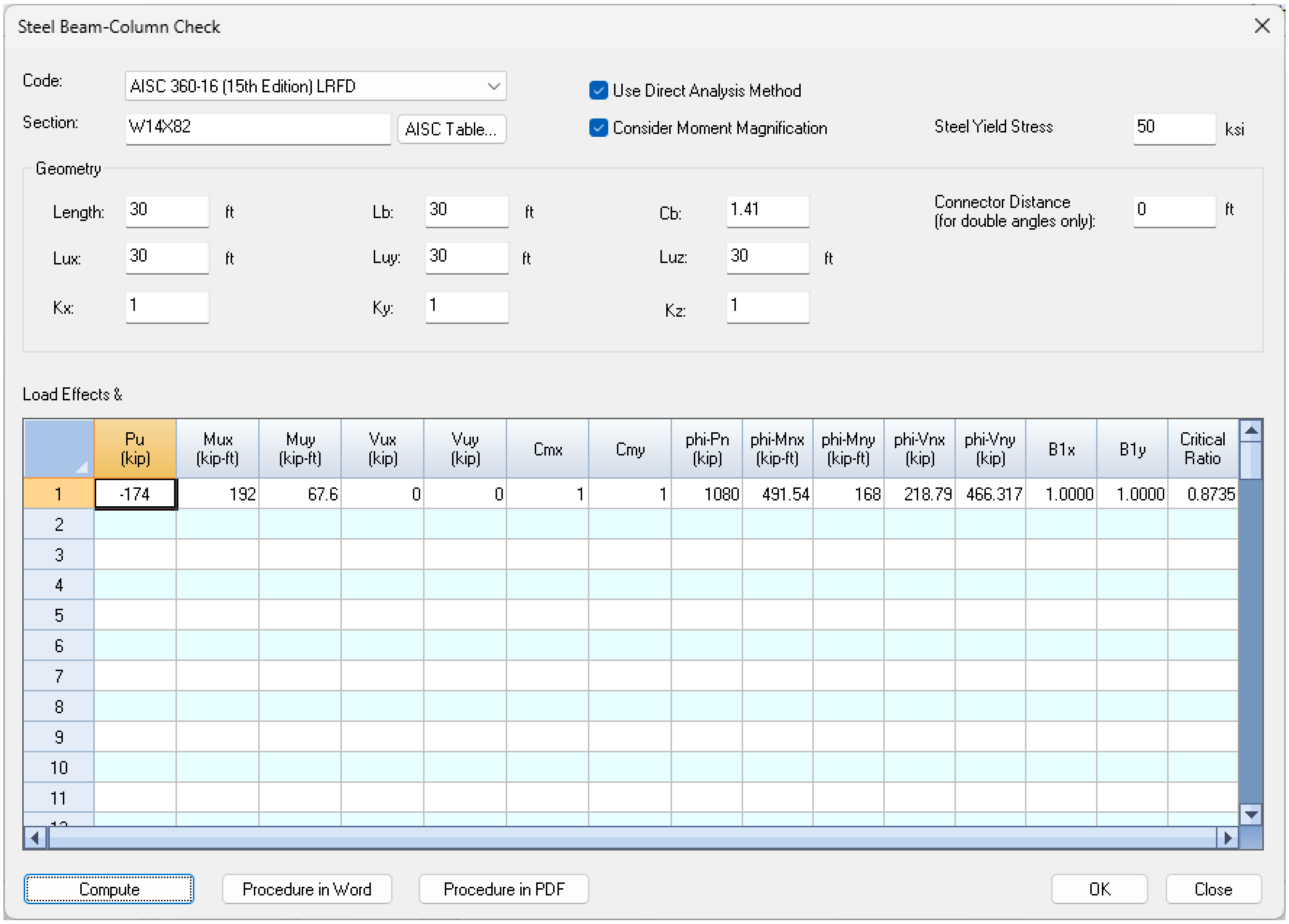


Result Comparison

The reference gives $_{\varphi}$Pn = 253 kips, $_{\varphi}$Mnx = 137 ft-kips, $_{\varphi}$Mny = 52.5 ft-kips while sCheck gives $_{\varphi}$Pn = 252.522 kips, $_{\varphi}$Mnx = 136.59 ft-kips, $_{\varphi}$Mny = 52.5 ft-kips. They are practically identical.

Example 6 – W Shape Column (Combined Axial Tension and Flexure)

[Ref 4, Example H.3]
Check the capacity of W14x82 column in combined axial tension and flexure.
Fy = 50 ksi, L = 30 ft, Kx = Ky = Kz = 1.0, Lux = Luy = Lb = 30 ft, Cb = 1.41.
Note: tension load must be entered as negative value.

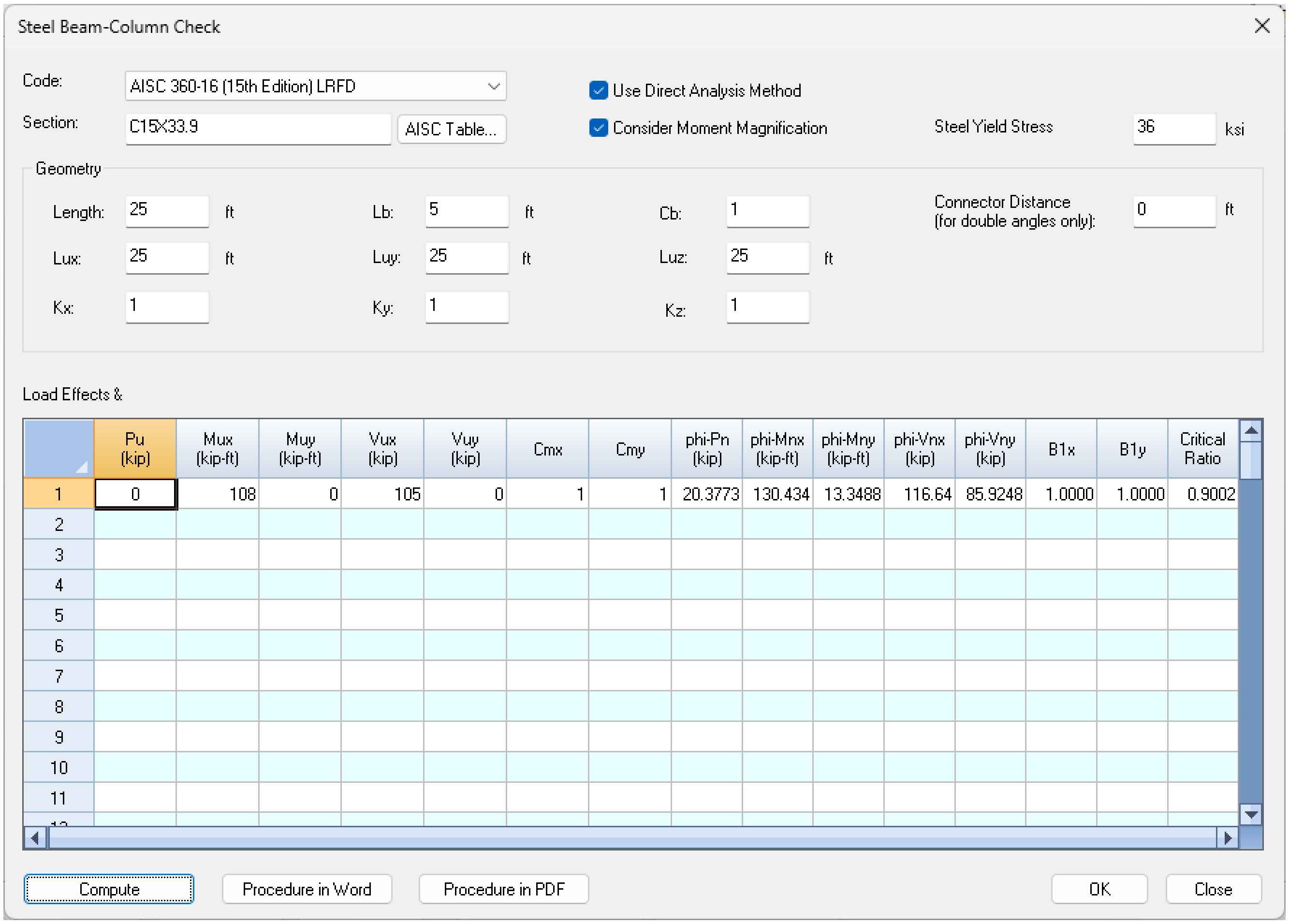


Result Comparison

The reference gives $_{\varphi}$Pn = 1080 kips, $_{\varphi}$Mnx = 492 ft-kips, $_{\varphi}$Mny = 168 ft-kips while sCheck gives $_{\varphi}$Pn = 1080 kips, $_{\varphi}$Mnx = 491.54 ft-kips, $_{\varphi}$Mny =168 ft-kips. They are practically identical. Please note that absolute value of axial load should be used for axial-flexural interaction formula.

Example 7 – C Shape Beam (Flexure and Shear)

[Ref 4, Example F.2-2A and G.2B]
Check the capacity of C15x33.9 column in flexure and shear.
Fy = 36 ksi, L = 25 ft, Lb = 5 ft, Cb = 1.0.

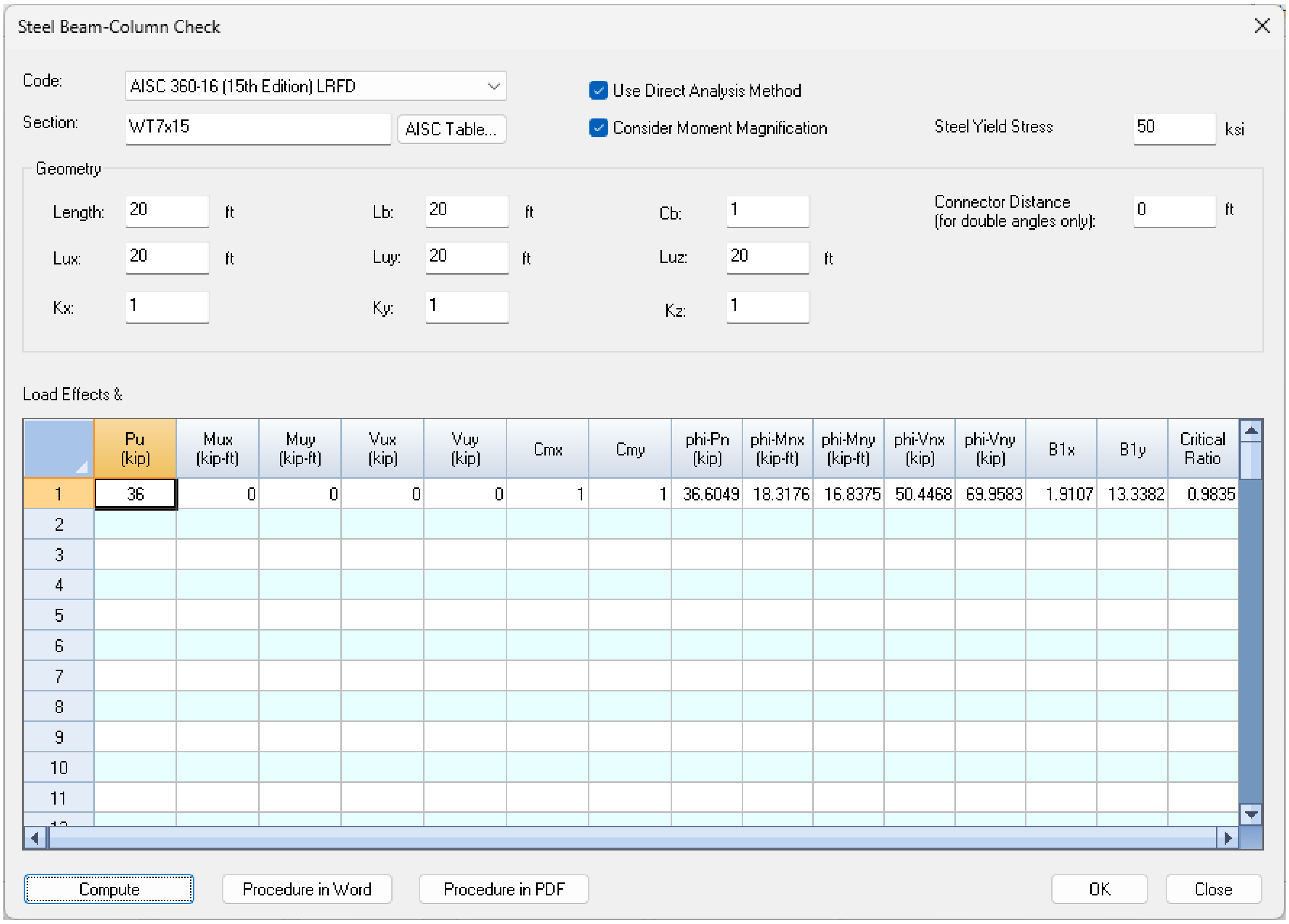


Result Comparison

The reference gives $_{\varphi}$Mnx = 130 ft-kips, $_{\varphi}$Vnx = 117 kips while sCheck gives $_{\varphi}$Mnx = 130.434 ft-kips, $_{\varphi}$Vnx = 116.64 kips. They are practically identical.

Example 8 – WT Shape Column (Axial Compression)

[Ref 4, Example E.8]
Check the capacity of WT7x15 column in axial compression.
Fy = 50 ksi, L = 20 ft, Kx = Ky = Kz = 1.0, Lu = 20 ft, Cb = 1.41.

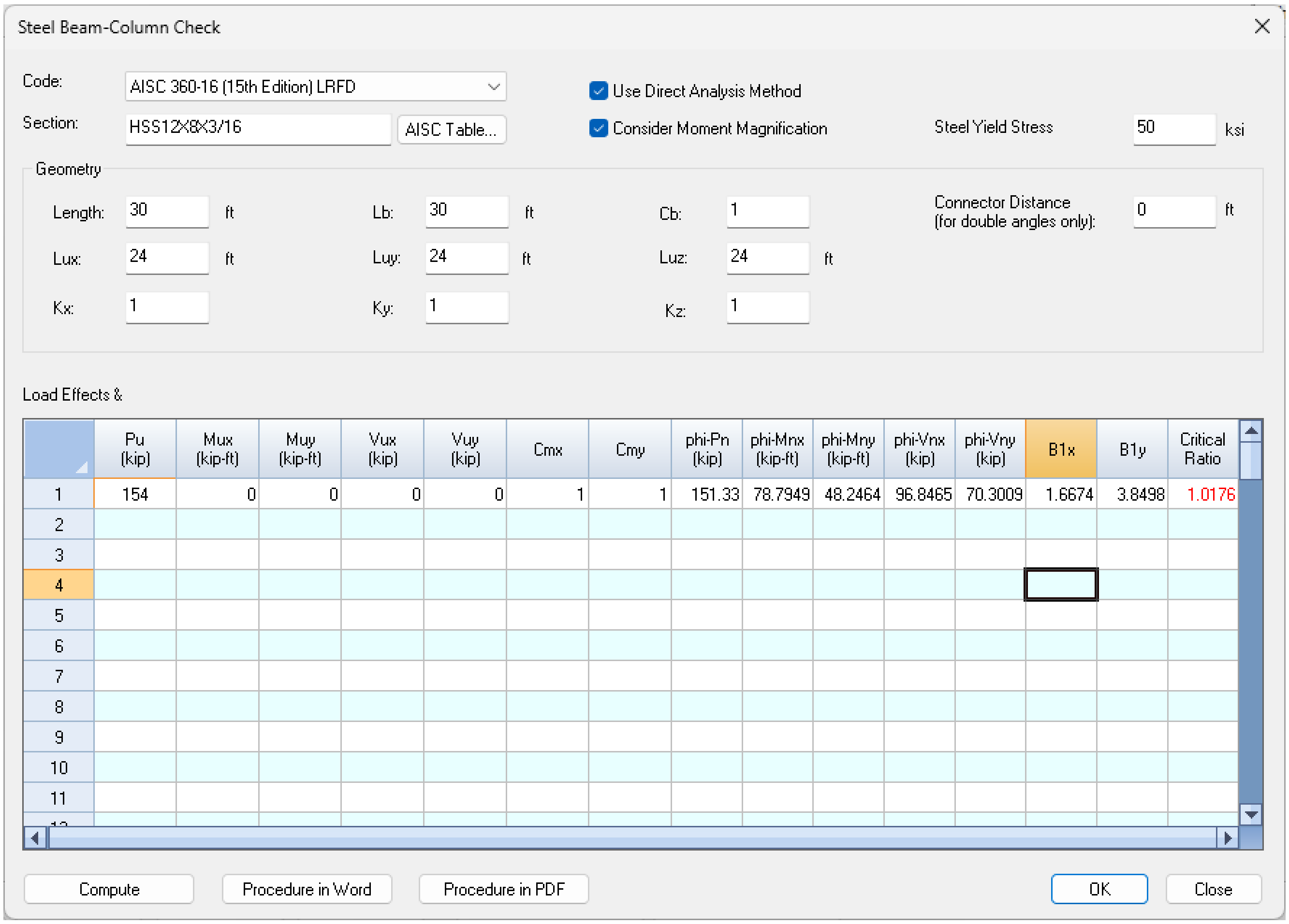


Result Comparison

The reference gives $_{\varphi}$Pn = 36.6 kips while sCheck gives $_{\varphi}$Pn = 36.6049 kips. They are practically identical.

Example 9 – HSS Shape Column (Axial Compression)

[Ref 4, Example E.10]
Check the capacity of HSS12x8x3/16 column in axial compression.
Fy = 50 ksi, L = 30 ft, Kx = Ky = 0.8, Kz = 1.0, Lu = 30 ft, Cb = 1.0.

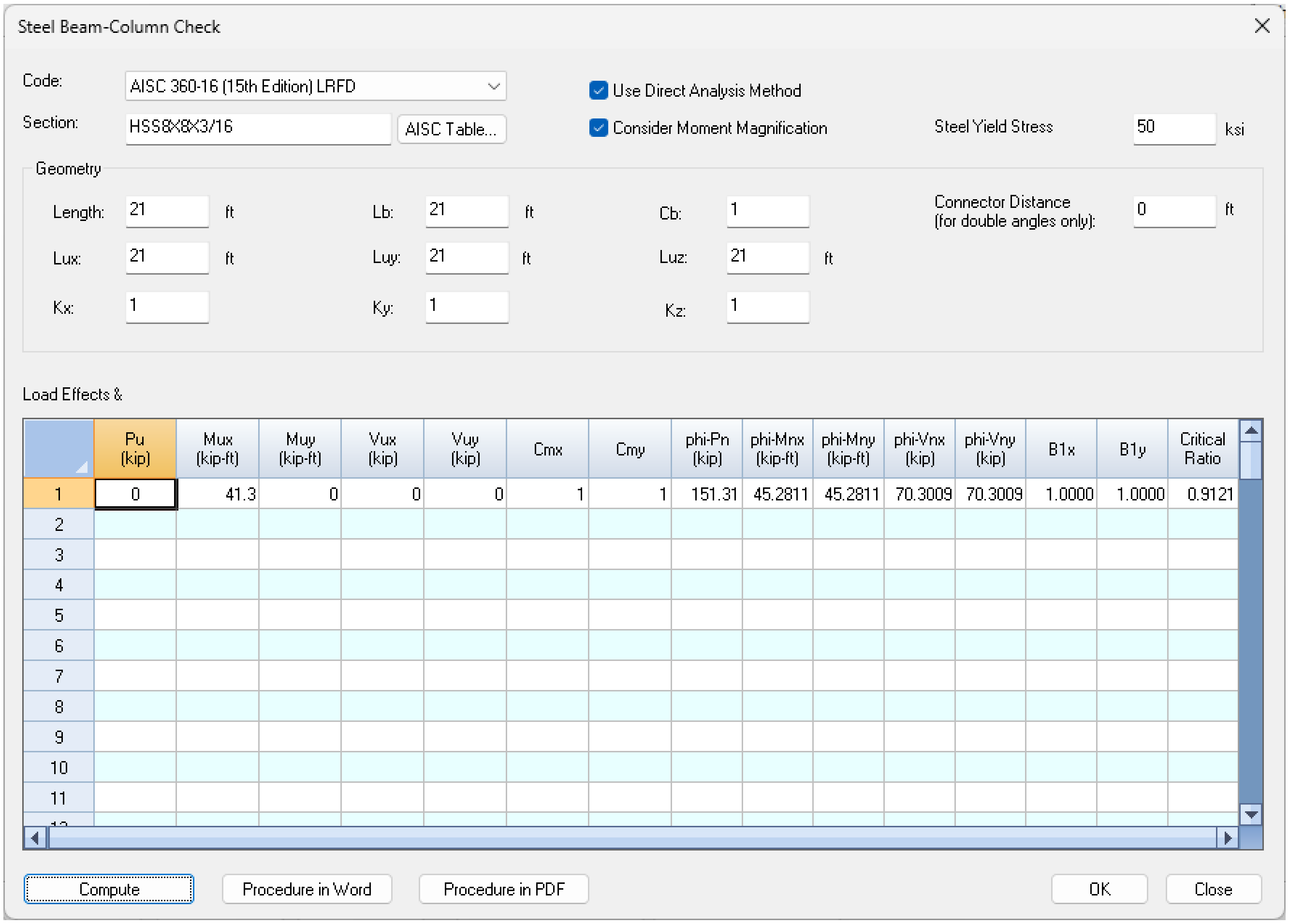


Result Comparison

The reference gives $_{\varphi}$Pn = 151 kips while sCheck gives $_{\varphi}$Pn = 151.33 kips. They are practically identical. The reference gives $_{\varphi}$Pn = 177 kips while sCheck gives $_{\varphi}$Pn = 177.832 kips for Lc = 18 ft. The reference gives $_{\varphi}$Pn = 74.2 kips while sCheck gives $_{\varphi}$Pn = 74.3864 kips for Lc = 40 ft.

Example 10 – Rectangular HSS Shape Beam (Flexure)

[Ref 4, Example F.8B]
Check the capacity of HSS8x8x3/16 column in flexure.
Fy = 50 ksi, L = 21 ft, Lb = 21 ft, Cb = 1.0.

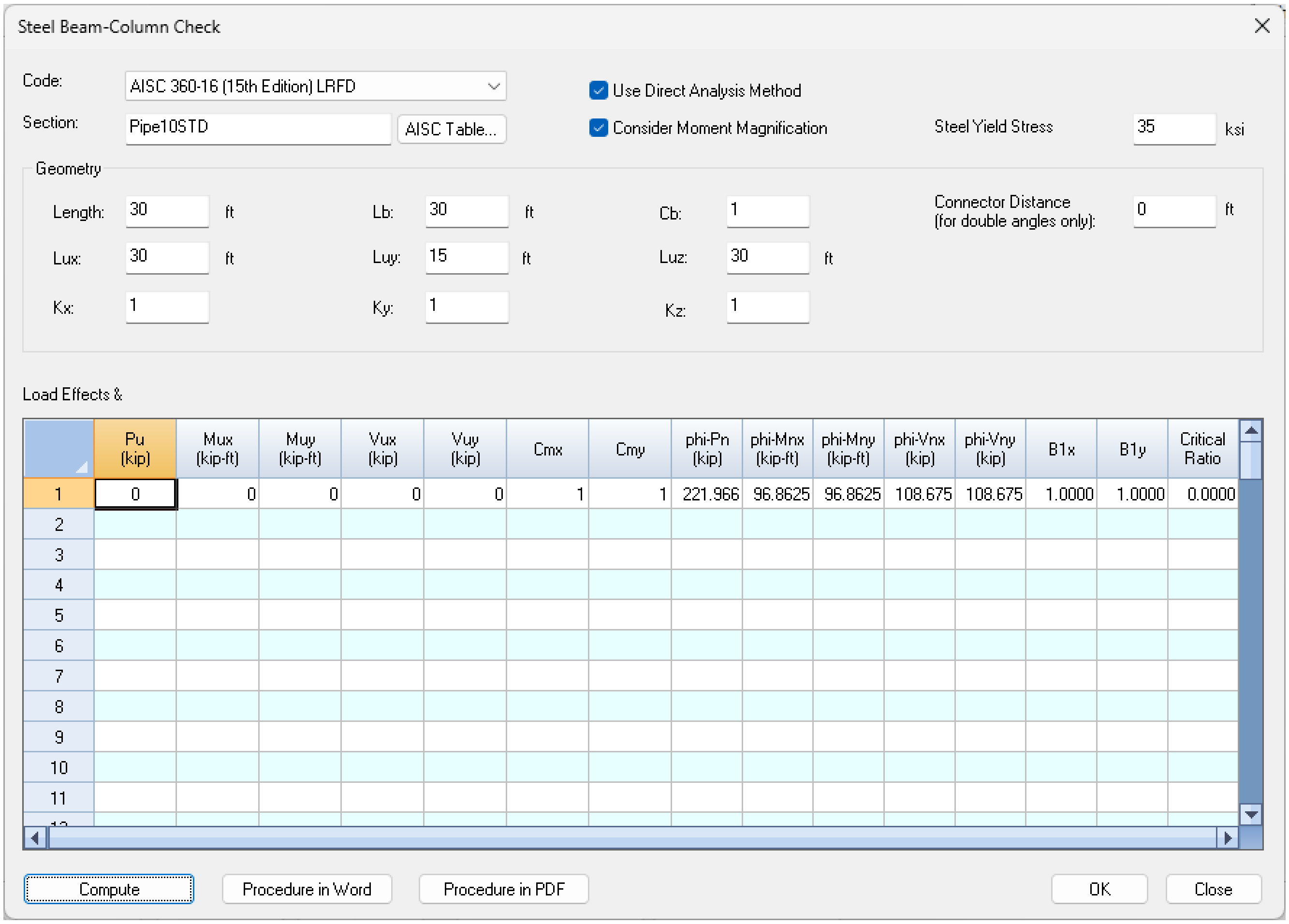


Result Comparison

The reference gives $_{\varphi}$Mnx = 45.4 ft-kips while sCheck gives $_{\varphi}$Mnx = 45.2811 ft-kips. They are practically identical.

Example 11 – Pipe Shape Column (Compression)

[Ref 4, Example E.11]
Check the capacity of Pipe10STD column in flexure.
Fy = 35 ksi, L = 30 ft

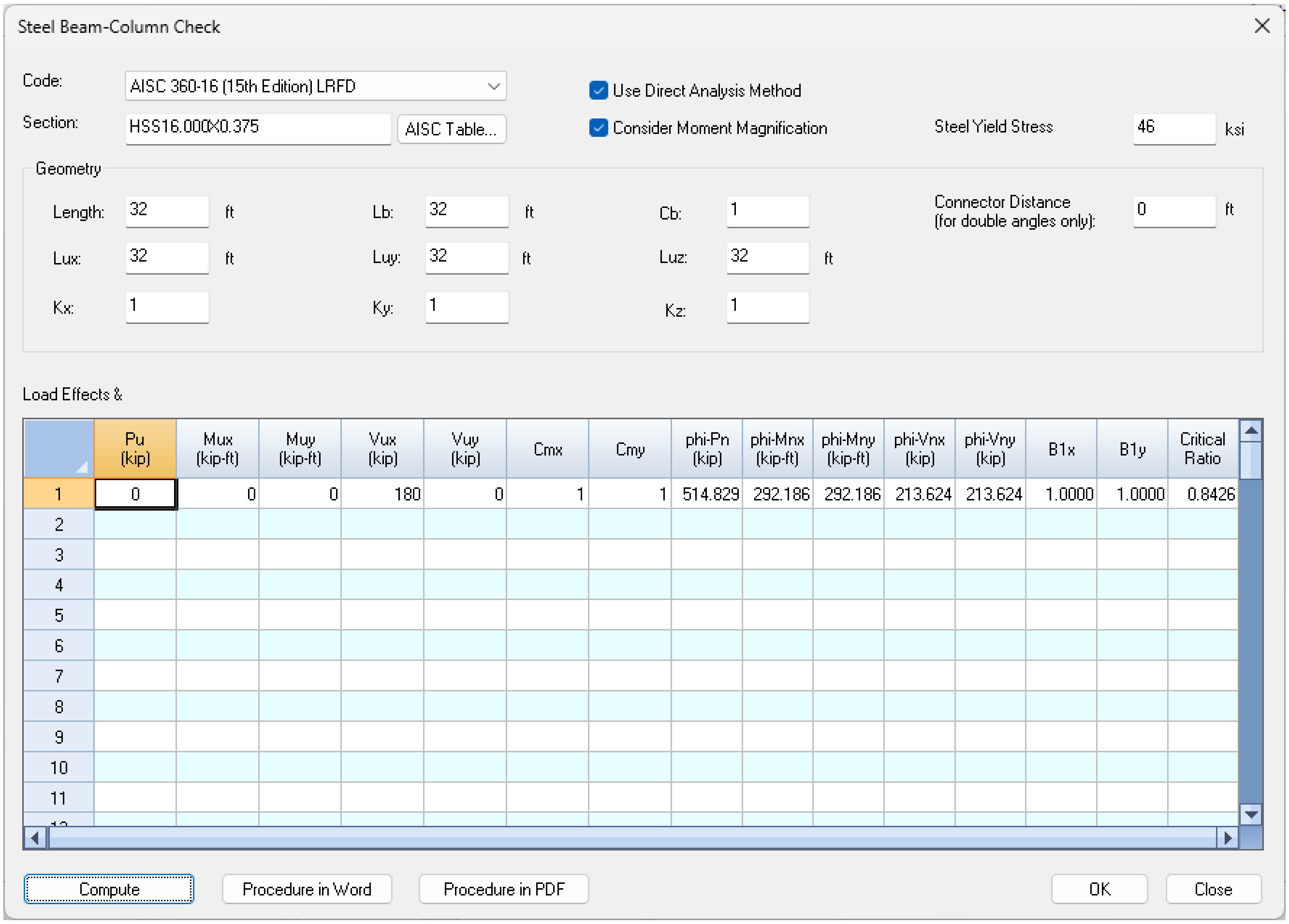


Result Comparison

The reference gives $_{\varphi}$Pn = 222 kips while sCheck gives $_{\varphi}$Pn = 221.966 kips. They are practically identical.

Example 12 – Round HSS Shape Beam (Shear)

[Ref 4, Example G.5]
Check the capacity of HSS16.000X0.375 column in shear.
Fy = 46 ksi, L = 32 ft

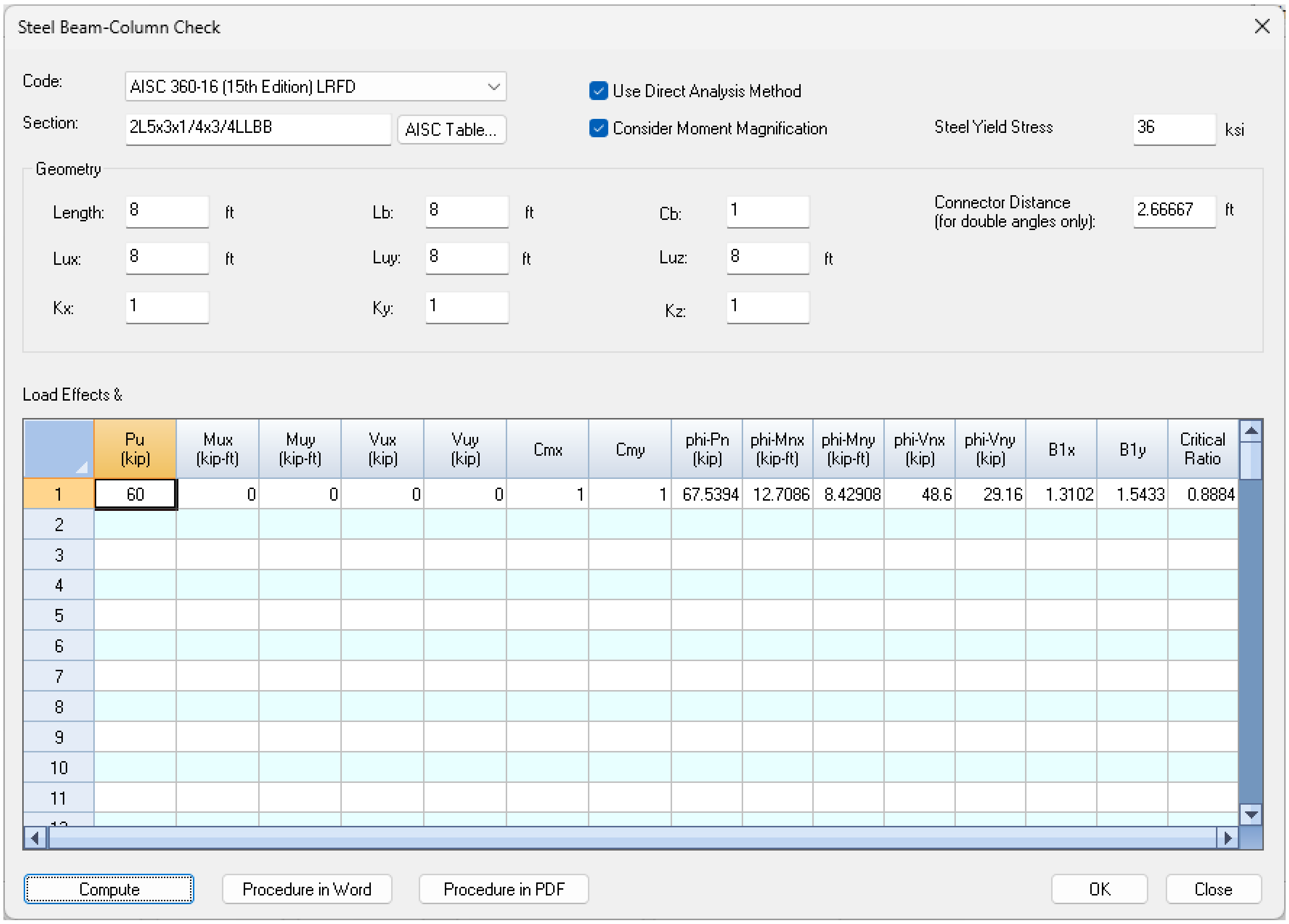


Result Comparison

The reference gives $_{\varphi}$Vnx = 213 kips while sCheck gives $_{\varphi}$Vnx = 213.624 kips. They are practically identical.

Example 13 – Double Angle Shape Beam (Axial Compression)

[Ref 4, Example E.6]
Check the capacity of 2L5x3x1/4x3/4LLBB column in axial compression.
Fy = 36 ksi, L = 8 ft, Kx = Ky = Kz = 1.0, Lux = Luy = Luz = 8 ft.
Connector distance = 32 in = 2.66667 ft.

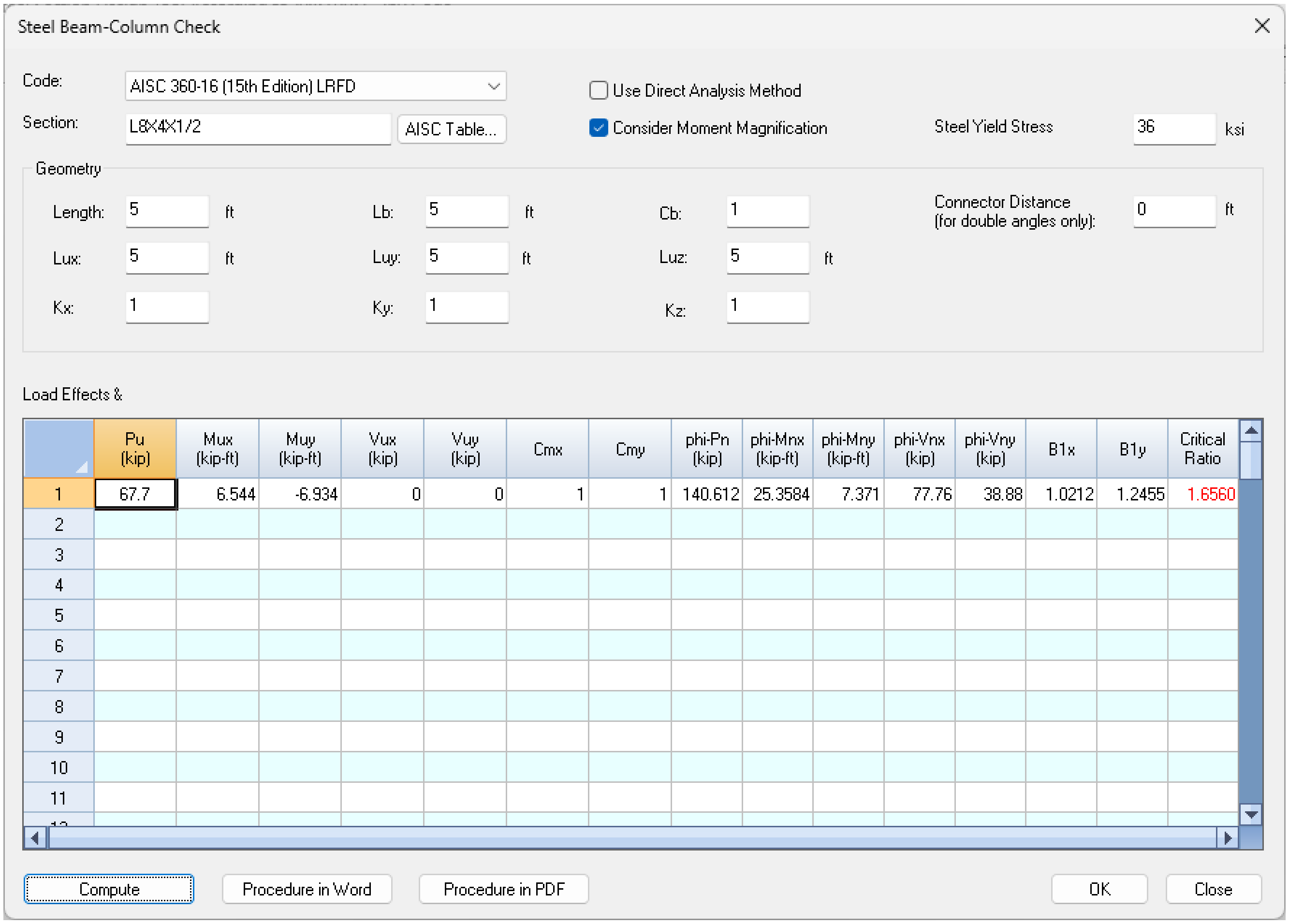


Result Comparison

The reference gives $_{\varphi}$Pn = 67.5 kips while sCheck gives $_{\varphi}$Pn = 67.5394 kips. *Please note that L_{cy}/r_y is modified based on AISC Specification Section E6.*

Example 14 – Single Angle Shape Column (Axial Compression and Flexure)

[Ref 4, Example E.14]
Check the capacity of L8X4X1/2 column in axial compression and flexure.
Fy = 36 ksi, L = 5 ft, Kx = Ky = Kz = 1.0, Lux = Luy = Luz = 5 ft.



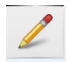
Result Comparison

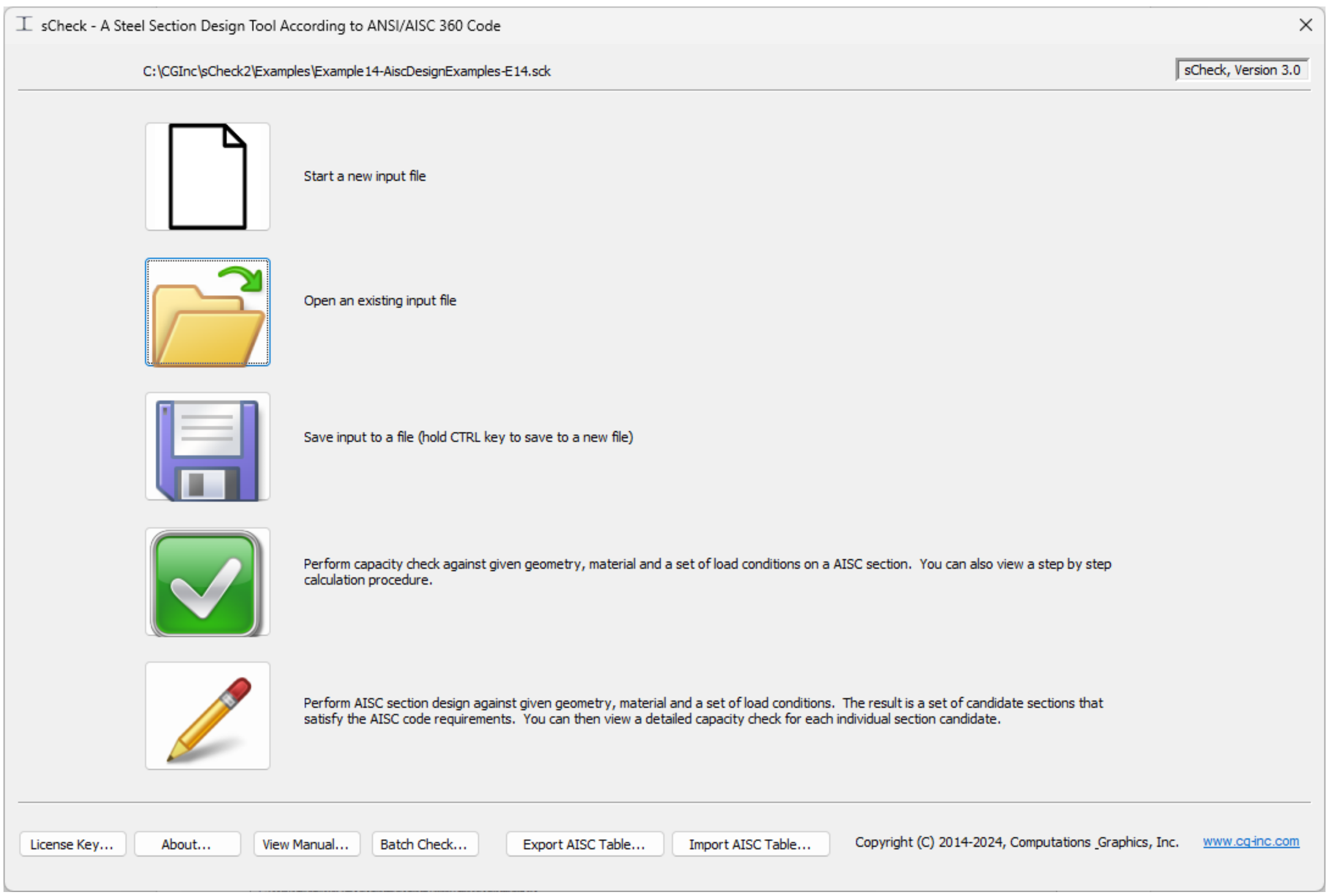
The reference gives $_{\varphi}$Pn = 141 kips, $_{\varphi}$Mnx ($_{\varphi}M_{nw}$) = 304 in-kips = 25.33 ft-kips, $_{\varphi}$Mny ($_{\varphi}M_{nz}$) = 88.5 in-kips = 7.375 ft-kips while sCheck gives $_{\varphi}$Pn = 140.612 kips, $_{\varphi}$Mnx ($_{\varphi}M_{nw}$) = 25.3584 ft-kips, $_{\varphi}$Mny ($_{\varphi}M_{nz}$) = 7.371 ft-kips. They are practically identical. Please note that for a single angle, the moment capacities are given about the principal w-w and z-z axes and the input moments Mux and Muy are transformed in the principal axes before flexural-axial interaction ratio is checked. In a conservative approach, the program always uses the absolute value for each term in Eq. H2-1, thus we have different critical ratio between sCheck and the reference.

Example 15 – W Shape Beam-Column Design – A Tutorial

[Ref 4, Example H.4]
Design a W8, W10 or W12 shape pinned beam-column that will resist the following conditions.
Fy = 50 ksi, L = 14 ft, Kx = Ky = Kz = 1.0, Lux = Luy = Lb = 14 ft, Cb = 1.14.
Pn = 30 kips, Mux = 90 ft-kips, Muy = 12 ft-kips, Cmx = 1.0, Cmy = 1.0.
The required moments do not include second-order effects.

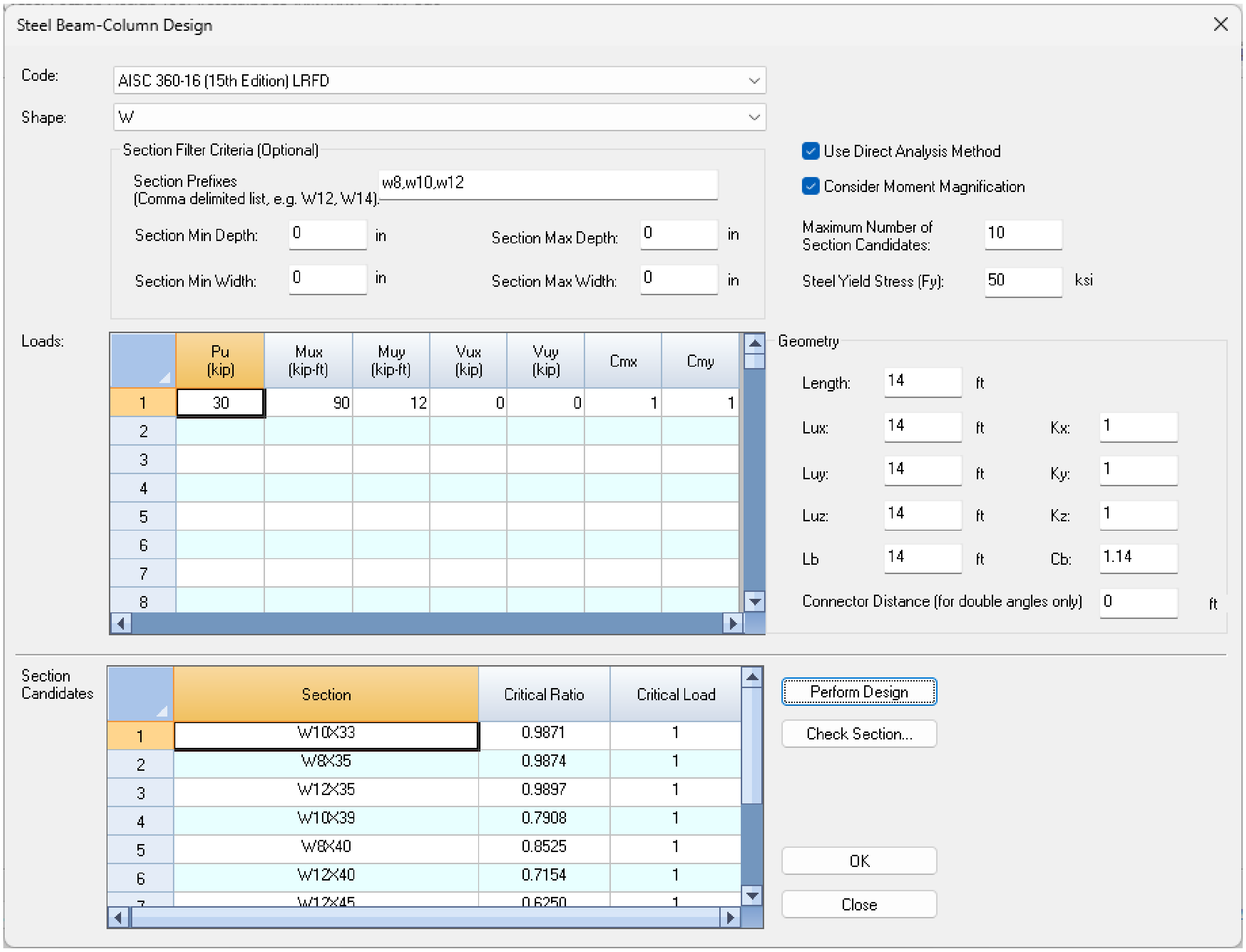
Step 1:

Click on the design button  from the sCheck main screen.

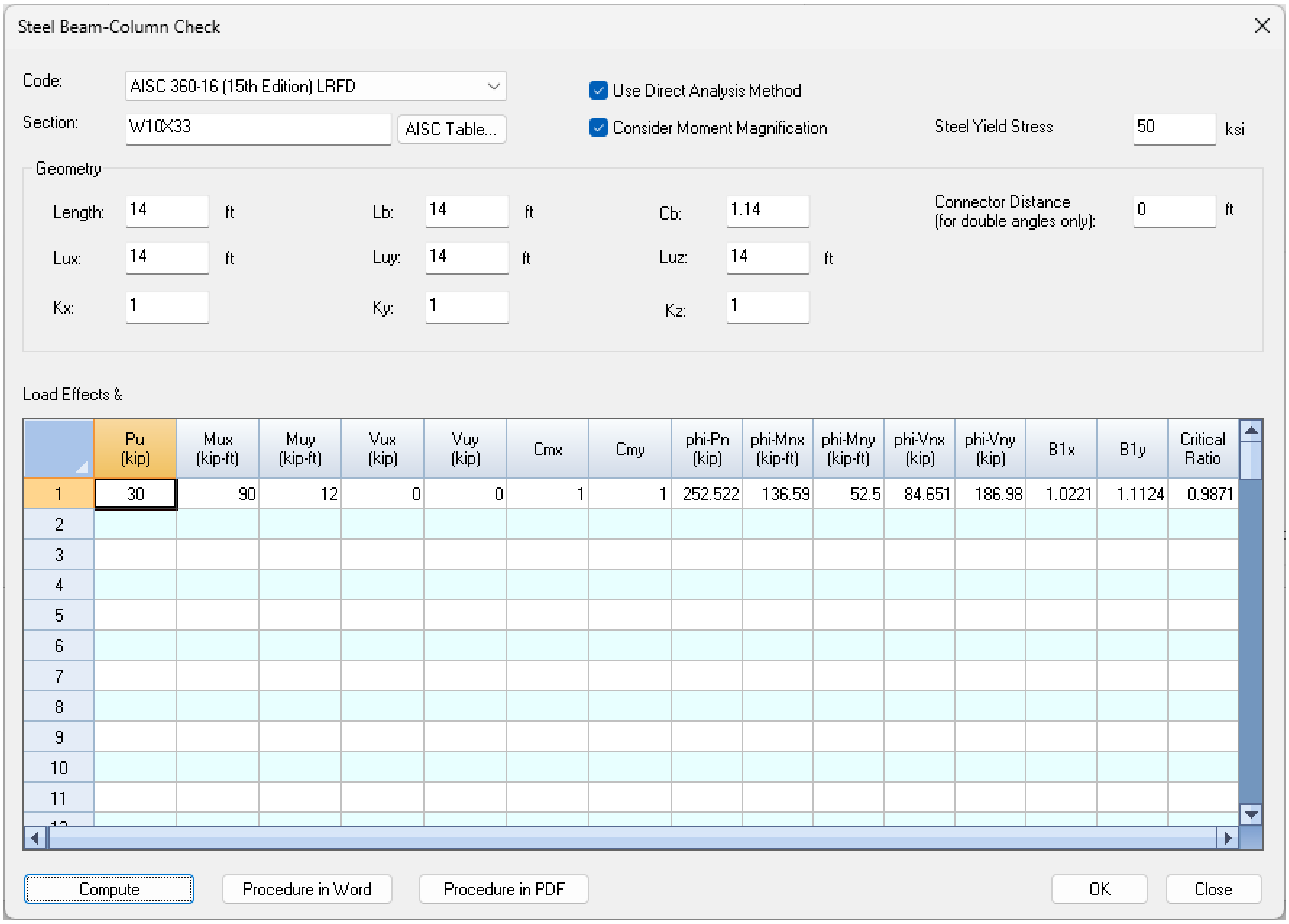


Step 2:

Enter the geometry, load condition and design criteria on the “Steel Beam-Column Design” dialog as following and click on “Perform Design”. By default, 10 section candidates will be chosen by the program. The sections are ordered in terms of section weight by linear foot.

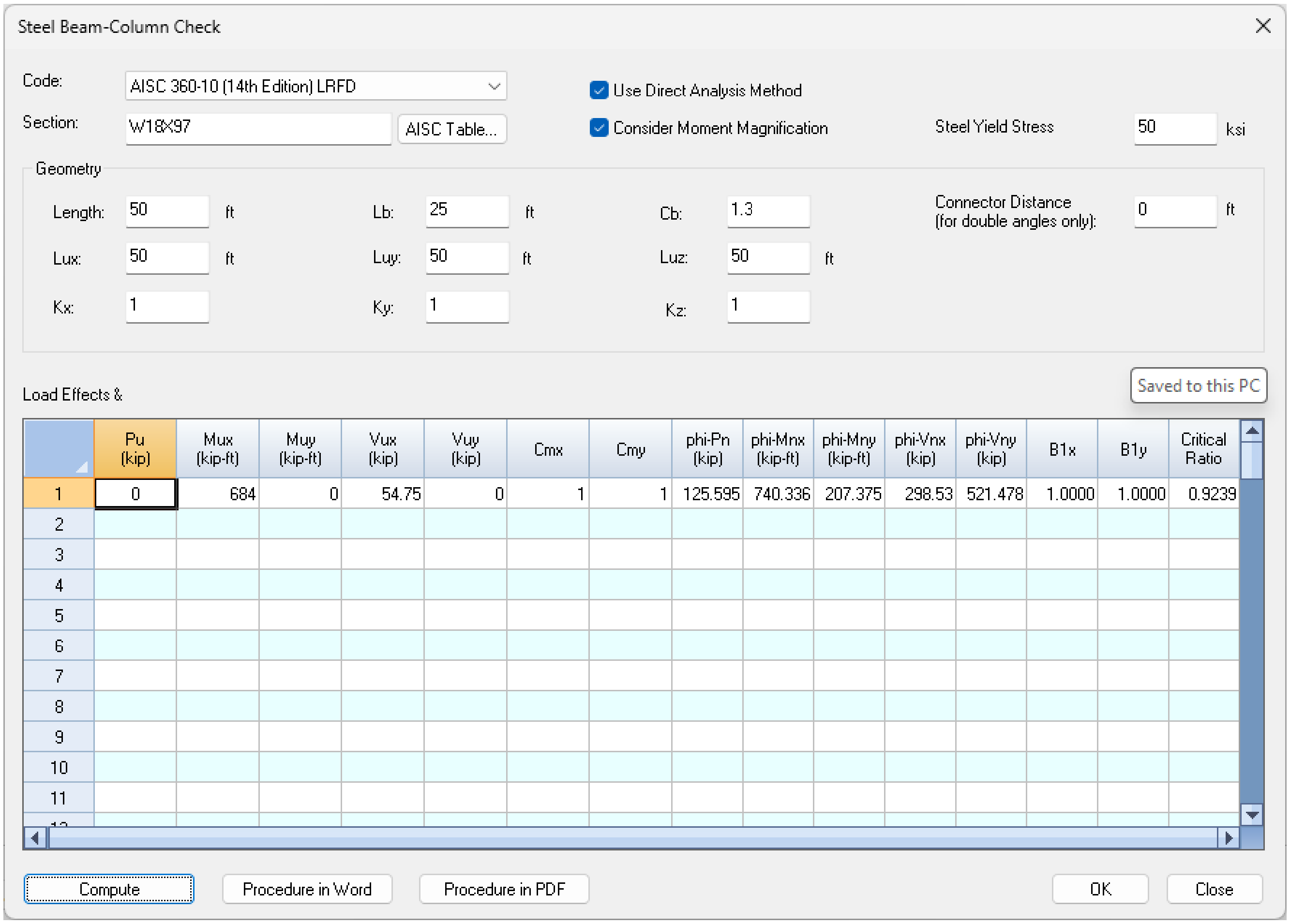


Step 3:
The first section candidate is W10x33, which is the one given by the reference. We can view the detailed check on a section candidate by first selecting the section candidate, then clicking on "Check Section" button on the "Steel Beam-Column Design" dialog above. The following is the "Steel Beam-Column Check" dialog. From here we are able to view very detailed calculation procedure by clicking on "Procedure in Word" or "Procedure in PDF".



Example 1 – W Shape Beam (Flexure in Major Axis, Lp < Lb < Lr)

[Ref 2, pp435]
Check the flexural capacity of W18x97 beam.
Fy = 50 ksi, L = 50 ft, Lb = 25 ft. Cb = 1.3.

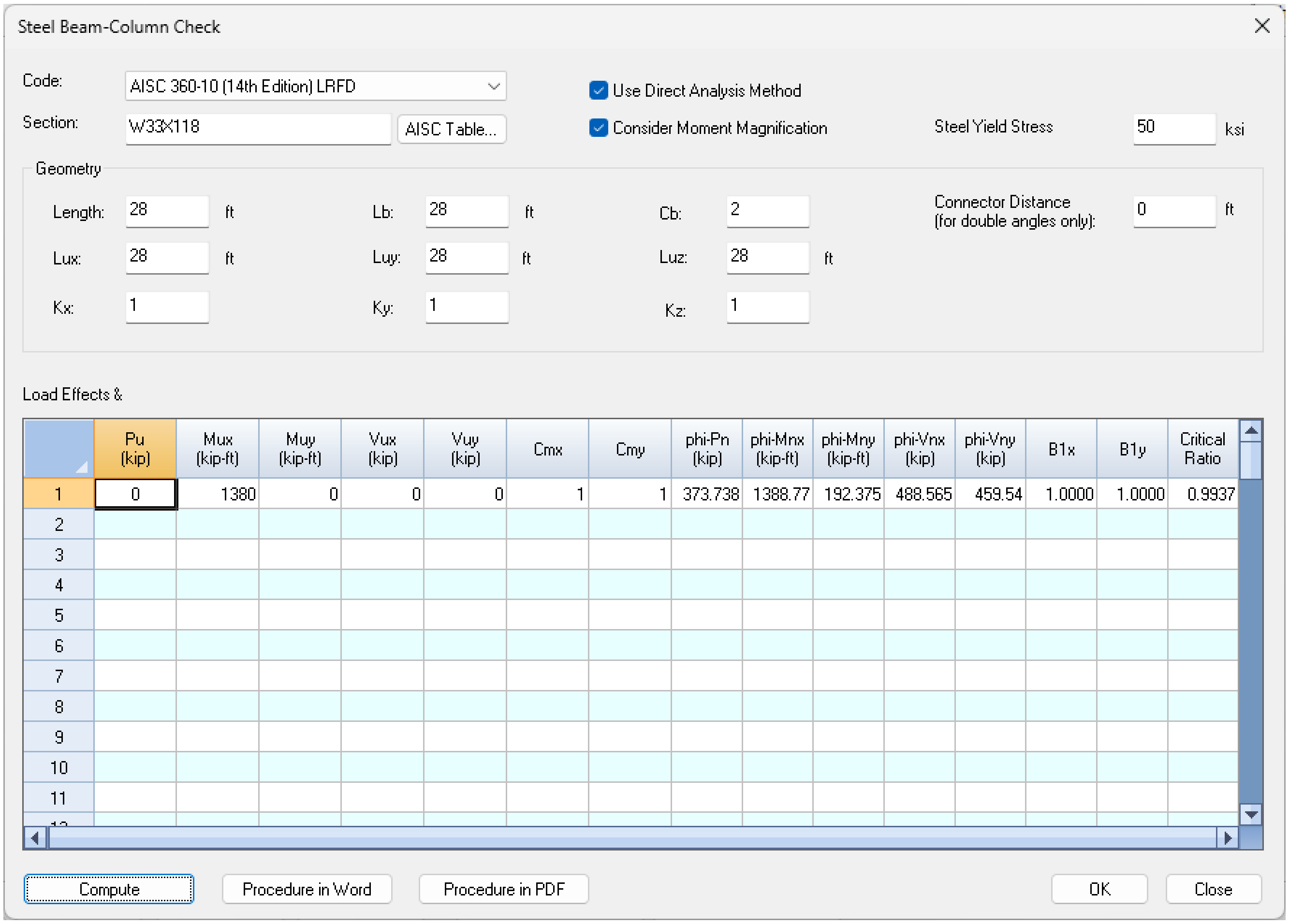


Result Comparison

The reference gives $_{\varphi}$Mnx = 740 ft-kips while sCheck gives $_{\varphi}$Mnx = 740.336 ft-kips. They are practically identical.

Example 2 – W Shape Beam (Flexure in Major Axis, Lb > Lr)

[Ref 2, pp440]
Check the flexural capacity of W33x118 beam.
Fy = 50 ksi, L = 28 ft, Lb = 28ft, Cb = 2.0.

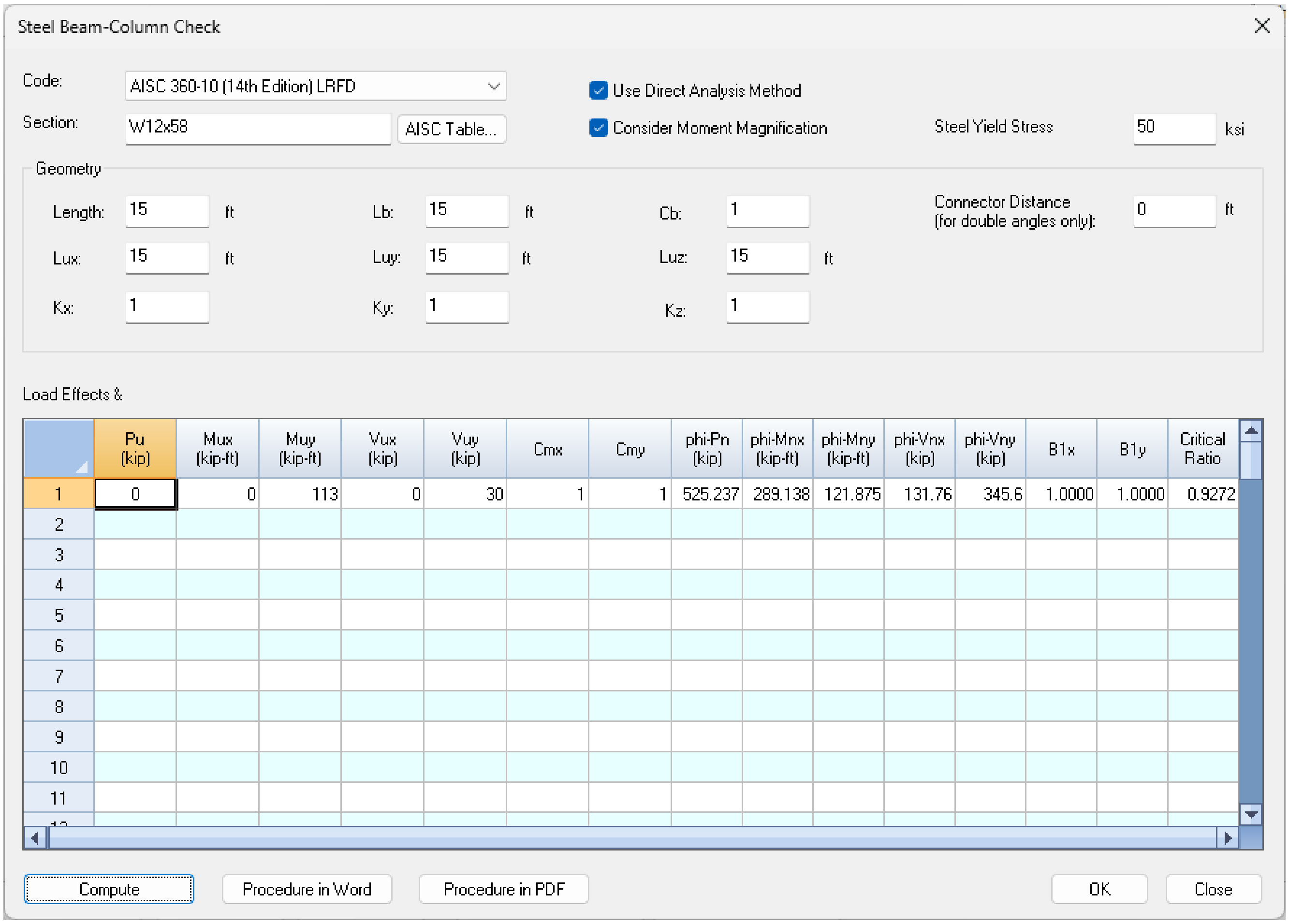


Result Comparison

The reference gives $_{\varphi}$Mnx = 1390 ft-kips while sCheck gives $_{\varphi}$Mnx = 1388.77 ft-kips. They are practically identical.

Example 3 – W Shape Beam (Flexure in Minor Axis)

[Ref 3, Example F.5]
Check the flexural capacity of W12x58 beam in minor axis.   Fy = 50 ksi, L = 15 ft.

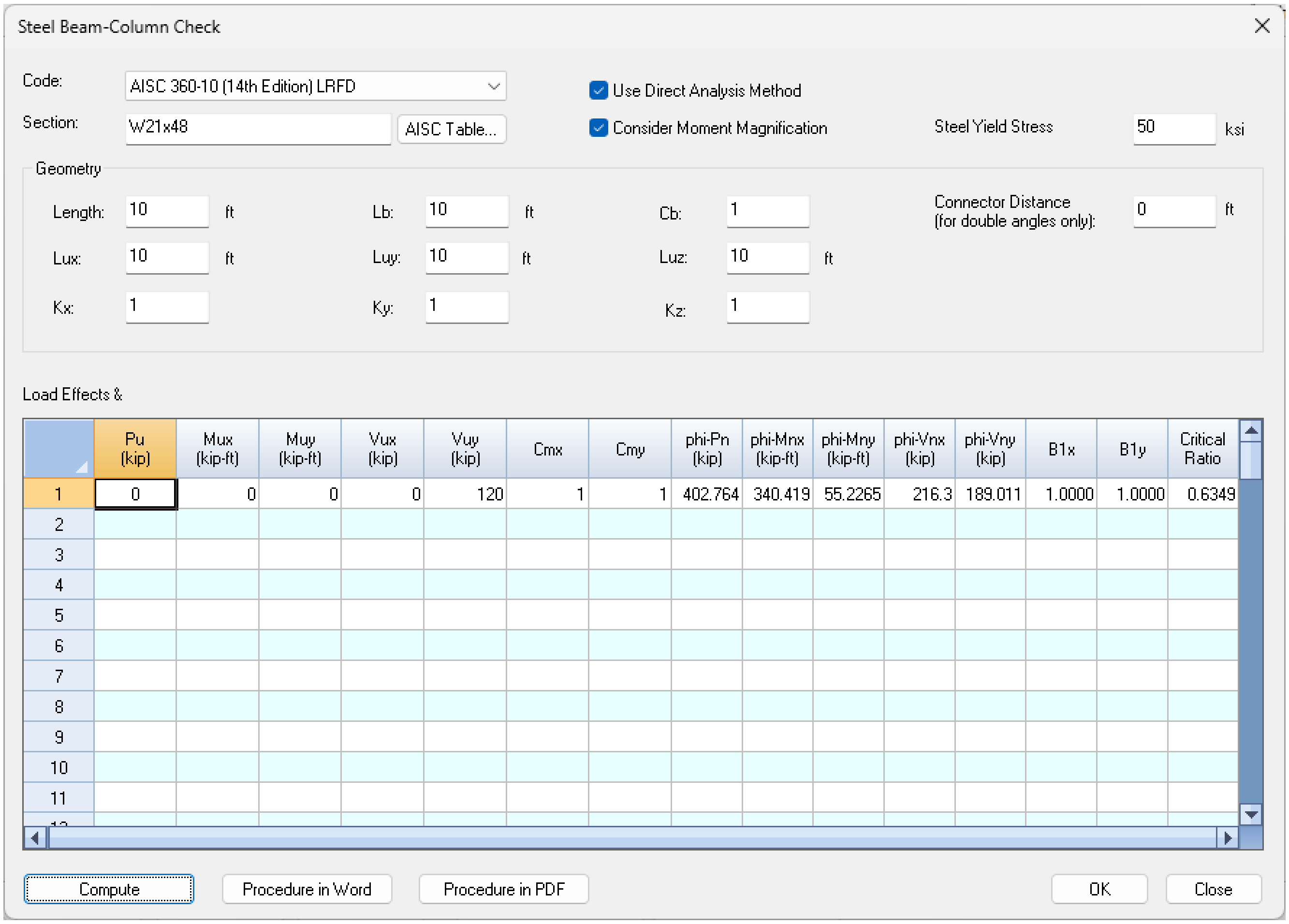


Result Comparison

The reference gives $_{\varphi}$Mny = 122 ft-kips while sCheck gives $_{\varphi}$Mny = 121.875 ft-kips.   They are practically identical.

Example 4 – W Shape Beam (Shear in Minor Axis)

[Ref 3, Example G.6]
Check the shear capacity of W21x48 beam in minor axis.
Fy = 50 ksi

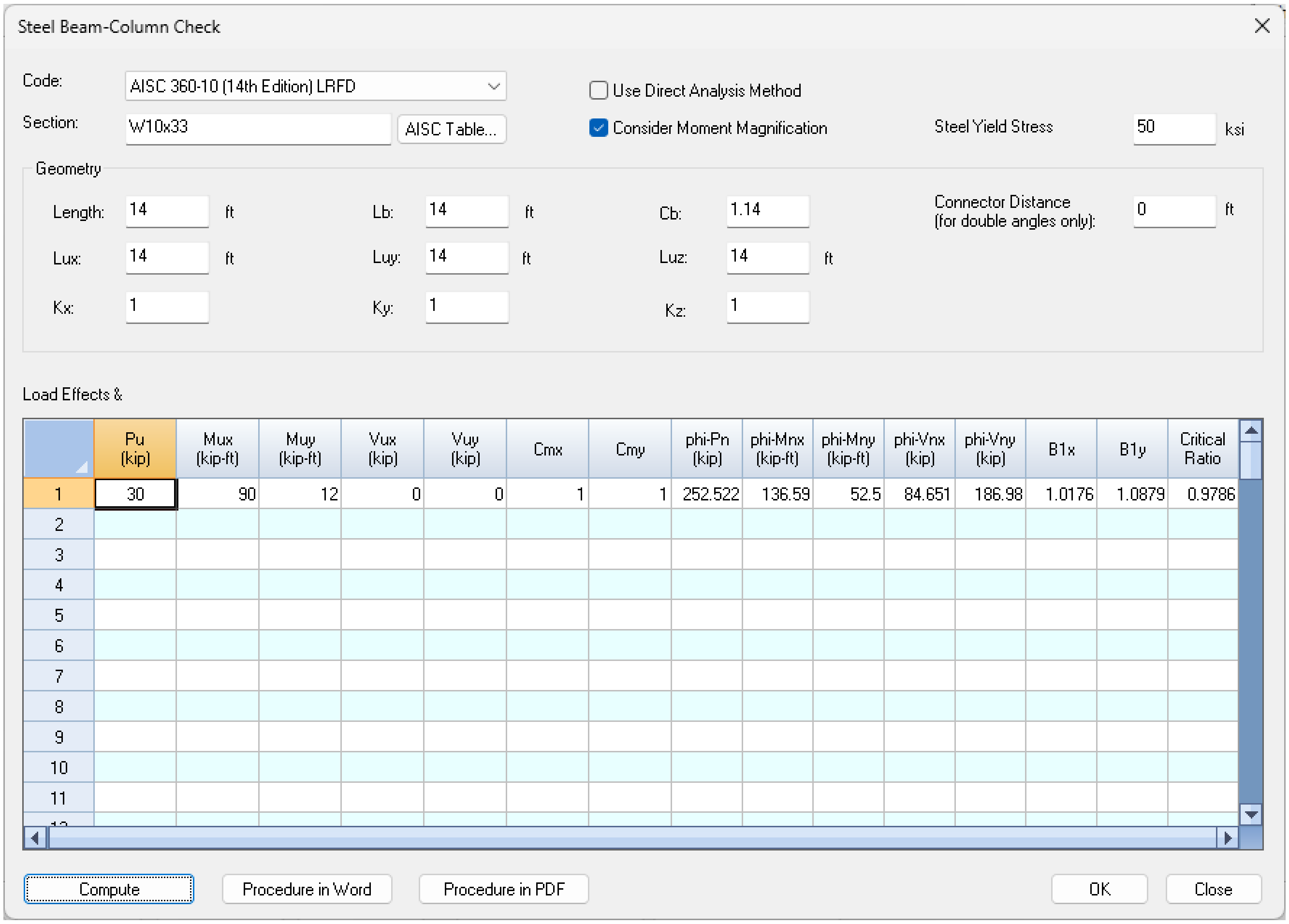


Result Comparison

The reference gives $_{\varphi}$Vny = 189 ft-kips while sCheck gives $_{\varphi}$Vny = 189.011 ft-kips. They are practically identical.

Example 5 – W Shape Column (Combined Axial Compression and Flexure)

[Ref 3, Example H.4]
Check the capacity of W10x33 column in combined axial compression and flexure.
Fy = 50 ksi, L = 14 ft, Kx = Ky = Kz = 1.0, Lux = Luy = Lb = 14 ft, Cb = 1.14.

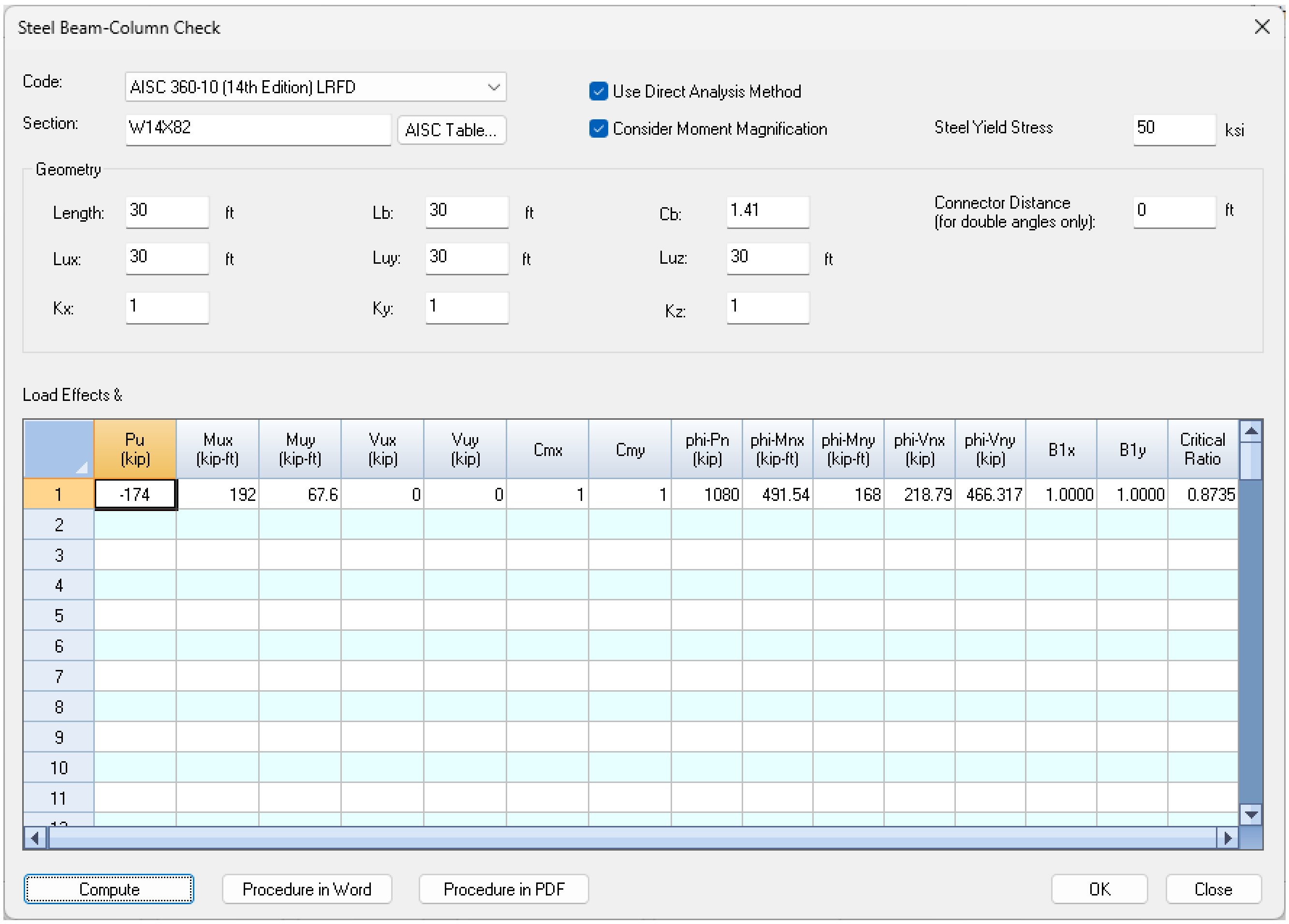


Result Comparison

The reference gives $_{\varphi}$Pn = 253 kips, $_{\varphi}$Mnx = 137 ft-kips, $_{\varphi}$Mny = 52.5 ft-kips while sCheck gives $_{\varphi}$Pn = 252.522 kips, $_{\varphi}$Mnx = 136.59 ft-kips, $_{\varphi}$Mny = 52.5 ft-kips. They are practically identical.

Example 6 – W Shape Column (Combined Axial Tension and Flexure)

[Ref 3, Example H.3]
Check the capacity of W14x82 column in combined axial tension and flexure.
Fy = 50 ksi, L = 30 ft, Kx = Ky = Kz = 1.0, Lux = Luy = Lb = 30 ft, Cb = 1.41.
Note: tension load must be entered as negative value.

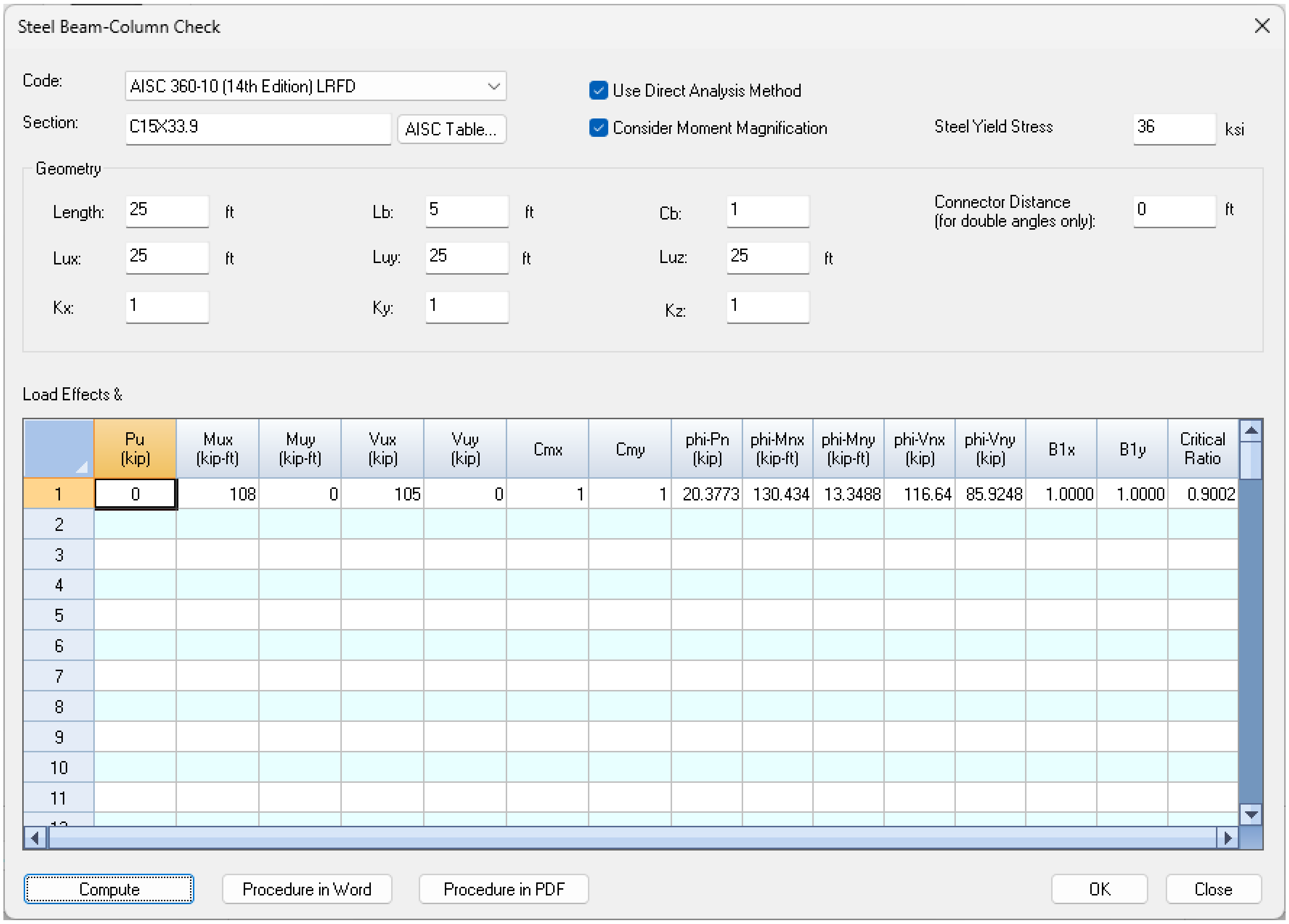


Result Comparison

The reference gives $_{\varphi}$Pn = 1080 kips, $_{\varphi}$Mnx = 492 ft-kips, $_{\varphi}$Mny = 168 ft-kips while sCheck gives $_{\varphi}$Pn = 1080 kips, $_{\varphi}$Mnx = 491.54 ft-kips, $_{\varphi}$Mny =168 ft-kips. They are practically identical. Please note that absolute value of axial load should be used for axial-flexural interaction formula.

Example 7 – C Shape Beam (Flexure and Shear)

[Ref 3, Example F.2-2A and G.2B]
Check the capacity of C15x33.9 column in flexure and shear.
Fy = 36 ksi, L = 25 ft, Lb = 5 ft, Cb = 1.0.

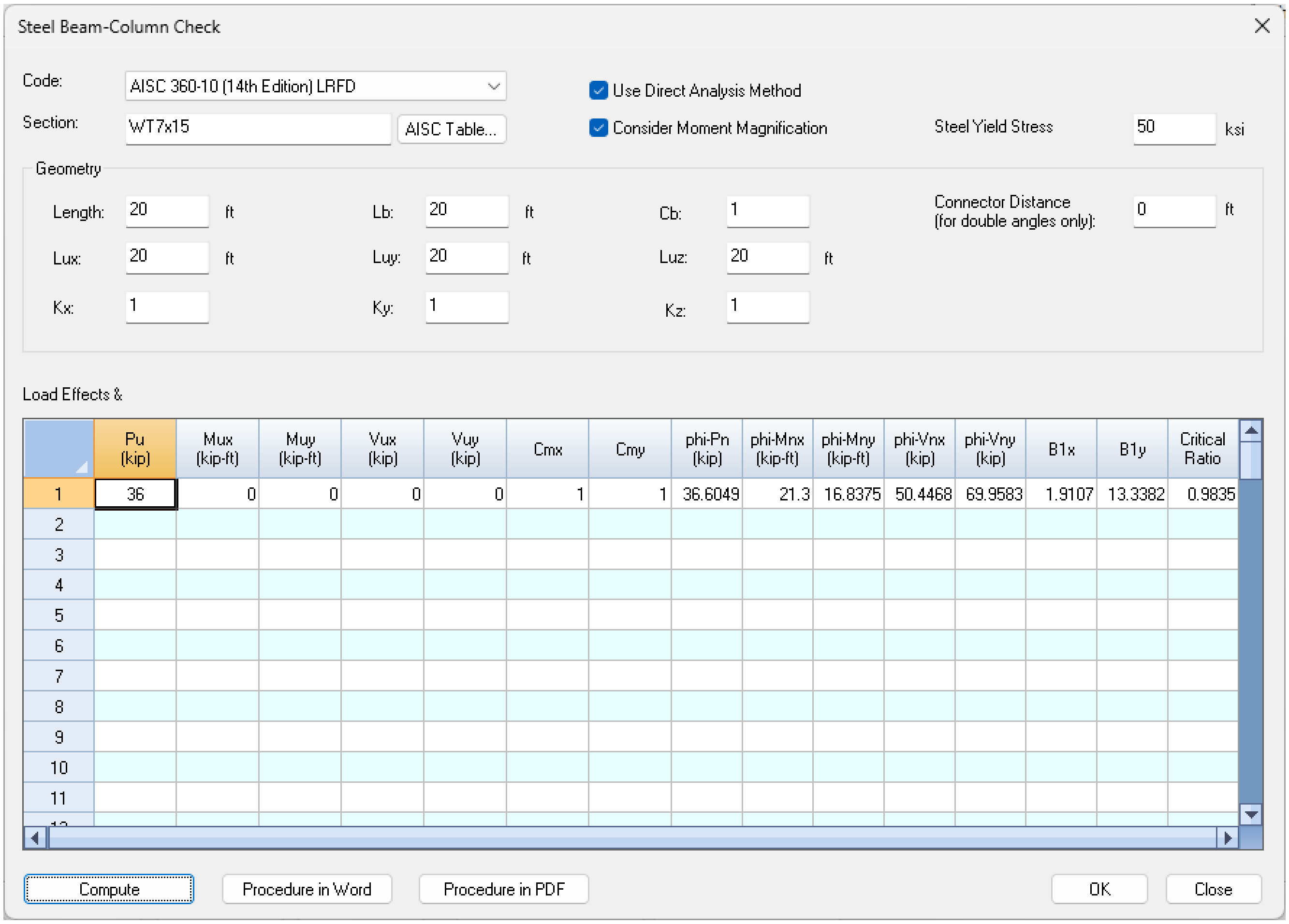


Result Comparison

The reference gives $_{\varphi}$Mnx = 130 ft-kips, $_{\varphi}$Vnx = 117 kips while sCheck gives $_{\varphi}$Mnx = 130.434 ft-kips, $_{\varphi}$Vnx = 116.64 kips. They are practically identical.

Example 8 – WT Shape Column (Axial Compression)

[Ref 3, Example E.8]
Check the capacity of WT7x15 column in axial compression.
Fy = 50 ksi, L = 20 ft, Kx = Ky = Kz = 1.0, Lu = 20 ft, Cb = 1.41.

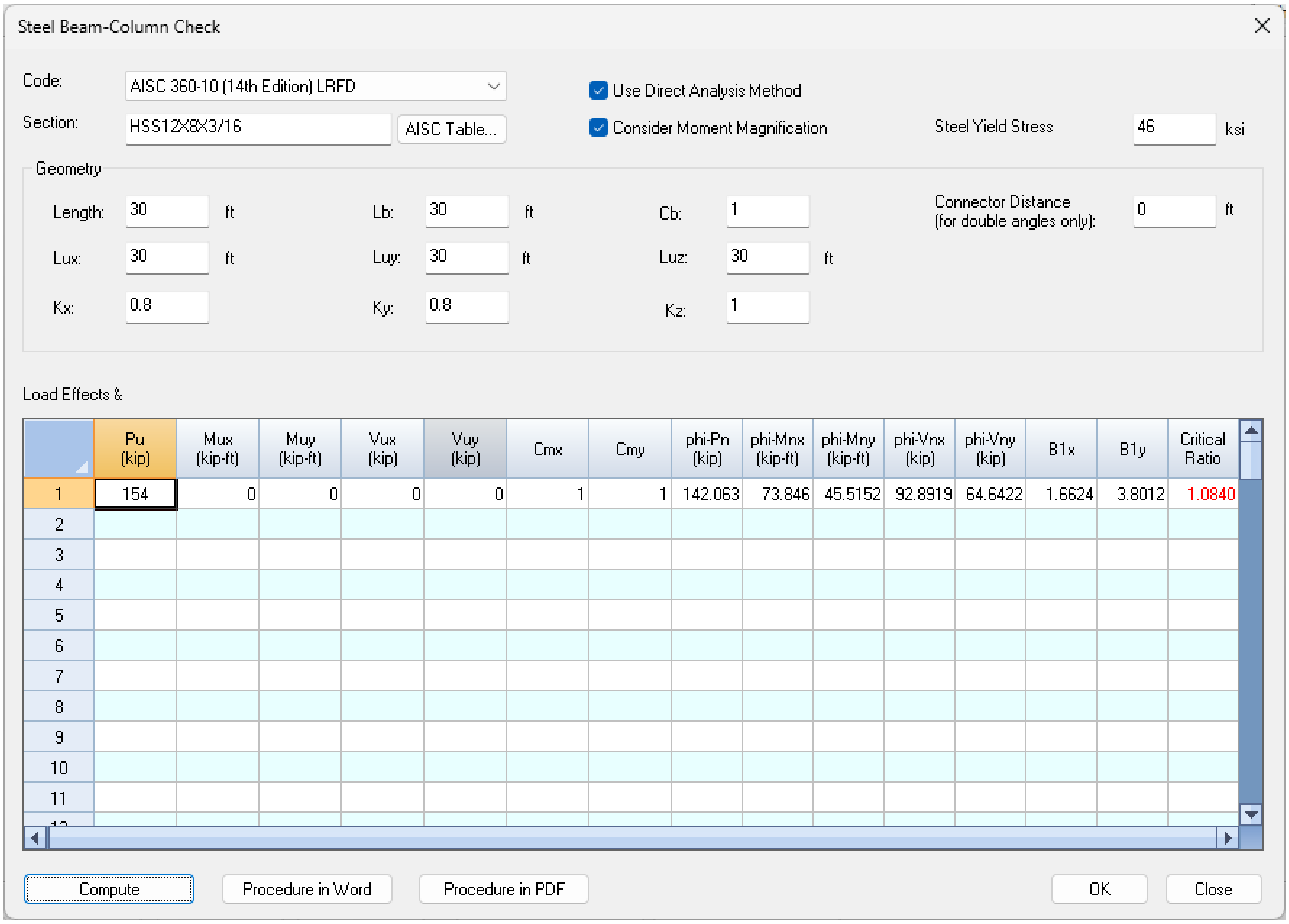


Result Comparison

The reference gives $_{\varphi}$Pn = 36.6 kips while sCheck gives $_{\varphi}$Pn = 36.6049 kips. They are practically identical.

Example 9 – HSS Shape Column (Axial Compression)

[Ref 3, Example E.10]
Check the capacity of HSS12x8x3/16 column in axial compression.
Fy = 46 ksi, L = 30 ft, Kx = Ky = 0.8, Kz = 1.0, Lu = 30 ft, Cb = 1.0.

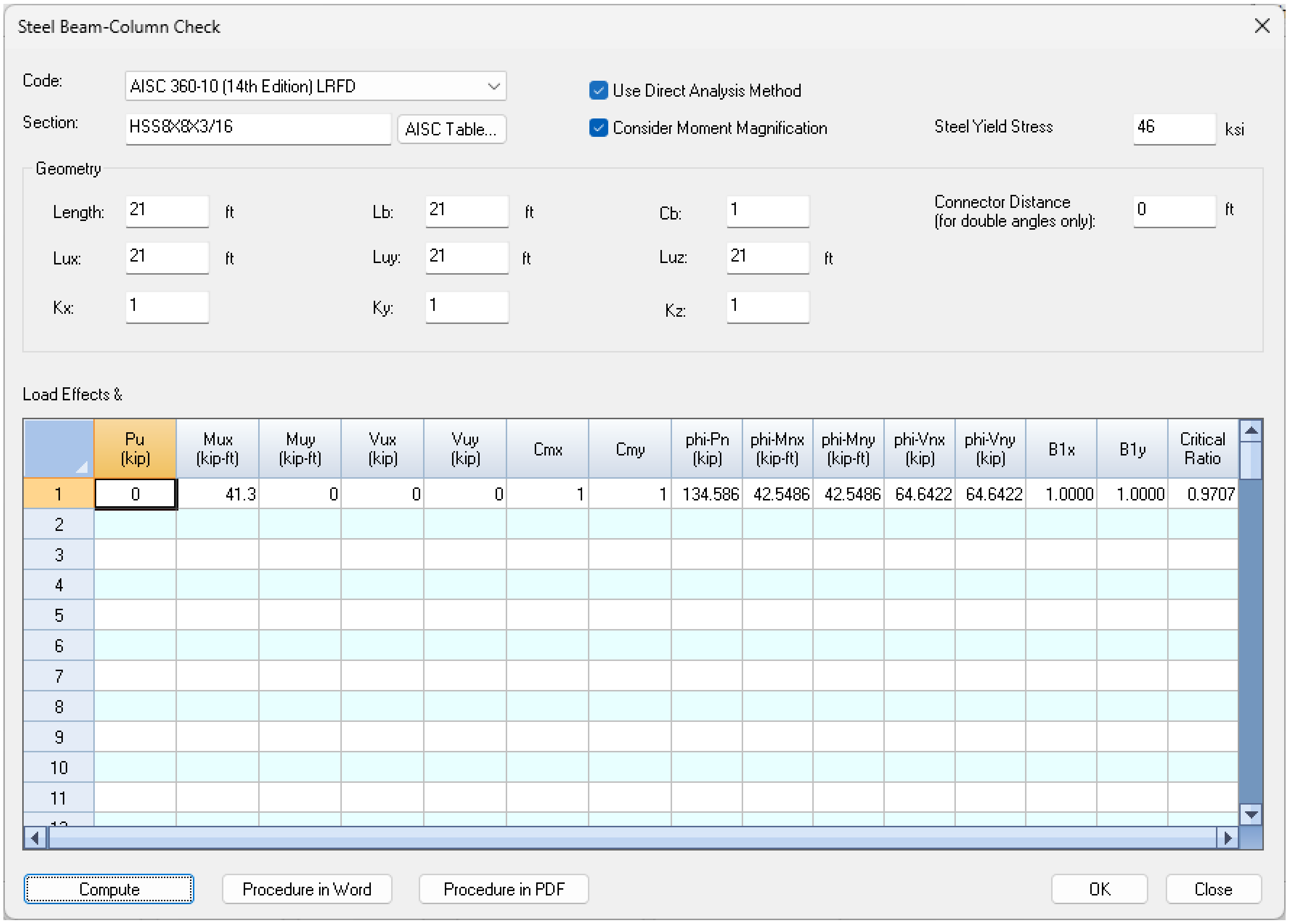


Result Comparison

The reference gives $_{\varphi}$Pn = 142 kips while sCheck gives $_{\varphi}$Pn = 142.063 kips. They are practically identical. Please be advised that sCheck takes a conservative initial assumption (f = Fy) in applying AISC specification Eq. E7-18.

Example 10 – Rectangular HSS Shape Beam (Flexure)

[Ref 3, Example F.8B]
Check the capacity of HSS8x8x3/16 column in flexure.
Fy = 46 ksi, L = 21 ft, Lb = 21 ft, Cb = 1.0.



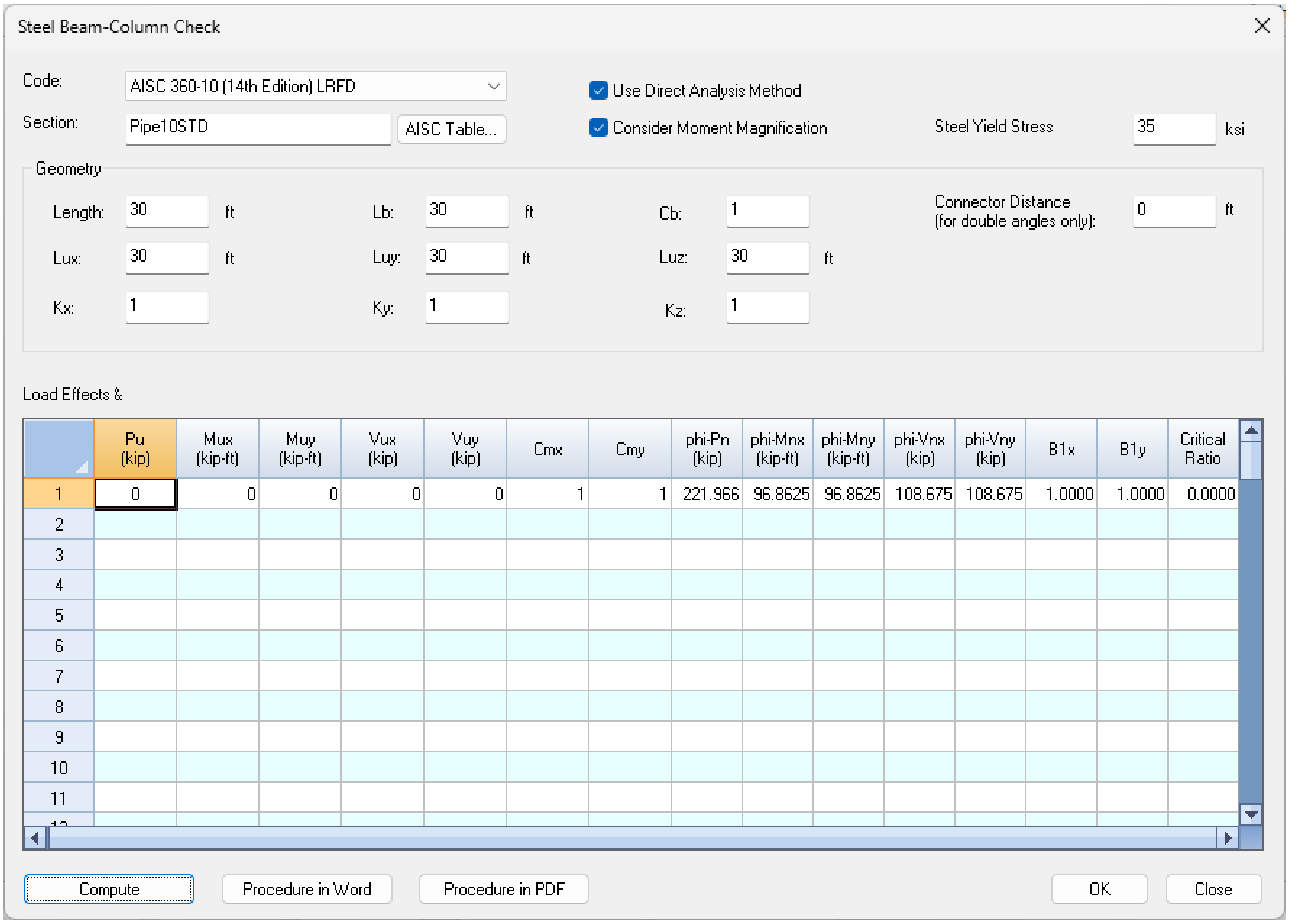
	Pu (kip)	Mux (kip-ft)	Muy (kip-ft)	Vux (kip)	Vuy (kip)	Cmx	Cmy	phi-Pn (kip)	phi-Mnx (kip-ft)	phi-Mny (kip-ft)	phi-Vnx (kip)	phi-Vny (kip)	B1x	B1y	Critical Ratio
1	0	41.3	0	0	0	1	1	134.586	42.5486	42.5486	64.6422	64.6422	1.0000	1.0000	0.9707
2															
3															
4															
5															
6															
7															
8															
9															
10															
11															

Result Comparison

The reference gives $_{\varphi}$Mnx = 42.5 ft-kips while sCheck gives $_{\varphi}$Mnx = 42.5486 ft-kips. They are practically identical.

Example 11 – Pipe Shape Column (Compression)

[Ref 3, Example E.11]
Check the capacity of Pipe10STD column in flexure.
Fy = 35 ksi, L = 30 ft

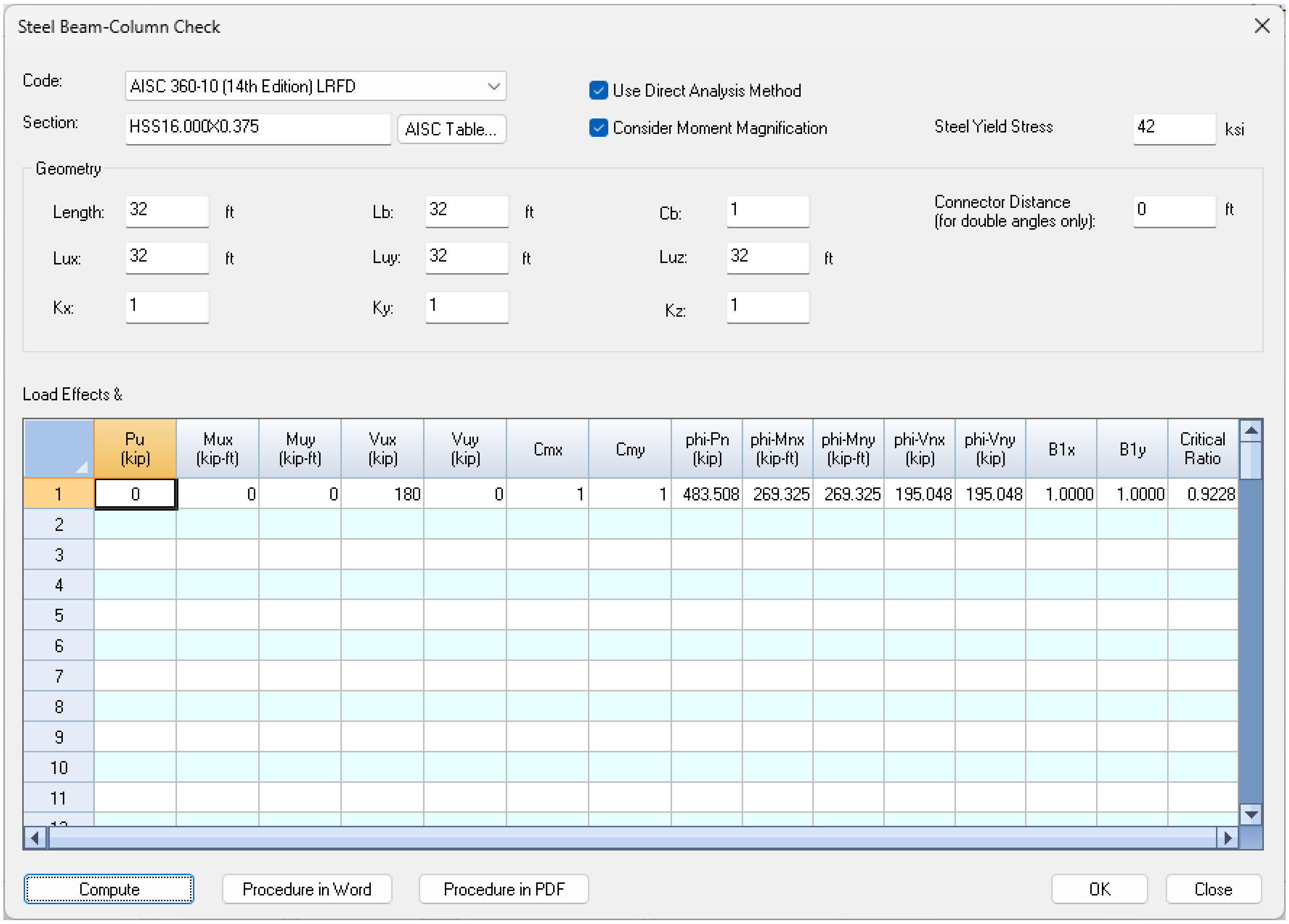


Result Comparison

The reference gives $_{\varphi}$Pn = 222 kips while sCheck gives $_{\varphi}$Pn = 221.966 kips. They are practically identical.

Example 12 – Round HSS Shape Beam (Shear)

[Ref 3, Example G.5]
Check the capacity of HSS16X0.375 column in shear.
Fy = 42 ksi, L = 32 ft

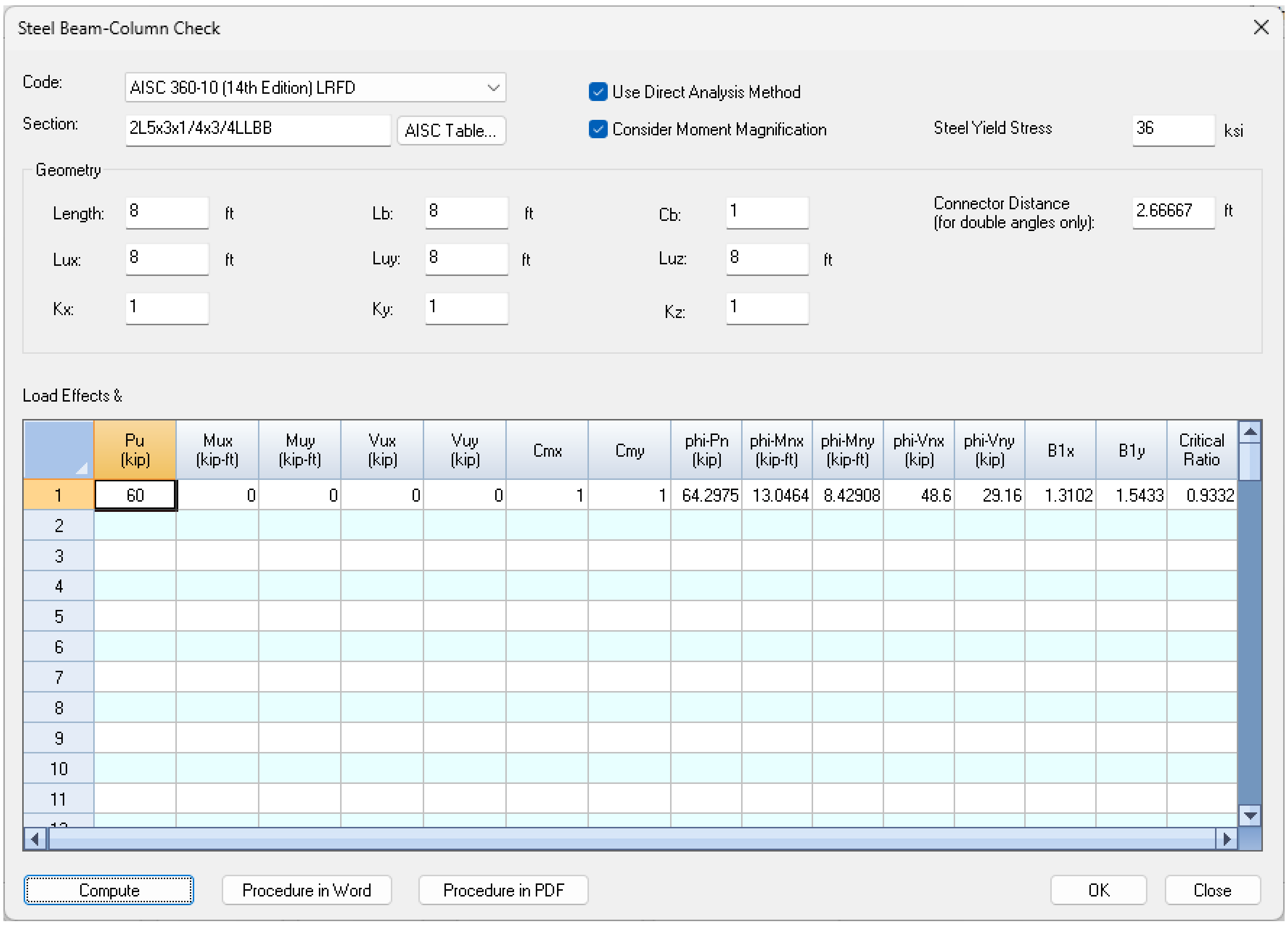


Result Comparison

The reference gives $_{\varphi}$Vnx = 195 kips while sCheck gives $_{\varphi}$Vnx = 195.048 kips. They are practically identical.

Example 13 – Double Angle Shape Beam (Axial Compression)

[Ref 3, Example E.6]
Check the capacity of 2L5x3x1/4x3/4LLBB column in axial compression.
Fy = 36 ksi, L = 8 ft, Kx = Ky = Kz = 1.0, Lux = Luy = Luz = 8 ft.
Connector distance = 32 in = 2.66667 ft.

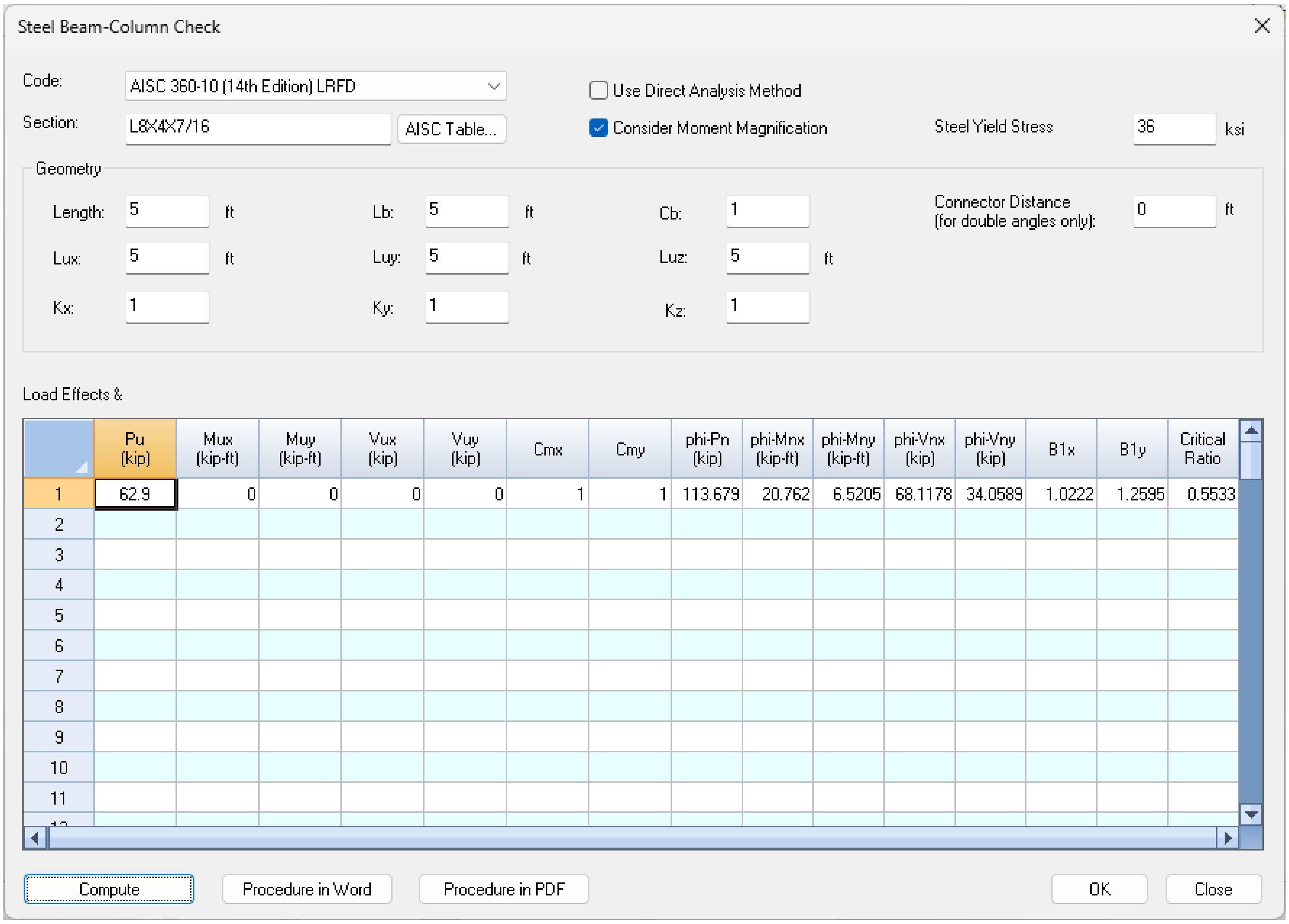


Result Comparison

The reference gives $_\varphi$Pn = 64.3 kips while sCheck gives $_\varphi$Pn = 64.2975 kips. They are practically identical.

Example 14 – Single Angle Shape Column (Axial Compression and Flexure)

[Ref 3, Example E.14]
Check the capacity of L8X4X7/16 column in axial compression and flexure.
Fy = 36 ksi, L = 5 ft, Kx = Ky = Kz = 1.0, Lux = Luy = Luz = 5 ft.



Result Comparison

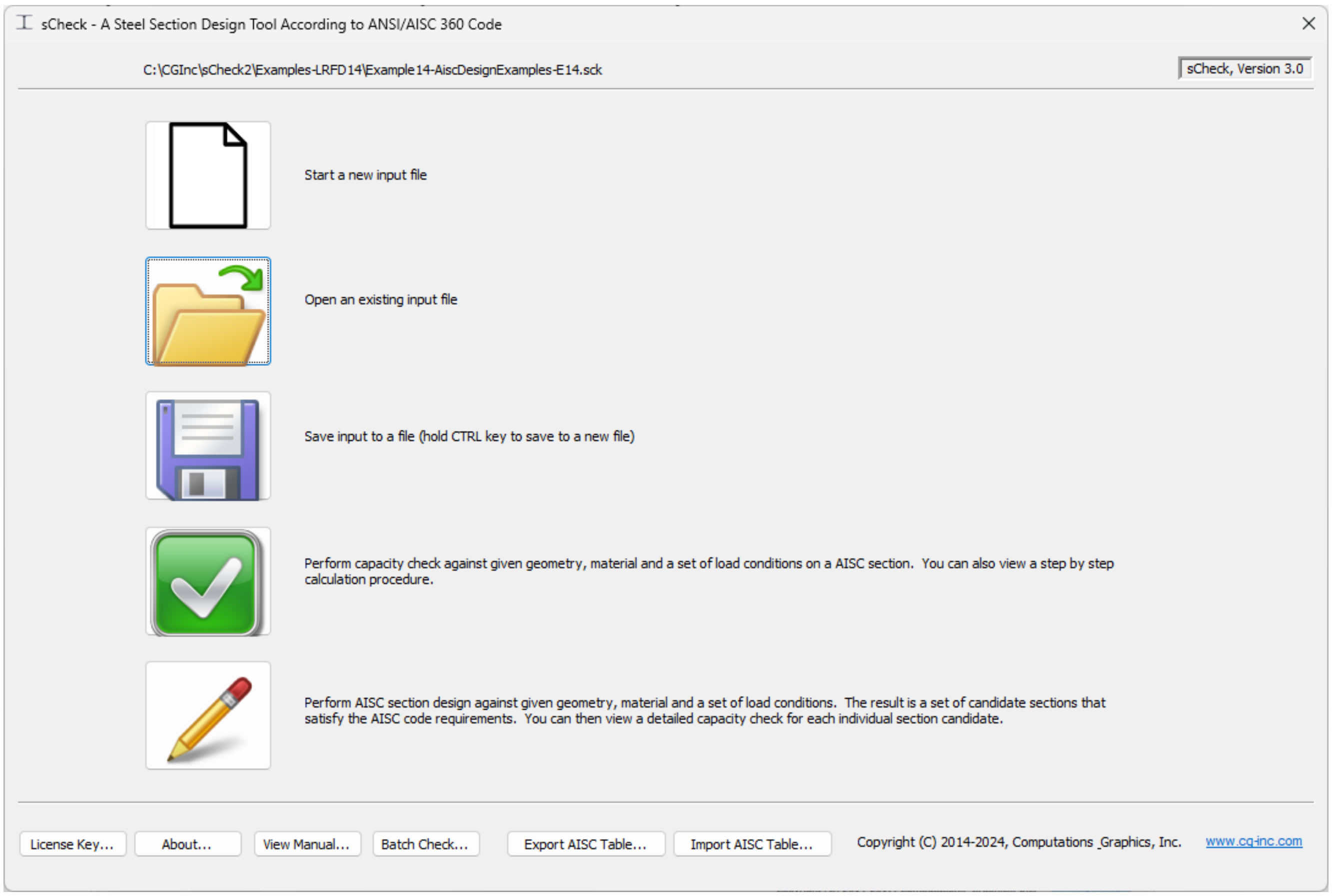
The reference gives $_{\varphi}Pn$ = 113 kips, $_{\varphi}Mnx$ ($_{\varphi}M_{nw}$) = 248 in-kips = 20.67 ft-kips, $_{\varphi}Mny$ ($_{\varphi}M_{nz}$) = 78.3 in-kips = 6.525 ft-kips while sCheck gives $_{\varphi}Pn$ = 113.679 kips, $_{\varphi}Mnx$ ($_{\varphi}M_{nw}$) = 20.762 ft-kips, $_{\varphi}Mny$ ($_{\varphi}M_{nz}$) = 6.5205 ft-kips. They are practically identical. Please note that for a single angle, the moment capacities are given about the principal w-w and z-z axes and the input moments Mux and Muy are transformed in the principal axes before flexural-axial interaction ratio is checked. In a conservative approach, the program always uses the absolute value for each term in Eq. H2-1, thus we have different critical ratio between sCheck and the reference.

Example 15 – W Shape Beam-Column Design – A Tutorial

[Ref 3, Example H.4]
Design a W8, W10 or W12 shape pinned beam-column that will resist the following conditions.
Fy = 50 ksi, L = 14 ft, Kx = Ky = Kz = 1.0, Lux = Luy = Lb = 14 ft, Cb = 1.14.
Pn = 30 kips, Mux = 90 ft-kips, Muy = 12 ft-kips, Cmx = 1.0, Cmy = 1.0.
The required moments do not include second-order effects.

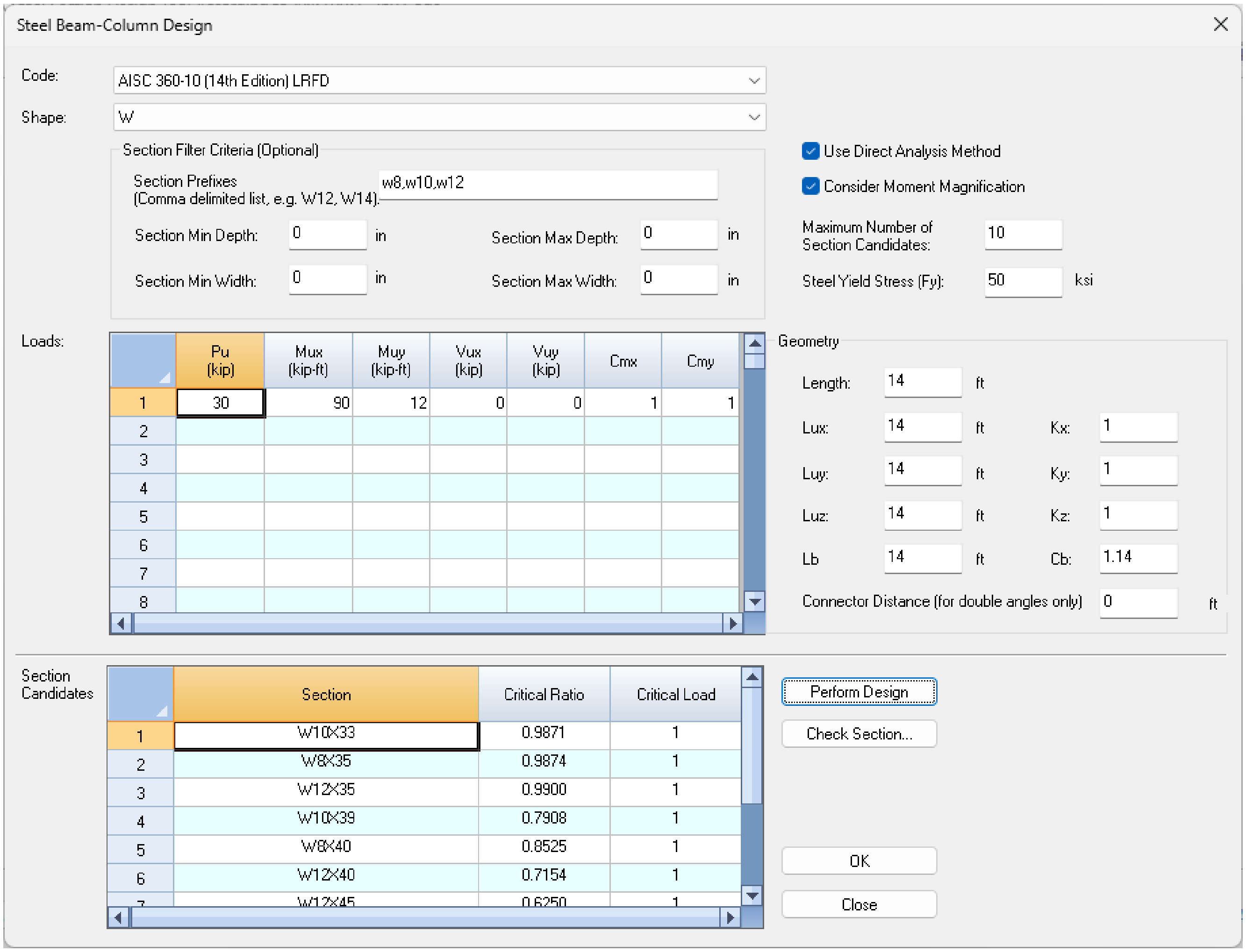
Step 1:

Click on the design button  from the sCheck main screen.



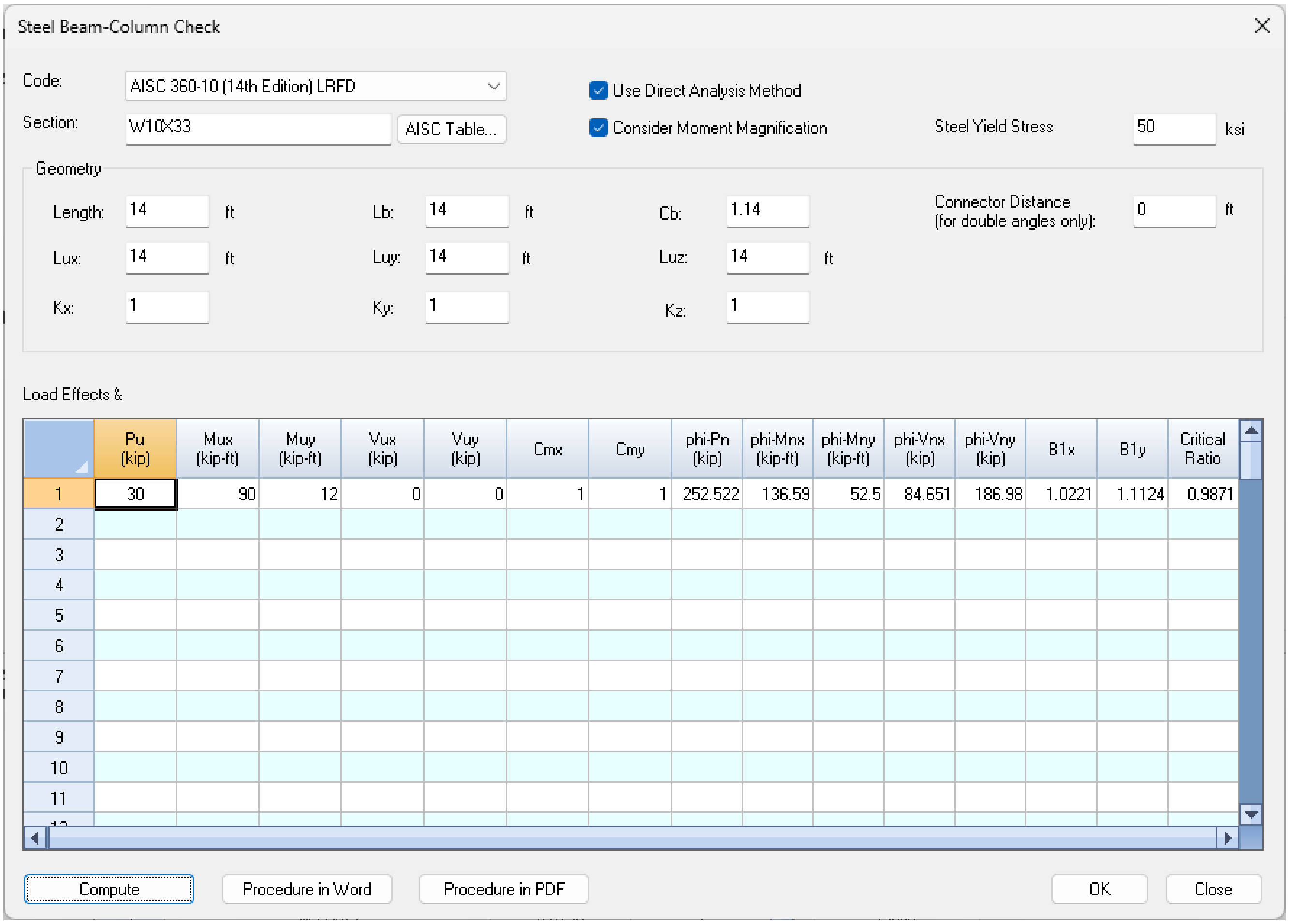
Step 2:

Enter the geometry, load condition and design criteria on the “Steel Beam-Column Design” dialog as following and click on “Perform Design”. By default, 10 section candidates will be chosen by the program. The sections are ordered in terms of section weight by linear foot.



Step 3:

The first section candidate is W10x33, which is the one given by the reference. We can view the detailed check on a section candidate by first selecting the section candidate, then clicking on “Check Section” button on the “Steel Beam-Column Design” dialog above. The following is the “Steel Beam-Column Check” dialog. From here we are able to view very detailed calculation procedure by clicking on “Procedure in Word” or “Procedure in PDF”.


Steel Beam-Column Check
Code: AISC 360-10 (14th Edition) LRFD
Section: W10X33
AISC Table...
Use Direct Analysis Method
Consider Moment Magnification
Steel Yield Stress 50 ksi
Geometry
Length: 14 ft
Lb: 14 ft
Cb: 1.14
Connector Distance (for double angles only): 0 ft
Lux: 14 ft
Luy: 14 ft
Luz: 14 ft
Kx: 1
Ky: 1
Kz: 1
Load Effects &
Pu (kip) | Mux (kip-ft) | Muy (kip-ft) | Vux (kip) | Vuy (kip) | Cmx | Cmy | phi-Pn (kip) | phi-Mnx (kip-ft) | phi-Mny (kip-ft) | phi-Vnx (kip) | phi-Vny (kip) | B1x | B1y | Critical Ratio
1 | 30 | 90 | 12 | 0 | 0 | 1 | 1 | 252.522 | 136.59 | 52.5 | 84.651 | 186.98 | 1.0221 | 1.1124 | 0.9871
2
3
4
5
6
7
8
9
10
11
Compute
Procedure in Word
Procedure in PDF
OK
Close

Chapter 8: Sample sCheck Detailed Report

The following lists an example of detailed calculation procedures in Word format that is automatically generated by sCheck.

General Info

File Name	C:\CGInc\sCheck3\Examples-LRFD16\Example15-AiscDesignExamples-H4.sck
Design Code	AISC 360-22 (16th edition) LRFD
Using Direct Analysis Method	Yes
Consider Multiplier B1 for P-delta Effect	Yes
Date & Time	11/26/2023 18:39

Section Property - W10X33

Property	Value	Unit	Property	Value	Unit	Property	Value	Unit
A = Ag	9.71	in^2	bf	7.96	in	tf	0.435	in
tw	0.29	in	d	9.73	in	h / tw	27.1	
Cw	791	in^6	h0	9.3	in	rts	2.2	in
Zx	38.8	in^3	Sx	35	in^3	Ix	171	in^4
rx	4.19	in	Zy	14	in^3	Sy	9.2	in^3
Iy	36.6	in^4	ry	1.94	in	J	0.583	in^4

Design Input

Input	Value	Unit	Input	Value	Unit	Input	Value	Unit
Pu = Pr	30	kips	Mux = Mxr	90	kip-ft	Muy = Myr	12	kip-ft
Cmx	1		Cmy	1		Vux	0	kips
Vuy	0	kips	Fy	50	ksi	Cb	1.14	
Lb	14	ft	Kx	1		Ky	1	
Kz	1		Lx	14	ft	Ly	14	ft
Lz	14	ft						

* Lcx = Kx * Lx; Lcy = Ky * Ly; Lcz = Kz * Lz

Axial Capacity Calculation

Step	Equation	Value	Note
Checking flange slenderness			
	b = bf / 2	3.98 in	
	b / tf	9.1494	
	$\lambda_r = 0.56\sqrt{\frac{E}{F_y}}$	13.487	
The section has non-slender flange element			
Checking web slenderness			
	b / t = h / tw	27.1	
	$\lambda_r = 1.49\sqrt{\frac{E}{F_y}}$	35.884	

The section has non-slender web			
Compressive strength to account for flexural buckling			
	$\frac{K_x L_x}{r_x}$	40.095	
	$\frac{K_y L_y}{r_y}$	86.598	
	$\frac{KL}{r} = \max\left(\frac{K_x L_x}{r_x}, \frac{K_y L_y}{r_y}\right)$	86.598	
	$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$	38.167 ksi	Eq.E3-4
	$4.71\sqrt{\frac{E}{F_y}}$	113.43	
	$\frac{KL}{r} \leq 4.71\sqrt{\frac{E}{F_y}}$		
	$F_n = \left(0.658^{\frac{F_y}{F_e}}\right) F_y$	28.896 ksi	Eq.E3-2
	$P_n = F_n A_g$	280.58 kips	Eq.E3-1
Compressive strength to account for torsional and flexural-torsional buckling			
	$F_e = \left(\frac{\pi^2 E C_w}{L_{cz}^2} + GJ\right) \frac{1}{I_x + I_y}$	70.092 ksi	Eq.E4-2
	$\frac{F_y}{F_e}$	0.71335	
	$\frac{F_y}{F_e} \leq 2.25$		
	$F_n = \left(0.658^{\frac{F_y}{F_e}}\right) F_y$	37.094 ksi	Eq.E3-2
	$P_n = F_n A_g$	360.18 kips	Eq.E4-1
	Flexural buckling controls: Pn	280.58 kips	
	$\phi_c P_n$	252.52 kips	

Moment Magnification Calculation

Step	Equation	Value	Note
Stiffness reduction parameter			

	$\alpha = 1.00(\text{LRFD})$	1	
	Pr / Pns = Pr / (Fy * Ae)	0.061792	
	$\alpha P_r / P_{ns} \leq 0.5$		
	$\tau_b = 1.0$	1	Eq.C2-2a
Moment magnifier B1 for P-delta effects in local x direction			
	$EI^* = 0.8\tau_b EI$	3.9672e+06 ksi	
	$P_{e1} = \frac{\pi^2 EI^*}{(K_1 L)^2}$	1387.3 kips	Eq.A-8-5
	$B_1 = \frac{C_m}{1 - \alpha P_r / P_{e1}} \geq 1$	1.0221	Eq.A-8-3
	Magnified Mux = Mux * B1	91.989 kip-ft	
Moment magnifier B1 for P-delta effects in local y direction			
	$EI^* = 0.8\tau_b EI$	8.4912e+05 ksi	
	$P_{e1} = \frac{\pi^2 EI^*}{(K_1 L)^2}$	296.93 kips	Eq.A-8-5
	$B_1 = \frac{C_m}{1 - \alpha P_r / P_{e1}} \geq 1$	1.1124	Eq.A-8-3
	Magnified Muy = Muy * B1	13.349 kip-ft	
Mrx = Mux; Mry = Muy			

Major Flexure Capacity Calculation

Step	Equation	Value	Note
Web compactness:			
	$\lambda = \frac{h_c}{t_w}$	27.1	
	$\lambda_{pw} = 3.76\sqrt{\frac{E}{F_y}}$	90.553	
	$\lambda_{rw} = 5.70\sqrt{\frac{E}{F_y}}$	137.27	
Web is compact			
Flange compactness:			

	$\lambda = \frac{b_f}{2t_f}$	9.1494	
	$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_y}}$	9.1516	
	$\lambda_{rf} = 1.0\sqrt{\frac{E}{F_y}}$	24.083	
Flange is compact			
Mnx to account for yielding			
	$M_n = M_p = F_y Z_x$	161.67 kip-ft	Eq.F2-1
Mnx to account for flange local buckling			
	$\lambda < \lambda_{pf}$		
	$M_n = M_p$	161.67 kip-ft	
Mnx to account for lateral-torsional buckling			
	$L_p = 1.76 r_y \sqrt{\frac{E}{F_y}}$	6.8525 ft	Eq.F2-5
	For I section, c	1	
	$L_r = 1.95 r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{S_x h_o} + \sqrt{\left(\frac{Jc}{S_x h_o}\right)^2 + 6.76\left(\frac{0.7F_y}{E}\right)^2}}$	21.776 ft	Eq.F2-6
	$M_n = M_p = F_y Z_x$	161.67 kip-ft	Eq.F2-1
Since Lp < Lb < Lr			
	$M_n = C_b \left[M_p - (M_p - 0.7F_y S_x)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p$	151.77 kip-ft	Eq.F2-2
	Controlling nominal flexural strength Mnx	151.77 kip-ft	
	$M_{cx} = \phi_b M_{nx}$	136.59 kip-ft	

Minor Flexure Capacity Calculation

Step	Equation	Value	Note
Mny to account for yielding			
	Fy * Zy	58.333 kip-ft	

	Fy * Sy	38.333 kip-ft	
	$M_n = M_p = F_y Z_y \leq 1.6 F_y S_y$	58.333 kip-ft	Eq.F6-1
Mny to account for lateral-torsional buckling			
	$\lambda < \lambda_{pf}$		
	$M_n = M_p$	58.333 kip-ft	
	Controlling nominal flexural strength Mny	58.333 kip-ft	
	$M_{cy} = \phi_b M_{ny}$	52.5 kip-ft	

Flexural and Axial Interaction Calculation

Step	Equation	Value	Note
	$\frac{P_r}{P_c} = \frac{P_u}{\phi_c P_n}$	0.1188	
	$\frac{P_r}{P_c} < 0.2$		
	$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0$	0.98713	Eq.H1-1b
Axial-flexural strength: OK			

Major Shear Capacity Calculation

Step	Equation	Value	Note
	$A_w = d t_w$	2.8217 in^2	
Computing Cv for major axis using G2.1			
	$k_v = 5.34$		
	h/t_w	27.1	
	$2.24\sqrt{E/F_y}$	53.946	
	$h/t_w \leq 2.24\sqrt{E/F_y}$		
	$C_{v1} = 1.0$		Eq.G2-2
Major shear strength			
	$V_n = 0.6 F_y A_w C_{v1}$	84.651 kips	Eq.G2-1

	$h/t_w \leq 2.24\sqrt{E/F_y}$		
	$\phi_v = 1.00$		
	$\phi_v V_n$	84.651 kips	
	$\frac{V_u}{\phi_v V_n}$	0	
Shear strength (major axis): OK			

Minor Shear Capacity Calculation

Step	Equation	Value	Note
	$A_w = 2b_f t_f$	6.9252 in^2	
Computing Cv2 for weak axis using G2.2			
	$k_v = 1.2$		
	$h/t_w = b/t_f$	9.1494	
	$1.10\sqrt{k_v E/F_y}$	29.02	
	$1.37\sqrt{k_v E/F_y}$	36.143	
	$h/t_w \leq 1.10\sqrt{k_v E/F_y}$		
	$C_{v2} = 1.0$	1	Eq.G2-9
Minor shear strength			
	$V_n = 0.6F_y b_f t_f C_{v2}$	207.76 kips	Eq.G6-1
	$\phi_v = 0.90$		
	$\phi_v V_n$	186.98 kips	
	$\frac{V_u}{\phi_v V_n}$	0	
Shear strength (minor axis): OK			

References

1. American Institute of Steel Construction, “Steel Construction Manual” 14th Edition
2. Charles G. Salmon, John E. Johnson, Faris A. Malhas, “Steel Structures – Design and Behavior”, 5th edition., Pearson Education, Inc., 2009
3. American Institute of Steel Construction, “Design Examples” Version 14.1
4. American Institute of Steel Construction, “Design Examples” Version 15.1
5. American Institute of Steel Construction, AISC 360-16, “Specification for Structural Steel Building” , July, 2016
6. American Institute of Steel Construction, “Design Examples” Version 16.0
7. American Institute of Steel Construction, AISC 360-22, “Specification for Structural Steel Building” , August, 2022