RISAFloor

Rapid Interactive Structural Analysis – Floor Systems

Verification Problems

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Verification Overview

Verification Methods

We at RISA maintain a library of dozens of test problems used to validate the computational aspects of RISA programs. In this verification package we will present a representative sample of these test problems for your review and compare RISAFloor to textbook or AISC 16th edition manual design examples.

The input for these test problems was formulated to test RISAFloor's performance, not necessarily to show how certain structures should be modeled and in some cases the input and assumptions we use in the test problems may not match what a design engineer would do in a "real world" application. The RISAFloor solutions for each problem are compared to an example in the AISC 16th edition, unless otherwise noted. If discrepancies occur between the RISAFloor and the referenced solution results we will give an explanation.

The data for each of these verification problems is provided. The files where these RISAFloor problems are located are in the **...\RISA User Data\%USERNAME%\RISA\Model Files\Examples** directory and they are called **Verification Problem 1.rfl** (…2, 3, etc). The PDF document is located in the **...\Program Files\RISA\Manuals** directory and is called **Floor Verification Problems.pdf**

Verification Version

This document contains problems that have been verified in RISAFloor version 19.0.

Verification Problem 1: Composite ASD #1

Fully Composite Beam in ASD

This problem represents a typical fully composite beam. This model tests the composite beam design properties, deflections, and load attribution. The hand verification of this problem can be taken directly from the ***AISC ASD 9th edition manual*** (Example 15 on Page 2-249). A model was built per the description given in the Manual. The beams were entered as generic wide flanges, but the program was allowed to choose the most efficient beam to meet deflection requirements.

Description / Problem Statement

A model was built per the description given in the example and solved using the AISC 9th Ed. ASD. Dead load deflection was limited to 1.5 inches and live load deflection was limited to L/360.

Span Length	=	36 ft
Beam spacing	=	8 ft
Slab thickness	=	4 in
Concrete	=	145 pcf
		3.0 ksi (n is assumed to equal 9)
Steel Yield	=	36 ksi
Live Load	=	100 psf
Partition Load	=	20 psf
Ceiling Load	=	8 psf
Beam Weight	=	7 psf (assumed)

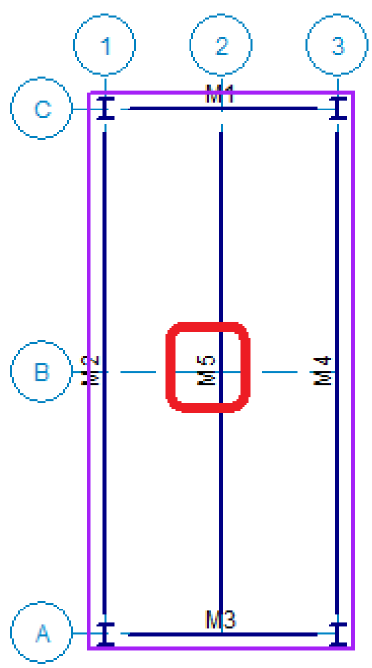


Figure 1.1

Validation Method

The model was built in RISAFloor exactly as it was described in the example. Results for member M5, shown above in Figure 1.1, are compared to the results in the design example.

The composite beam properties, beam stresses and deflections are compared with the results given in the ASD manual, with these modifications.

Notes:

1. In order to force the n value (Es/Ec) to be equal to the value assumed in the example (9.0), the Ec was set to 3222.22 ksi.
2. The actual beam self-weight was set to zero by making the density zero so that results would match the example's assumptions.
3. Partition and ceiling load was lumped in with live load for total live load.

Comparison

Results Comparison (Units Specified Individually)			
Value	RISAFloor Solution (AISC 9th Ed. ASD)	ASD Manual Hand Calculation	% Difference
M_{DL}	71.71 k-ft	71.30 k-ft	0.58
M_{LL}	165.89 k-ft	166.00 k-ft	0.07
V_{max}	26.4 kips	26.4 kips	0.00
b_{eff}	48 in + 48 in = 96 in	96 in	0.00
I_{tr}	2411.05 in^4	2420.00 in^4	0.37
S_{tr}	121.96 in^3	122.00 in^3	0.04
y_{bar}[1]	20.7 in + 4 in – 4.93 in = 19.77 in	19.80 in	0.15
f_c	0.443 ksi	0.440 ksi	0.68
Δ_{DL}	0.684 in	0.680 in	0.63
Δ_{LL}	0.553 in	0.550 in	0.54
f_{by}	26.89 ksi	26.80 ksi	0.33
V_h	234 kips	234 kips	0.00
# of Studs	42	21*2 = 42	0.00

Table 1.1 – Results Comparison

[1]$y_{bar} = d_{beam} + t_{slab} - Y\ Top$

Conclusion

The program results match the ASD manual within a reasonable round off error.

Verification Problem 2: Composite LRFD #1

Fully Composite Beam ASD

This problem represents a typical fully composite beam (LRFD). The hand verification of this problem can be taken directly from the fifth edition of Salmon and Johnson's, Steel Structures: Design and Behavior (Example 16.11.1 on Page 829).

Description / Problem Statement

A model was built per the description given in the example and solved using the AISC 16th Ed. LRFD.

Span Length	=	28 ft
Beam spacing	=	8 ft
Slab thickness	=	4 in
Concrete	=	145 pcf (0.15 pcf used in textbook, see notes on next page)
		4 ksi (n is assumed to equal 9)
Steel Yield	=	50 ksi
Live Load	=	200 psf
Beam Weight	=	40 plf (assumed)

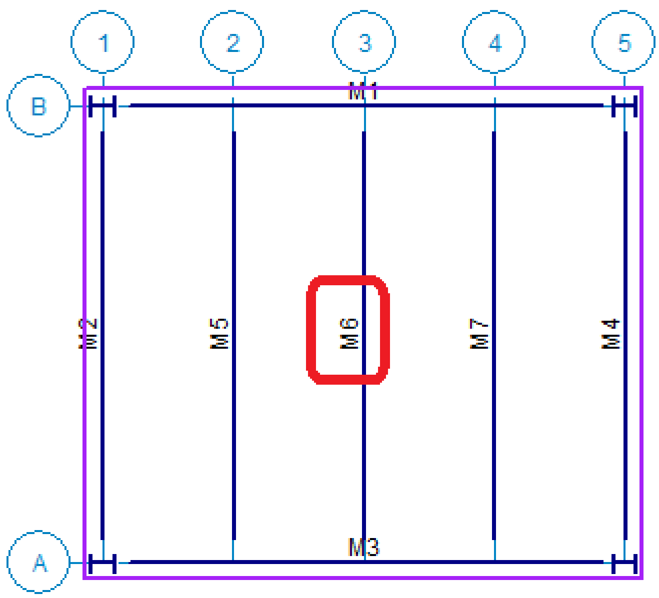


Figure 2.1

Validation Method

The model was built in RISAFloor exactly as it was described in the example, with the one exception of the concrete density: RISAFloor assumes a normal weight concrete self-weight of 145 pcf while the textbook rounds this up to 150 pcf. Results for Member M6, shown above in Figure 2.1, are compared to the results in the design example.

Notes:

1. To force the n value (Es/Ec) to be equal to the assumed 8.0, the Ec was set to 3625 ksi.
2. To compare results an allowance for beam weight of 40 plf was added as a line load. The actual beam self-weight was set to zero so that results would match the example's assumptions.

Comparison

Results Comparison (Units Specified Individually)			
Value	RISAFloor Solution (AISC 16th Ed. LRFD)	Textbook Hand Calculation	% Difference
M_{DL}*	41.8 k-ft	43.0 k-ft	2.76
M_{LL}	156.8 k-ft	157.0 k-ft	0.13
b_{eff}	42 in + 42 in = 84 in	84 in	0.00
M_u	301.1 kip-ft	301.0 kip-ft	0.03
T	384 kips	384 kips	0.00
C_f	384 kips	384 kips	0.00
a	1.345 in	1.34 in	0.37
M_n	357.7 kip-ft	357.7 kip-ft	0.00
Q_n**	21.5 kip/stud	26.1 kip/stud	n/a**
# of Studs	38	30	n/a**

Table 2.1 – Results Comparison

*RISAFloor defaults to a self-weight of 0.145 kcf where the example uses 0.150 kcf.

**The Salmon & Johnson example is based on the AISC 13th Edition steel manual which assumes a R_g=R_p=1.0 when calculating the available capacity of the studs. This has since changed in newer versions of the AISC Design Manuals. This effects the stud capacity and then the number of studs.

Conclusion

The program results match the textbook example within a reasonable round off error. The only significant difference is attributed to a calculation difference in the textbook and the self-weight of the deck.

Verification Problem 3: Composite LRFD #2

Fully Composite Beam LRFD

This problem represents a typical fully composite beam (LRFD). The hand verification of this problem can be taken directly from the 5th edition (2013) of William Segui's textbook, Steel Design (Example 9.1 and 9.2 Pages 597-603).

Description / Problem Statement

A W16x36 beam supports a 5 inch concrete slab (no deck) with an effective width of 87 inches. The concrete slab has an f'c = 4 ksi, Use an "n" value of 8.

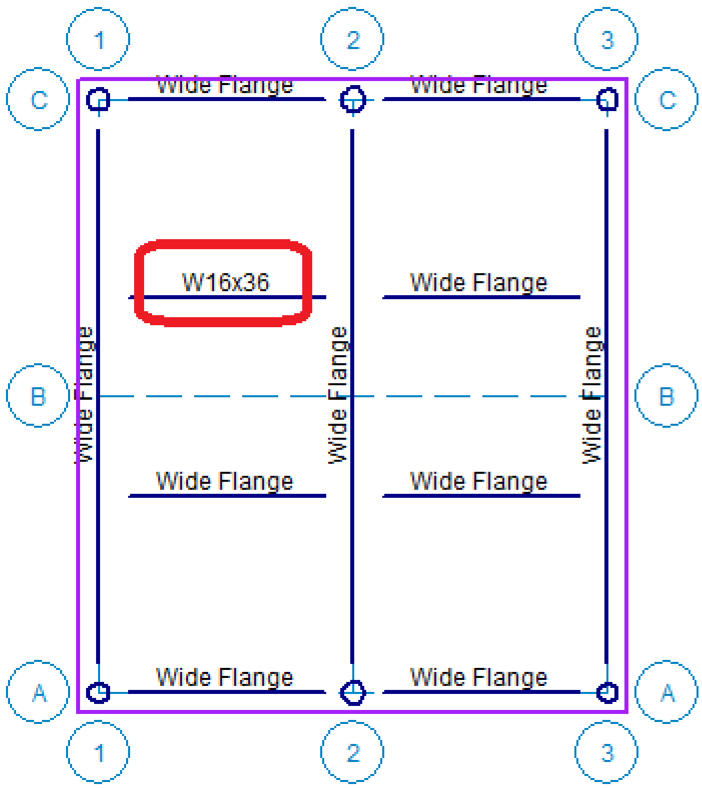


Figure 3.1

Validation Method

The model was built in RISAFloor exactly as it was described in the AISC example. Results for member PUR9, shown above in Figure 3.1, are compared to the results in the design example.

To force the n value (Es/Ec) to be equal to the assumed 8.0, the Ec was set to 3625 ksi.

Comparison

Comparison of Results (Units Specified Individually)			
Item	RISAFloor	Text	% Difference
Y_top (in)	4.205	4.205 / 4.143[1]	0.00
I_{tr} (in^4)	1530.0	1530 / 1528[1]	0.00
C (steel) (kips)	530	530	0.00
a (in)	1.792	1.792	0.00
M_n (kip-ft)	532.4	532.2	0.04

Table 3.1 – Results Comparison

[1]This example demonstrates an important limitation of the program. RISAFloor assumes the concrete slab will be above the neutral axis. When the neutral axis is within the slab, the program does NOT adjust the section properties to neglect the portion of the concrete that is in tension.

Conclusion

Even given the program limitations, the results match the textbook example within a reasonable round off error. The only significant difference is attributed to the small portion of the concrete slab which is located below the neutral axis.

Verification Problem 4: Composite LRFD #3

Fully Composite Beam LRFD

This problem represents a typical fully composite beam. The hand verification of this problem can be taken directly from the 5th edition (2013) of William Segui's textbook, Steel Design (Example 9.6, Pages 616-620).

Description / Problem Statement

A W16x26 joist supports a 3.5 inch concrete slab over a 1.5 inch metal deck. Beam spacing is 10ft, span is 30 feet. Concrete f'c = 4 ksi, with 20psf post-composite dead load, 20 psf construction live load, and 100 psf live load.

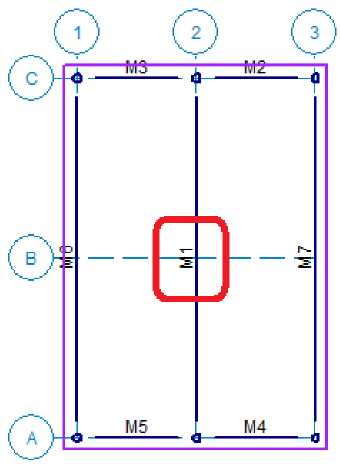


Figure 4.1

Validation Method

The model was built in RISAFloor exactly as it was described in the example. Results for member M1, shown above in Figure 4.1, are compared to the results in the design example.

The composite beam properties, beam stresses and deflections are compared with the results given in the text.

Comparison

Comparison of Results (Units Specified Individually)			
Item	RISAFloor	Text	% Difference
M_u (DLPre+LLConst) (kip-ft)	96.6	98.6	2.03[1]
M_u (DL+LL) (kip-ft)	267.6	279.0	0.86[1]
C_f (kips)	384	384	0.00
a (in)	1.255	1.255	0.00[2]
$\phi*M_n$ (kip-ft)[2]	309	309	0.00[2]
Q_n (kips)	9.57	9.57	0.00

Table 4.1 – Results Comparison

[1] RISAFloor defaults to a self-weight of 0.145 kcf where the example uses 0.150 kcf.
[2] To find this value in RISAFloor, solution must be forced to be 100% composite.

Conclusion

The program results match the textbook example within a reasonable round off error.

Verification Problem 5: Composite LRFD #4

Composite Beam LRFD

This problem represents a composite beam (LRFD). The hand verification of this problem can be taken directly from the 2nd edition of the LRFD Manual of Steel Construction (Example 5-1, Pages 5-12 to 5-14).

Description / Problem Statement

Steel Yield	=	50 ksi
Span	=	30 ft
Beam spacing	=	10 ft.
Slab Thickness	=	3.25 supported by a 3 inch metal deck perpendicular to beam
Concrete	=	115 pcf
		3.5 ksi
Live Load	=	1.3 k/ft (service)
Dead Load	=	0.9 k/ft (service)

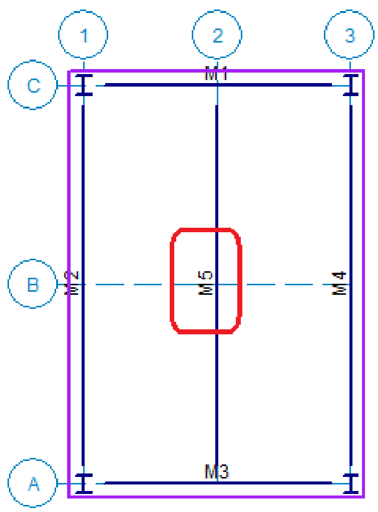


Figure 5.1

Validation Method

The composite beam properties, beam stresses and deflections for member M5 (shown in Figure 5.1 above) are compared with the results given in the text.

Comparison

Comparison of Results (Units Specified Individually)			
Item	RISAFloor	Text	% Difference
M_u (1.2DL +1.6LL) (kip-ft)	356	360	1.11
M_s (LL) (kip-ft)	146	146	0.00
ϕ *M_n (kip-ft)	364	363	0.28[1]
$a_{req'd}$ (in)	1.434	1.43	0.28[1]
ϕ *V_n (kips)	104	106	1.92
Q_n (kips)	19.9	19.8	0.51
# of studs required	40	40	0.00
LL Ratio	L/538	L/434	19.33[2]

Table 5.1 – Results Comparison

[1]To find this value in RISAFloor, solution must be forced to be 100% composite.

[2]The difference in live load deflection ratio is due to the text books use of the lower bound moment of inertia tables. RISAFloor uses the I_{tr} explicitly.

Conclusion

The program results match the textbook example within a reasonable round off error.

Verification Problem 6: Floor Vibrations

AISC Design Guide 11(1st Edition) – Floor Vibration

This example was pulled from the ***AISC Design Guide 11 – Floor Vibrations due to Human Activity*** Example 4.4 (pages 24-26).

A model was built per the description given in the textbook.

Description

Span Length	=	35 ft
Beam spacing	=	10 ft
Slab thickness	=	3.25 in + 2 inch ribs
Concrete	=	110 pcf
		4 ksi (n is assumed to equal 9)
Beam	=	W18x35
Girder	=	W21x50
Slab + Deck	=	41 psf
Live Load	=	11 psf
DL Equip	=	4 psf
Beam Weight	=	40 plf (assumed)

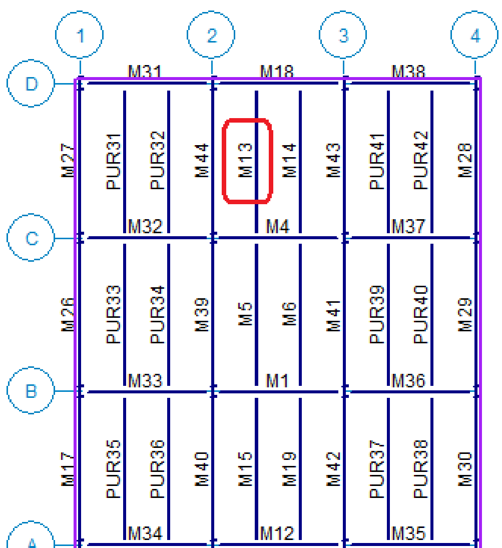


Figure 6.1

Validation Method

The model was built in RISAFloor exactly as it was described in the AISC design guide. Results for member M13, shown above in Figure 6.1, are compared to the results in the design example.

Comparison of Beam

Comparison of Results (Units Specified Individually)			
Value	RISAFloor	DG-11[1]	% Difference
B (in)	120	120	0.00
I_j (in^4)	1832.5	1833	0.03
w_j (plf)	603	605	0.33
Δ_j (in)	0.383	0.384	0.26
f_j (Hz)	5.71	5.71	0.00
D_s (in^4/ft)	8.22	8.25	0.36
D_j (in^4/ft)	183.3	183	0.16
B_j (ft)	32.2	32.2	0.00
W_j (kip)	102	102	0.00
I_g (in^4)	3273.5	3285	0.35
w_g (plf)	2161	2168	0.32
Δ_g (in)	0.415	0.415	0.00
f_g(Hz)	5.49	5.49	0.00
B_g (ft)	63.9	63.8	0.16
W_g (kip)	118.3	119	0.59
W (kip)	110	111	0.90
f_n(Hz)	4.03	4.03	0.00
a_p/g	0.48%	0.48%	0.00

Table 6.1 – Results Comparison

Conclusion

The program results match the textbook example within a reasonable round-off error.

Verification Problem 7: Steel Joist Design

Steel Joist Design per SJI

This problem represents a "special" steel joist with a non-uniform loading ('SP' joist). This model tests the equivalent uniform load calculations and subsequent joist size estimation. The hand verification of this problem can be taken directly from the Estimating Joist Size For Special Loadings Example in the ***Steel Joist Institute (SJI) Recommended Code of Standard Practice for Steel Joists and Joist Girders***, effective August 1st, 2002, p124.

Description & Problem Statement

A model was built per the description given in the example.

Span Length	= 30'-0"	Tapered Load (RLL)	= 160 plf @ 0'-0" to 0 plf @ 8'-0"
Beam Spacing	= 10'-0"	Point Load '1' (RLL)	= 500 lbs @ 21'-0"
Dead Load (DL)	= 9 psf	Point Load '2' (RLL)	= 800 lbs @ 23'-0"
Live Load (RLL)	= 18 psf	Point Load '3' (RLL)	= 300 lbs @ 27'-0"

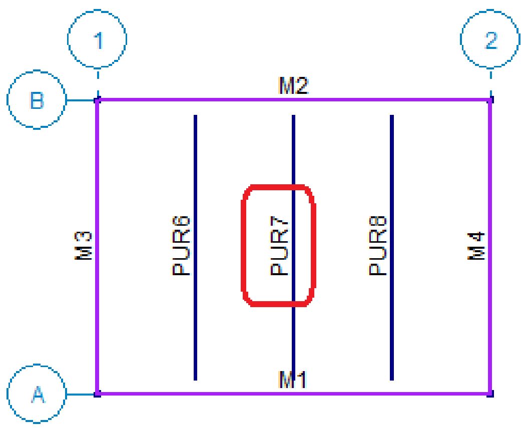


Figure 7.1

Validation Method

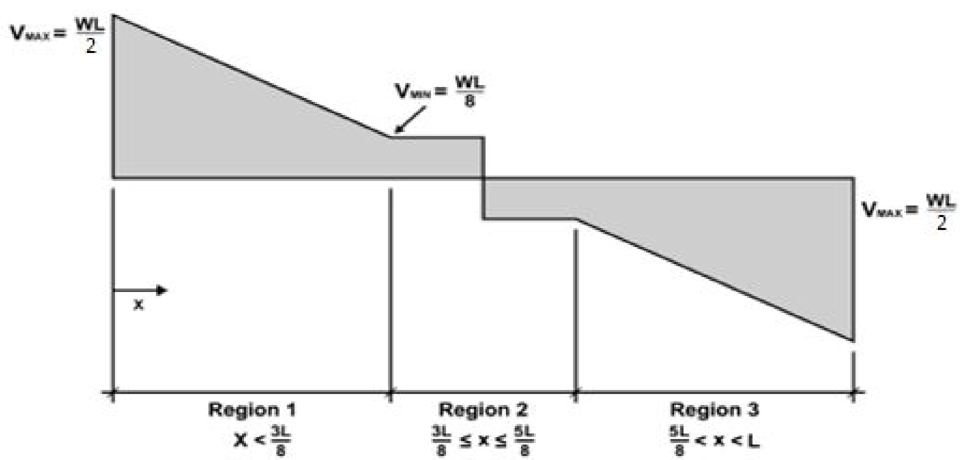
The equivalent uniform loads (UDL, ULL, & UTL) on member PUR7, shown above in Figure 7.1, are compared with the results given in the SJI example. Because the example assumes a joist self-weight that is included in the uniform dead load on the joist, the joist size has been hard coded to an 18K9 (10.2 plf). This combined with a dead load of 8 psf has been used to produce a 90 plf dead load on the member.

Comparison

Comparison of Results (Units Specified Individually)			
Value	RISAFloor	SJI Example	% Difference
R_L (lbs)	4926	5000	1.5
R_R (lbs)	5296	5340	0.8
UTL Due to R_{Max} (lbs/ft)	393	356	NA[1]
L_1 (V= 0 kips) (ft)	16.07	30-13.85 = 16.15[2]	0.5
M_{Max} @ L_1 (ft-lbs)	36,036	36,903	2.3
UTL Due to M_{Max} (lbs/ft)	Not Listed[3]	328	NA

Table 7.1 – Results Comparison

[1]The equivalent uniform total load (UTL) due to shear given in the SJI example is not calculated as is done in RISA. The SJI example uses the maximum end reaction to create a shear envelope. RISAFloor will choose a joist such that the capacity shear envelope of the joist under a standard uniform load is still greater than the actual shear at all locations along the length of the joist. Because the joist will have a shear strength greater than zero at the mid-span of the beam, the shear envelope for a joist with a maximum uniform load capacity of W, would be given by the figure shown below:



The minimum end reaction required to produce a shear envelope that will encompass the shear at X = 23'-0" is 5906.6 lbs. This equates to an equivalent uniform total load of UTL = 2*5906.6/30 = 394 lbs/ft. The program produces UTL = 392 lbs/ft (0.5% error), which is within a reasonable rounding error.

[2]The SJI example starts from the opposite end as is done in RISAFloor, so the inverse value is calculated here.

[3]The UTL due to M_{Max} is not specifically listed by the program because it did not control when compared to the UTL due to R_{Max}.

Conclusion

The program results match the SJI example for most values. Any items not matching are explained in the footnotes above.

Verification Problem 8: Composite ASD – 16th

AISC 16th Edition Design Examples – Composite Beam Design

This problem represents a composite beam (ASD). The hand verification of this problem can be taken directly from the ***AISC Design Examples Version 16.0*** (Example I.1, p.I-4 to I-16).

Description

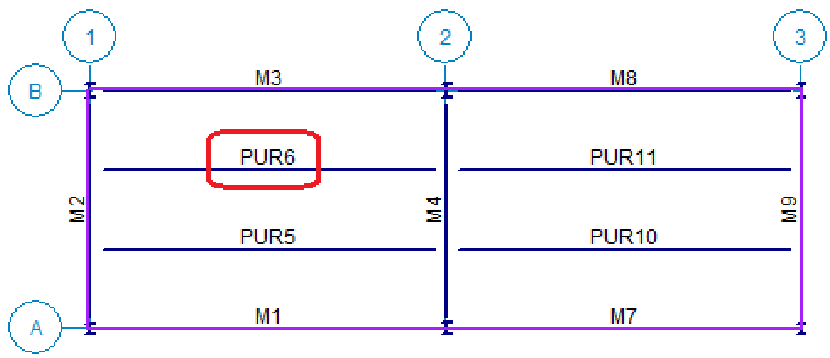


Figure 8.1

Validation Method

The model was built in RISAFloor exactly as it was described in the AISC design guide using ASD design. Results for member PUR6, shown above in Figure 8.1, are compared to the results in the design example.

Comparison

Comparison of Results (Units Specified Individually)			
Value	RISAFloor	AISC Example I.1	% Difference
Pre-Composite Flexural Strength Demand (kip-ft)	265.8	266.0	0.8
Pre-Composite DL Deflection (in)	0.59	2.59 – 2.00 = 0.59[1]	0.00
Composite Flexural Strength Demand, Mu (kip-ft)	480.9	481	0.02
Available Flexural Strength, Mn (kip-ft)	515.8[2]	512	0.74
Compression Block Depth, a (in)	0.971	0.946	2.61
Composite LL Deflection (in)	0.982[3]	1.26	22.1
Steel Anchor Shear Capacity (kips)	396.3	390.0	1.60

Table 8.1 – Results Comparison

[1]The AISC calculates a dead load deflection of 2.59 in and they specify to use a 2.00 in camber. The difference of the two would give a final dead load deflection of 0.59 in. RISAFloor gives a deflection value of 0.587 in.

[2]The available flexural strength is M_n / Ω_b. In RISAFloor these values are 861.318 kip-ft and 1.67, respectively.

[3]The AISC uses the I_{LB} to calculate the live load deflection, while RISA uses the I_{EFF}. I_{LB} = 2520 in^4 while I_{EFF} is calculated by RISA as 3241 in^4. Both methods are acceptable. The ratios of the deflections and the ratios of the moment of inertias is equal, thus the difference is explained (0.982/1.26 = 2520/3241).

Conclusion

The program results match the textbook example within a reasonable round-off error, with the exception of the difference in moment of inertia.

Verification Problem 9: Composite LRFD – 16th

AISC 16th Edition Design Examples – Composite Girder Design

This problem represents a composite girder (LRFD). The hand verification of this problem can be taken directly from the ***AISC Design Examples Version 16.0*** (Example I.2, Pages I-17 to I-35).

Description

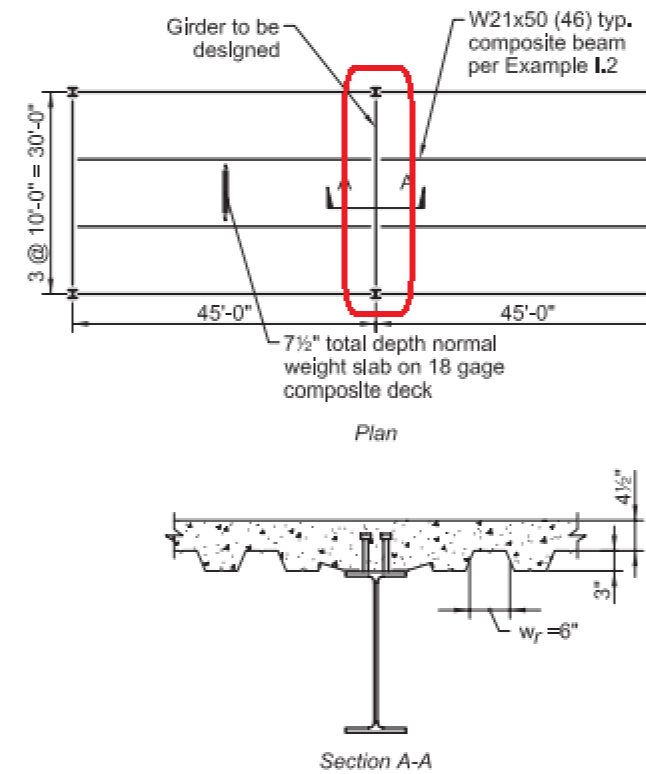


Figure 9.1

Validation Method

The model was built in RISAFloor exactly as it was described in the AISC design guide using LRFD. The results for member M4, as shown in Figure 9.1 above, are compared to the results in the design example.

Comparison

Comparison of Results (Units Specified Individually)			
Value	RISAFloor	AISC Example I.2	% Difference
Pre-Composite Flexural Strength Demand (kip-ft)	624.5	624	0.08
Pre-Composite (Non-Composite) Flexural Strength (kip-ft)	677.1	677	0.01
Pre-Composite Deflection (in)	0.297+ 0.75 = 1.047	1.00	4.49[1]
Composite Flexural Demand (kip-ft)	1218.5	1220	0.12
Steel Yielding, Cf (kips)	1120	1120	0.00
Composite Nominal Moment Resistance (ϕ *M_n) (kip-ft)	0.9*1430.33 = 1287.3[2]	1280	0.57
Steel Anchor Strength (kips)	21.54	21.5	0.19
Total Steel Anchor Strength (kips)	603.04 (28 anchors)[3]	581 (27 anchors)	n/a[3]
I_{tr} (in^4)	6776.8	6800	0.34
I_{equiv} (in^4)	5531[4]	5490	0.75
Live Load Deflection (in)	0.513	0.547	n/a[5]
ϕ *V_n (kips)	315.48	315	0.15

Table 9.1 – Results Comparison

[1]The AISC calculates a pre-composite deflection of 1.0 inches and then mentions a 0.75" camber. RISAFloor gives the PreDL deflection as 0.292" because it subtracts out the 0.75" camber. The non-cambered deflection is compared.

[2]The nominal moment resistance is ϕ *M_n. In RISAFloor these values are given separately.

[3]The program takes the total number of studs (26+3+26) and divides by two. This yields 27.5 anchors. The program then rounds up to the nearest anchor (28).

[4]RISAFloor's I_{eff} term corresponds to I_{equiv}.

[5]Similar to Verification Problem #8 (based on Example I.1 from the *AISC 16th edition Design Examples*), this example uses the I_{LB} whereas RISAFloor uses the I_{EFF}.

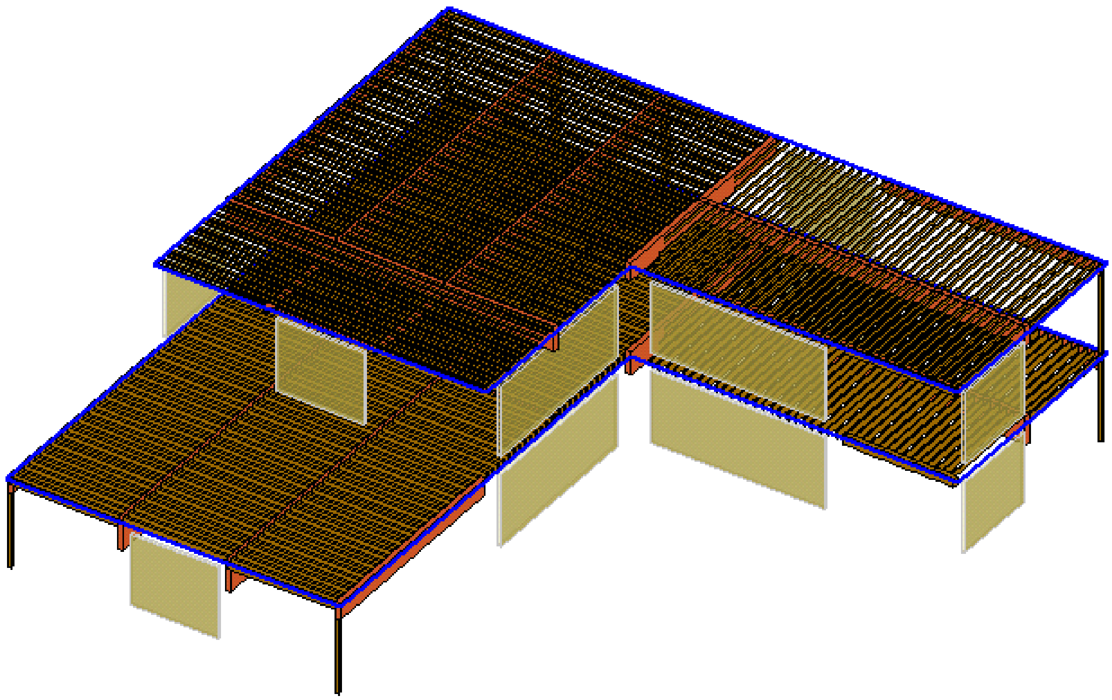
Conclusion

The program results match the textbook example within a reasonable round-off error.

Verification Problem 10: Live Load Reduction

Description

This problem represents a simple two story model that is loaded with reducible live loads on both the Roof and Floor levels. We will use this model to confirm the live load reduction calculations.

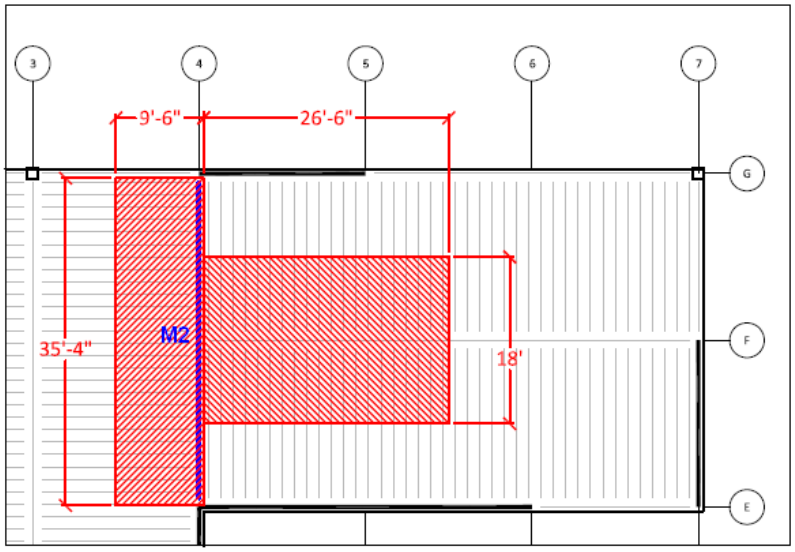


Validation Method

The values obtained by the RISAFloor model will be compared to the following hand calculations per the ASCE 7-2016, sections 4.7 and 4.8. The live load reduction reducible areas can be found in the detail report for each of the following members.

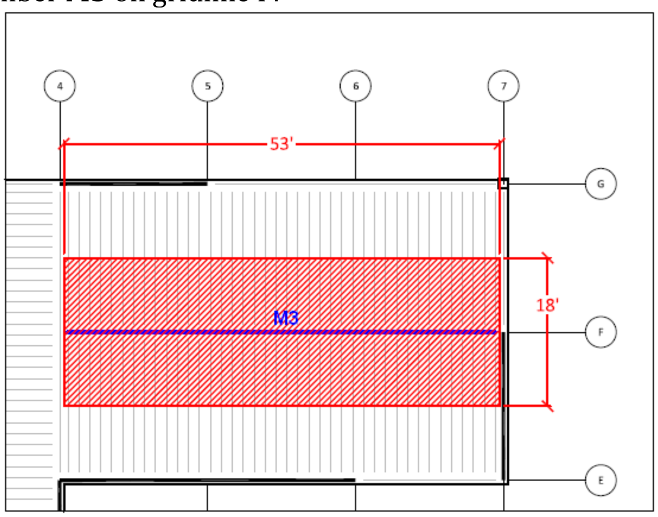
Roof Level

- Member **M2** on gridline 4:



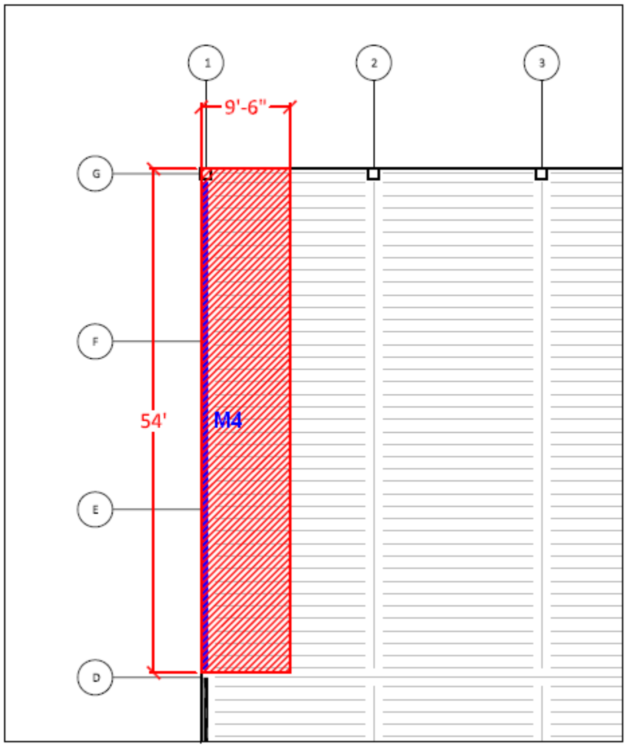
$$A_T := (35.333\ ft \cdot 9.5\ ft) + (26.5\ ft \cdot 18\ ft) = 812.664\ ft^2$$

- Member **M3** on gridline F:



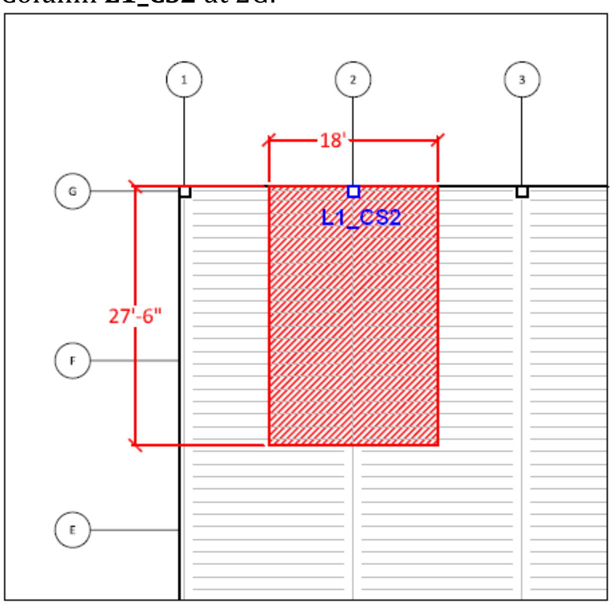
$$A_T := (53\ ft \cdot 18\ ft) = 954\ ft^2$$

- Member **M4** on gridline 1:



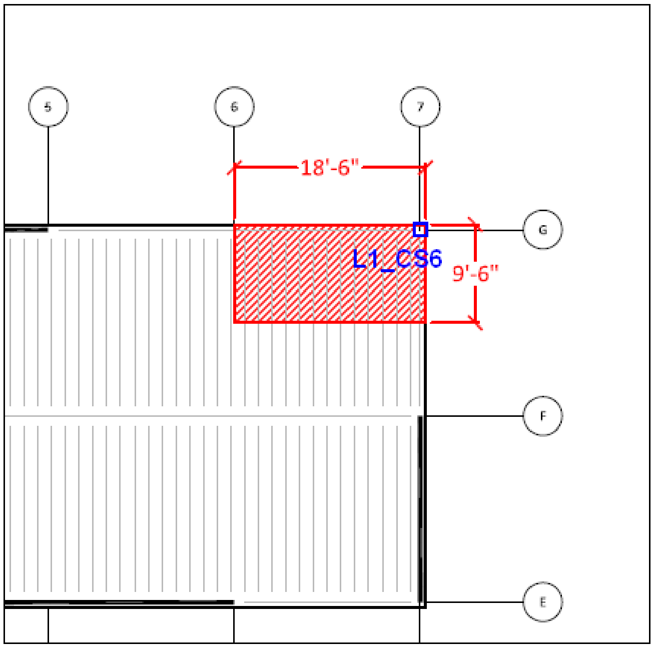
$A_T := (54\ ft \cdot 9.5\ ft) = 513\ ft^2$

- Column **L1_CS2** at 2G:



$A_T := (27.5\ ft \cdot 18\ ft) = 495\ ft^2$

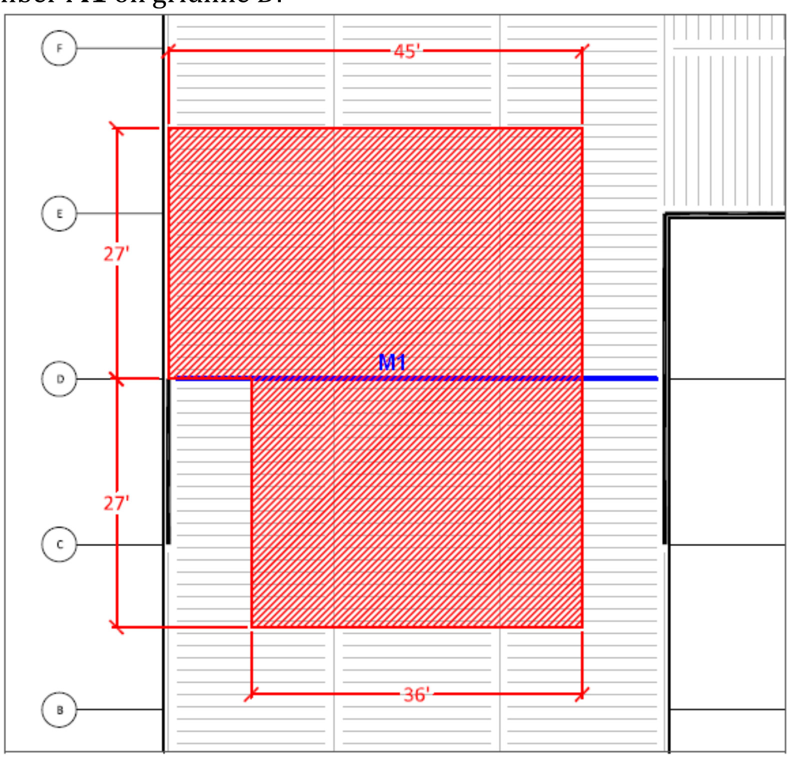
- Column **L1_CS6** at 1G:



$A_T := (18.5\ ft \cdot 9.5\ ft) = 175.75\ ft^2$

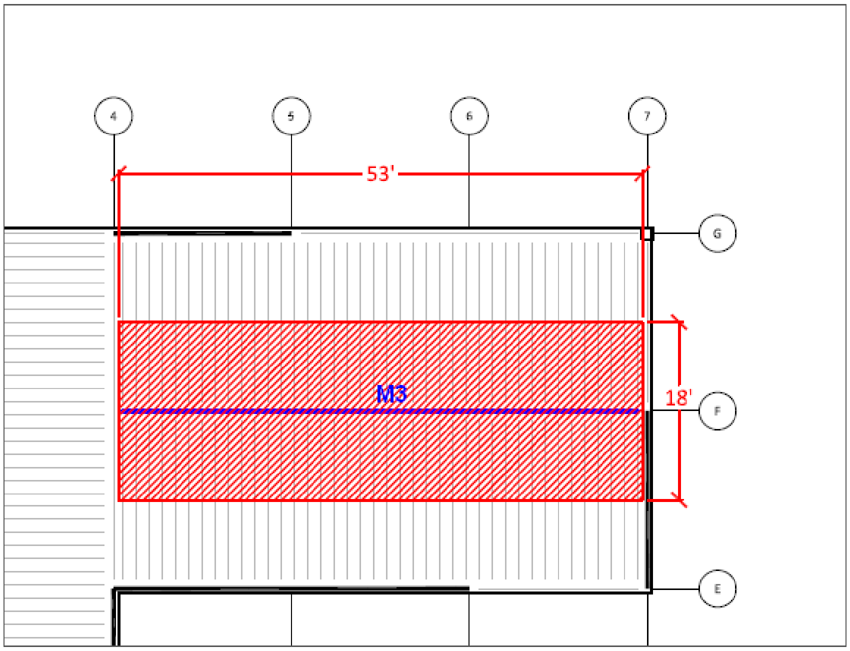
Floor Level

- Member **M1** on gridline D:



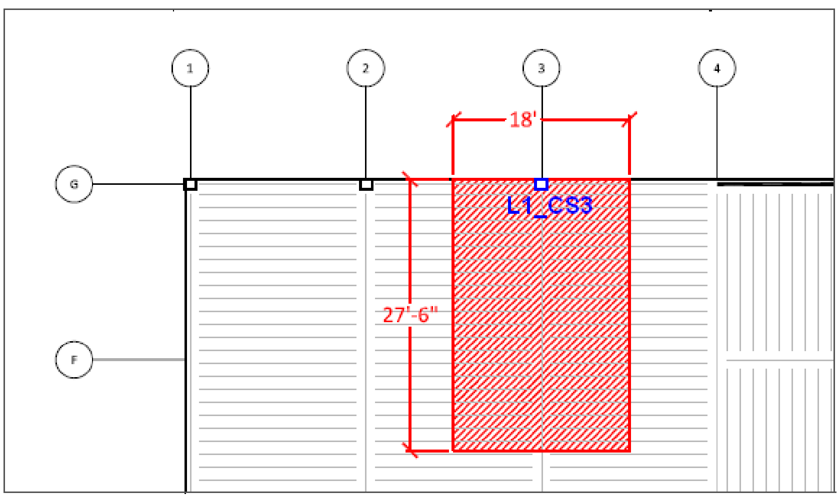
$A_T := (45\ ft \cdot 27\ ft) + (27\ ft \cdot 36\ ft) = 2187\ ft^2$

- Member **M3** on gridline F:



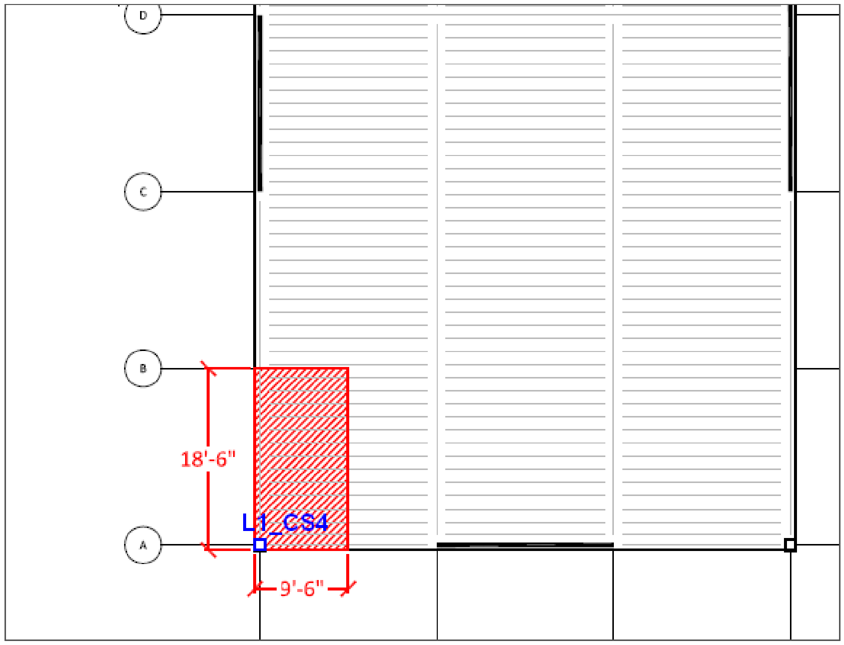
$$A_T := (53\ ft \cdot 18\ ft) = 954\ ft^2$$

- Column **L1_CS3** AT 3G:



$$A_{T\_Roof} := (27.5\ ft \cdot 18\ ft) = 495\ ft^2$$
$$A_{T\_Floor} := (27.5\ ft \cdot 18\ ft) = 495\ ft^2$$
$$A_{T\_Total} := A_{T\_Roof} + A_{T\_Floor} = 990\ ft^2$$

- Column **L1_CS4** at 1A:



$$A_T := (18.5\ ft \cdot 9.5\ ft) = 175.75\ ft^2$$

Comparison

Floor Level: Roof

	Unreduced Design Live Load	Trib. Area (Hand Calc)	Trib. Area (RISAFloor Calc)	Reduc. Factor	Reduc. Factor	Reduced Design Live Load			
Member	L_o (psf)	A_T (ft^2)	A_T (ft^2)	R1	R2	L_r (psf)	Factor	RISAFloor RLL Factor	% Diff
M2	20	812.66	819.01	0.60	1.00	12.00	0.6	0.6	0.00
M3	20	954.00	953.99	0.60	1.00	12.00	0.6	0.6	0.00
M4	20	513.00	513.00	0.69	1.00	13.74	0.687	0.687	0.00
CS2 (G-2)_L1	20	495.00	495.00	0.71	1.00	14.10	0.705	0.705	0.00
CS6 (G-7)_L1	20	175.75	175.54	1.00	1.00	20.00	1	1	0.00

Table 10.1 – Results Comparison at the Roof

Floor Level: Floor

	Unreduced Design Live Load	Trib. Area (Hand Calc)	Trib. Area (RISAFloor Calc)	Element Factor (per Table 4-2)	Reduced Design Live Load			
Member	L_o (psf)	A_T (ft^2)	A_T (ft^2)	K_{LL}	L (psf)	Factor	RISAFloor LL Factor	% Diff
M1	100	2187.00	2190.01	2	50	0.500	0.5	0.00
M3	100	954.00	953.99	2	59.34	0.593	0.593	0.00
CS3 (G-3)_L1	100	990.00	990.00	4	48.84	0.488	0.488	0.00
CS4 (A-1)_L1	100	175.75	175.542	4	81.57	0.816	0.816	0.00

Table 10.2 – Results Comparison at the Floor

Conclusion

The program results match the hand calculation results within a reasonable rounding error.