# TABLE OF CONTENTS

## PREFACE

1. **GETTING STARTED** ................................................................. 1
   1.1 MINIMUM SYSTEM REQUIREMENTS ..................................................... 1
   1.2 DOWNLOADING AND INSTALLATION ................................................... 1
   1.3 RUNNING THE PROGRAM ................................................................... 1
   1.4 TECHNICAL SUPPORT ..................................................................... 1

2. **TUTORIAL** .................................................................................. 3
   2.1 INTRODUCTION .............................................................................. 3
   2.2 ROOF FRAMING DESIGN ................................................................. 4
     2.2.1 ROOF RAFTER DESIGN ............................................................... 4
     2.2.2 MULTI-SPAN ROOF BEAM DESIGN ............................................... 7
     2.2.3 ROOF COLUMN DESIGN ............................................................. 8
   2.3 FLOOR FRAMING DESIGN ............................................................... 11
     2.3.1 FLOOR JOIST DESIGN ............................................................... 11
     2.3.2 HEADER DESIGN ................................................................... 13
     2.3.3 MULTI-SPAN STEEL FLOOR BEAM DESIGN ................................ 15
     2.3.4 STEEL COLUMN DESIGN ........................................................... 17
   2.4 FOUNDATION DESIGN .................................................................. 19
     2.4.1 SQUARE FOOTING DESIGN ....................................................... 19

3. **USING THE PROGRAM** .............................................................. 21
   3.1 HELP ......................................................................................... 21
     QUICKHELP ...................................................................................... 21
     HELP FILE ....................................................................................... 21
     MORE HELP ..................................................................................... 21
   3.2 AUTOSIZE ................................................................................... 21
   3.3 CALCULATE ................................................................................ 22
   3.4 STRESS VALUES ......................................................................... 22
   3.5 LOADING DIAGRAM ..................................................................... 22
   3.6 SHEAR, MOMENT, AND DEFLECTION DIAGRAM ......................... 22
   3.7 CALCULATOR ............................................................................. 22
   3.8 MEMBER NOTES ......................................................................... 23
   3.9 CHANGING THE BUILDING CODE .................................................. 23
   3.10 PRINTING ................................................................................ 23
     3.10.1 PRINTING MEMBERS ............................................................ 23
     PRINTING FROM THE MODULE ....................................................... 23
     PRINTING MULTIPLE MEMBERS ..................................................... 23
SPAN
TRIBUTARY WIDTH
ROUND MEMBERS
FLAT USE
5.2.3 DEFLECTION CRITERIA
RECOMMENDED DEFLECTION CRITERIA
DEFLECTION LIMIT CALCULATIONS FOR INTERIOR SPANS
DEFLECTION LIMIT OPTIONS FOR CANTILEVERS
5.2.4 DURATION FACTOR
CODE REQUIRED DURATION FACTOR VALUES
CONTROLLING DURATION FACTOR DETERMINATION
5.2.5 SNOW/NON-SNOW
5.2.6 UNBRACED LENGTH
5.2.7 SLOPE ADJUSTMENTS
ROOF BEAM-ROOF PITCH ADJUSTMENTS
ROOF BEAM-SLOPE AND ROOF PITCH ADJUSTMENTS
ROOF RAFTER-PITCH ADJUSTMENTS
5.2.8 LOADING
LOAD COMBINATIONS
DEAD LOADS
LIVE LOADS
5.2.9 FLOOR LIVE LOAD REDUCTION
REDUCTIONS FOR 94 AND 97 UBC, 2000, 2003 AND 2006 IBC
REDUCTIONS FOR 94, 97 AND 99 SBC
REDUCTIONS FOR 93, 96 AND 99 BOCA
5.2.10 CAMBER AMPLIFICATION FACTOR
6. DESIGN CALCULATIONS
6.1 GENERAL DESIGN CALCULATIONS
6.2 WOOD DESIGN
6.2.1 MATERIAL PROPERTIES
6.2.2 ADJUSTMENT FACTORS
SIZE FACTOR ADJUSTMENT
VOLUME FACTOR ADJUSTMENT
STABILITY FACTOR ADJUSTMENT
REPETITIVE MEMBER FACTOR
INCISING FACTOR
WET USE FACTOR
6.2.3 ADJUSTED ALLOWABLE STRESS VALUES
ADJUSTED BENDING STRESS – Fb’
ADJUSTED SHEAR STRESS – Fv’
ADJUSTED COMPRESSION PARALLEL TO GRAIN – Fc’
ADJUSTED COMPRESSION PERPENDICULAR TO GRAIN – Fc⊥’
ADJUSTED MODULUS OF ELASTICITY– E’
6.2.4 WOOD SECTION PROPERTIES
AREA
SECTION MODULUS
MOMENT OF INERTIA
6.2.5 REQUIRED WOOD SECTION PROPERTIES
AREA REQUIRED
SECTION MODULUS REQUIRED
MOMENT OF INERTIA REQUIRED
I-JOIST CALCULATIONS ........................................................................................................ 70
PLYWOOD CALCULATIONS .................................................................................................. 70

9.2 ROOF RAFTER ............................................................................................................. 71
9.2.1 DESCRIPTION ........................................................................................................... 71

10. COLUMNS .................................................................................................................... 73

10.1 INPUTS AND DESCRIPTION ...................................................................................... 72
    DESCRIPTION ................................................................................................................ 73
    UNBRACED LENGTH ...................................................................................................... 73
    COLUMN DIMENSIONS ............................................................................................... 74
    COLUMN END CONDITION ............................................................................................ 74
    COLUMN BENDING COEFFICIENT ............................................................................... 74
    LOAD ECCENTRICITY .................................................................................................... 74
    LATERAL LOADS ............................................................................................................. 74
    LAMINATED COLUMNS ................................................................................................. 74
    MACHINE STRESS RATED LUMBER ............................................................................. 75
    STUD DESIGN ................................................................................................................ 75
    CANTILEVER COLUMN ................................................................................................. 75

10.2 COLUMN CALCULATIONS ........................................................................................... 76
    DEFLECTION .................................................................................................................. 76
    SHEAR ............................................................................................................................ 76
    MOMENT ........................................................................................................................ 76
    LOAD COMBINATIONS .................................................................................................... 76

10.3 WOOD COLUMNS ...................................................................................................... 77
    10.3.1 PROPERTIES AND CALCULATIONS ................................................................ 77
    TABULATED WOOD COLUMN DESIGN STRESSES .................................................... 77
    ADJUSTED DESIGN STRESSES ................................................................................... 77
    DURATION FACTOR AND LOAD CASES ....................................................................... 77
    SECTION MODULUS ...................................................................................................... 77
    COMPRESSION SLENDERNESS RATIO .......................................................................... 78
    COMPRESSIVE STRESS ................................................................................................. 78
    BENDING STRESS ......................................................................................................... 78
    BASE REACTIONS .......................................................................................................... 78
    ADJUSTMENT FACTORS ............................................................................................... 79
    COLUMN STABILITY FACTOR ...................................................................................... 79
    BUILT UP COLUMNS ..................................................................................................... 79
    COMBINED STRESS FACTOR ....................................................................................... 79
    COLUMN ADEQUACY .................................................................................................... 80

10.4 STEEL COLUMNS ....................................................................................................... 81
    10.4.1 PROPERTIES AND CALCULATIONS ................................................................. 81
    TABULATED STEEL COLUMN DESIGN PROPERTIES .................................................. 81
    COMPRESSIVE STRESS ................................................................................................. 81
    COLUMN SLENDERNESS RATIO ................................................................................... 81
    ALLOWABLE COMPRESSIVE STRESS ........................................................................... 81
    BENDING STRESS ......................................................................................................... 82
    ALLOWABLE BENDING STRESS ..................................................................................... 82
    EULER’S STRESS ............................................................................................................ 82
    COLUMN COMBINED STRESS CALCULATIONS .......................................................... 82
    BASE REACTIONS .......................................................................................................... 83
    COLUMN ADEQUACY .................................................................................................... 83
11. FOOTINGS

11.1 DESCRIPTION

11.1.1 SQUARE/RECTANGULAR FOOTING INPUTS
- COLUMN TYPE
- LOADING
- MATERIAL PROPERTIES
- SOIL BEARING PRESSURE
- REINFORCEMENT
- CONCRETE COVER
- PLAIN CONCRETE
- COLUMN DIMENSIONS
- BASEPLATE
- FOOTING DIMENSIONS

11.1.2 CONTINUOUS FOOTING INPUTS
- STEMWALL TYPE AND WEIGHT

11.1.3 FOOTING DESIGN PROCESS

11.1.4 INADEQUACY TROUBLESHOOTING
- FOOTING SIZE INADEQUATE
- FOOTING DEPTH INADEQUATE
- ONE-WAY SHEAR FAILURE
- PUNCHING SHEAR FAILURE
- FOOTING WEIGHT EXCEEDS BEARING PRESSURE
- BEARING INADEQUATE (SQUARE/RECTANGULAR/ROUND FOOTING ONLY)
- FAILURE IN BENDING
- DEVELOPMENT LENGTH INADEQUATE
- PLAIN CONCRETE FLEXURE FAILURE

11.2 CALCULATIONS

11.2.1 GENERAL CALCULATIONS
- EFFECTIVE SOIL BEARING CAPACITY
- LOADING
- FOOTING SIZE
- FOOTING DEPTH

11.2.2 BEARING CALCULATIONS (SQUARE/RECTANGULAR FOOTING ONLY)
- CONCRETE BEARING STRENGTH

11.2.3 PUNCHING SHEAR CALCULATIONS (SQUARE/RECTANGULAR FOOTING ONLY)
- CRITICAL PERIMETER
- PUNCHING SHEAR
- ALLOWABLE PUNCHING SHEAR STRESS
- PUNCHING SHEAR ADEQUACY

11.2.4 BEAM SHEAR CALCULATIONS
- BEAM SHEAR STRESS
- ALLOWABLE BEAM SHEAR STRESS
- BEAM SHEAR ADEQUACY

11.2.5 REINFORCEMENT CALCULATIONS
- STEEL REQUIRED BASED ON MOMENT
- MINIMUM STEEL REQUIREMENTS
- SPACING REQUIREMENTS
- REINFORCEMENT PROVIDED
- BAND WIDTH REINFORCEMENT (RECTANGULAR FOOTINGS ONLY)

11.2.6 REINFORCEMENT DEVELOPMENT LENGTH CALCULATIONS
- DEVELOPMENT LENGTH REQUIRED (ACI 318-89) (Revised 1992)
- DEVELOPMENT LENGTH REQUIRED (ACI 318-95, 99, 02, 05)
12. COLLAR TIE

12.1 INPUTS AND DESCRIPTION

DESCRIPTION

FIGURE 12-1 COLLAR TIE

LOADING

COLLAR TIE

BRACING

DESIGN PROCESS

CONNECTION DESIGN

12.2 COLLAR TIE CALCULATIONS/ASSUMPTIONS

APPENDIX A- SOLID SAWN SPECIES CLASSIFICATIONS

NAILED LAMINATED COLUMNS

BOLTED LAMINATED COLUMNS

APPENDIX A - SOLID SAWN SPECIES CLASSIFICATION

APPENDIX B - BOLTING AND NAILING REQUIREMENTS FOR LAMINATED COLUMNS

APPENDIX C - NOMENCLATURE

REFERENCES

INDEX
PREFACE

The purpose of StruCalc is to provide a fast, easy-to-use, engineering tool to size most of the common structural members required for residential and light commercial design. StruCalc is not designed to replace the need for a structural engineer. However, it is intended for designers and architects to do preliminary sizing and design of structural members. Furthermore, StruCalc is designed to assist the structural engineer by performing many of the tedious and time consuming calculations required for the simplest of structural applications. It's easy-to-use interface and single member design flexibility has also made it a valuable tool for many building officials across the United States.

The need for an easy-to-use structural program became evident to us after several years of residential and commercial design. A majority of our office time was spent doing repetitive and time-consuming hand calculations. Fortunately, the advent of the affordable personal computer opened a new era in engineering design. It gave the small, one-man businesses access to high end computing power. However, most of the programs on the market were (and still are) complicated and expensive. Therefore, we have endeavored to meet the need for a reasonably priced, fast, and easy-to-use structural analysis program.

We trust that StruCalc 8.0 will meet your needs and expectations.

The StruCalc Design Team
1. GETTING STARTED

1.1 MINIMUM SYSTEM REQUIREMENTS

- Microsoft Windows 2000, NT, XP, or Vista (32 bit or 64 bit).
- 70 MB of free hard drive space
- Internet access for authorization and updating
- Microsoft .Net Framework, can be downloaded at www.microsoft.com/net

StruCalc will run on any computer capable of running Windows, but as is typical of most software, a faster machine will provide better performance.

1.2 DOWNLOADING AND INSTALLATION

The unlockable program can be downloaded at www.strucalc.com. Follow the directions online in order to download the program to your computer or movable storage media.

From Downloaded Install File:

1. Start Windows.
2. Double Click on the Downloaded Executable.
3. Follow the directions on the screen.
4. Enter you license key that was provided on your purchase receipt. Contact Cascade Consulting Associates if you do not have your license key.

From optional CD:

1. Start Windows.
2. Insert the StruCalc 8.0 CD in the CD-ROM drive.
3. If the install program starts automatically skip to step 7.
4. Click Start on the Windows taskbar and choose My Computer.
5. Double Click on the drive containing the StruCalc 8.0 CD.
6. Double-click on the Setup.exe contained on the CD.
7. Follow the directions on the screen.
8. Enter you license key that was provided on your purchase receipt. Contact Cascade Consulting Associates if you do not have your license key.

1.3 RUNNING THE PROGRAM

To run StruCalc 8.0 choose StruCalc 8.0 from the Start menu or Double Click on the icon on the Windows Desktop.

1.4 TECHNICAL SUPPORT

If you did not order priority technical support when you purchased StruCalc 8.0 you can still receive free email technical support. If you would like to order technical support call us at (800) 279-1353 or go to our website and place your order at www.strucalc.com. For technical support call (541) 753-6112 Monday through Friday 9:00 AM to 5:00 PM Pacific Standard Time, fax questions to (541) 753-9422, or e-mail us at: techelp@strucalc.com.
2. TUTORIAL

2.1 INTRODUCTION

This tutorial is intended for the first time user of StruCalc, or for any user who needs a review on how the program works. In addition, this tutorial is intended to illustrate the ease in which StruCalc can be used to design structural members.

We will be designing several structural members found in a typical residence using the 2006 International Building Code. If you use a different building code the answers may not match exactly. First we will design the roof framing members and then we will systematically work our way through the floor framing and then down to the footings. See figure 2-1.

FIGURE 2-1 STRUCTURAL SECTION
2.2 ROOF FRAMING DESIGN

To start off the structural analysis of the residence we will design three roof framing members: the roof rafters, a continuous roof beam, and a wood post. See figure 2-2

![Figure 2-2 Roof Framing Plan](image)

**FIGURE 2-2 ROOF FRAMING PLAN**

2.2.1 ROOF RAFTER DESIGN

The interior span length of the roof rafters in figure 2-2 is 12 feet and the eave span length is two feet. The unbraced length of the rafters is zero, since the roof sheathing continuously braces the rafters. The roof dead load is the combined weight of the roofing materials and the rafters, in this situation it is 15 pounds per square foot. The region and applicable building code determine the roof live load, for this case it will be 25 pounds per square foot. (snow zone). Also the rafters have a 4:12 pitch.

To design the rafters using StruCalc perform the following steps:

1. Start **StruCalc 8.0 for Windows**.

2. **Click** the **Roof Rafter** button.
3. Change the material properties on Navigation Toolbar to the following:

![Navigation Toolbar]

4. Change the **Roof Rafter Module** inputs to the following (note that you can quickly move through the input boxes by pressing the tab key):

![Roof Rafter Module]

5. Click the **Calculate** button or press Enter.
The design is inadequate for both moment and deflection.

### StruCalc Adequacy

<table>
<thead>
<tr>
<th>Inadequate By: 47.4 %</th>
<th>Controlling Factor: Moment / Depth Required 6.68 in.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Moment: 47.4 %</td>
</tr>
</tbody>
</table>

6. **Click** the AutoSize button on the Navigation Toolbar on the left. **Click** on the #2 to highlight it in the AutoSize form.

7. **Click** the AutoSize button in the top toolbar or double click the #2 grade highlighted above to start the AutoSize process.

8. **Select** the 2 x 8 at 24” o.c. as shown below and **click Select and Return button** or **double click** the highlighted selection to load it into the current design.

<table>
<thead>
<tr>
<th>Grade</th>
<th>Laminations</th>
<th>Width</th>
<th>Depth</th>
<th>O.C. Spacing</th>
<th>Adequacy</th>
</tr>
</thead>
<tbody>
<tr>
<td>#2</td>
<td>1</td>
<td>1.5</td>
<td>5.5</td>
<td>8</td>
<td>103.5%</td>
</tr>
<tr>
<td>#2</td>
<td>1</td>
<td>1.5</td>
<td>5.5</td>
<td>12</td>
<td>35.7%</td>
</tr>
<tr>
<td>#2</td>
<td>1</td>
<td>1.5</td>
<td>5.5</td>
<td>16</td>
<td>1.7%</td>
</tr>
<tr>
<td>#2</td>
<td>1</td>
<td>1.5</td>
<td>7.25</td>
<td>19.2</td>
<td>36%</td>
</tr>
<tr>
<td>#2</td>
<td>1</td>
<td>1.5</td>
<td>7.25</td>
<td>24</td>
<td>8.8%</td>
</tr>
<tr>
<td>#2</td>
<td>1</td>
<td>1.5</td>
<td>9.25</td>
<td>32</td>
<td>5.9%</td>
</tr>
<tr>
<td>#2</td>
<td>1</td>
<td>3.5</td>
<td>3.5</td>
<td>8</td>
<td>22.8%</td>
</tr>
<tr>
<td>#2</td>
<td>1</td>
<td>3.5</td>
<td>5.5</td>
<td>12</td>
<td>216.5%</td>
</tr>
<tr>
<td>#2</td>
<td>1</td>
<td>3.5</td>
<td>5.5</td>
<td>16</td>
<td>137.4%</td>
</tr>
<tr>
<td>#2</td>
<td>1</td>
<td>3.5</td>
<td>5.5</td>
<td>19.2</td>
<td>97.8%</td>
</tr>
<tr>
<td>#2</td>
<td>1</td>
<td>3.5</td>
<td>5.5</td>
<td>24</td>
<td>58.3%</td>
</tr>
<tr>
<td>#2</td>
<td>1</td>
<td>3.5</td>
<td>5.5</td>
<td>32</td>
<td>3.2%</td>
</tr>
</tbody>
</table>

The design is adequate.

### StruCalc Adequacy

<table>
<thead>
<tr>
<th>Adequate By: 8.8 %</th>
<th>Controlling Factor: Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Moment: 8.8 %</td>
</tr>
</tbody>
</table>
2.2.2 MULTI-SPAN ROOF BEAM DESIGN

The length of the continuous ridge beam is 30 feet. The beam is supported at the midpoint and therefore has two equal spans of 15 feet. See figure 2-2. The unbraced length at the top of the beam is two feet, due to the bracing of the roof rafters at that spacing. The unbraced length of the bottom of the beam is the distance between supports. The roof dead load is the combined weight of the roofing materials and the rafters and we are going to use 15 pounds per square foot. Beam self-weight is taken into account by the program and does not need to be entered. The region and applicable building code determine the roof live load, for this case it will be 25 pounds per square foot (snow zone). The beam is supporting a roof tributary width of six feet on both sides and the roof pitch is 4:12. The tributary width is half of the rafter span on each side of the beam.

To design this beam using StruCalc perform the following steps:

1. Click the Multi-Span Roof Beam button.

2. Change the material properties on Navigation Toolbar to the following:

   ![Material Properties Image]

   Note: In this case we have chosen to use an unbalanced glulam (24F-V4) for a multi-span situation. A balanced glulam such as a 24F-V8 will have a smaller beam size for a multi-span situation because it is much more efficient. However, when designing glulam beams for small projects, the cost of a 24F-V8 and the time it takes to get one (lumber yards usually have to special order them) is often times not justified.

3. Change the Multi-Span Roof Beam Module to the following:

   ![Beam Module Image]
4. *Click* the **AutoSize** button on the **Navigation Toolbar** on the left. *Click* on the 2F-V4 to highlight it in the AutoSize form.

5. *Click* the **AutoSize** button in the top toolbar or double click the 24F-V4 grade highlighted above to start the AutoSize process.

6. *Highlight* the 5.125 x 10.5 as shown below and *click Select and Return button* or *double click* the highlighted selection to load it into the current design.

   The design is adequate.

2.2.3 **ROOF COLUMN DESIGN**

The length of the column supporting the multi-span roof beam is approximately 11 ½ feet. See figure 2-1. Since the column is not framed in a wall the unbraced length is 11 ½ feet in both the X and Y direction. For this design we will assume a pinned-pinned condition, therefore Ke is equal to one. The load eccentricity is zero in both directions and the duration factor is 1.15 due to the roof snow load. The live (snow) load on the column is 5625 lbs and the dead load is 3776 lbs as shown in figure 2-3 below under Reactions “B” on the preview analysis. You can view the design output by either

*clicking* the Print Preview button on the **Navigation toolbar** on the left or by printing the analysis by *clicking* the print button in the bottom toolbar.
To design this column using StruCalc perform the following steps:

1. Click the **Column** button.

2. Change the material properties on **Navigation Toolbar** to the following:

3. Change the **Column Module** form to the following:
4. Click the **AutoSize** button on the **Navigation Toolbar** at the left. Select the #1 and #2 grade by holding down the **Ctrl** Key on your keyboard and **clicking** the two grades.

![Select Grade and click auto-size](image)

5. **Click** the **AutoSize** button in the **top** toolbar to start the AutoSize process.

6. **Highlight** the #1 5.5 x 5.5 as shown below and **click Select and Return button** or **double click** the highlighted selection to load it into the current design.

![Grade Laminations Width Depth Adequacy](image)

<table>
<thead>
<tr>
<th>Grade</th>
<th>Laminations</th>
<th>Width</th>
<th>Depth</th>
<th>Adequacy</th>
</tr>
</thead>
<tbody>
<tr>
<td>#1</td>
<td>1</td>
<td>3.5</td>
<td>13.25</td>
<td>28.8%</td>
</tr>
<tr>
<td>#1</td>
<td>1</td>
<td>5.5</td>
<td>5.5</td>
<td>32%</td>
</tr>
<tr>
<td>#1</td>
<td>1</td>
<td>7.5</td>
<td>7.5</td>
<td>81.1%</td>
</tr>
<tr>
<td>#1</td>
<td>1</td>
<td>9.5</td>
<td>9.5</td>
<td>89.6%</td>
</tr>
<tr>
<td>#2</td>
<td>1</td>
<td>3.5</td>
<td>13.25</td>
<td>15.4%</td>
</tr>
<tr>
<td>#2</td>
<td>1</td>
<td>5.5</td>
<td>7.5</td>
<td>40.4%</td>
</tr>
<tr>
<td>#2</td>
<td>1</td>
<td>7.5</td>
<td>7.5</td>
<td>74.4%</td>
</tr>
<tr>
<td>#2</td>
<td>1</td>
<td>9.5</td>
<td>9.5</td>
<td>85.5%</td>
</tr>
</tbody>
</table>

The design is adequate:

![Adequate By 32.0 % Controlling Factor: Combined Stress Factor](image)

**StruCalc Adequacy**

Congratulations! You have finished designing the roof members now move on to the floor framing.
2.3 FLOOR FRAMING DESIGN

Now we will design four floor framing members: the floor joists, a header, a continuous steel floor beam, and a steel column. See figure 2-4 below.

![Floor Framing Plan](image)

**FIGURE 2-4 UPPER FLOOR FRAMING PLAN**

2.3.1 FLOOR JOIST DESIGN

The span length of the floor joists in figure 2-4 is 12 feet. The unbraced length of the floor joists is zero, because the joists are continuously supported by the floor sheathing. Gypsum wallboard will be applied to the bottom of the joists therefore the bottom of the joists will be fully braced. The floor dead load is the combined weight of the floor materials and the joists, in this situation it is 15 pounds per square foot. The applicable building code determines the floor live load, for this case it will be 40 pounds per square foot.

To design the floor joists using StruCalc perform the following steps:

1. **Click** the Floor Joist button.
2. Change the **Navigation Toolbar** to the following:

![Navigation Toolbar](image)

3. Change the **Floor Joist Module** form to the following:

![Floor Joist Module](image)

4. *Click* the **Autosize** button on the **Navigation Toolbar** on the left. *Click* on the #2 to highlight it in the AutoSize form.
5. Click the Autosize button in the top toolbar or double click the #2 grade highlighted above to start the AutoSize process.

6. Highlight the 2x8 @ 16” O.C. selection and click Select and Return or double click the selection.

<table>
<thead>
<tr>
<th>Grade</th>
<th>Laminations</th>
<th>Width</th>
<th>Depth</th>
<th>O.C. Spacing</th>
<th>Adequacy</th>
</tr>
</thead>
<tbody>
<tr>
<td>#2</td>
<td>1</td>
<td>1.5</td>
<td>5.5</td>
<td>8</td>
<td>11.6%</td>
</tr>
<tr>
<td>#2</td>
<td>1</td>
<td>1.5</td>
<td>7.25</td>
<td>12</td>
<td>37.4%</td>
</tr>
<tr>
<td>#2</td>
<td>1</td>
<td>1.5</td>
<td>7.25</td>
<td>16</td>
<td>3%</td>
</tr>
<tr>
<td>#2</td>
<td>1</td>
<td>1.5</td>
<td>9.25</td>
<td>19.2</td>
<td>28.1%</td>
</tr>
<tr>
<td>#2</td>
<td>1</td>
<td>1.5</td>
<td>9.25</td>
<td>24</td>
<td>2.5%</td>
</tr>
<tr>
<td>#2</td>
<td>1</td>
<td>1.5</td>
<td>13.25</td>
<td>32</td>
<td>12.2%</td>
</tr>
<tr>
<td>#2</td>
<td>1</td>
<td>3.5</td>
<td>5.5</td>
<td>8</td>
<td>117.7%</td>
</tr>
<tr>
<td>#2</td>
<td>1</td>
<td>3.5</td>
<td>5.5</td>
<td>12</td>
<td>56.3%</td>
</tr>
<tr>
<td>#2</td>
<td>1</td>
<td>3.5</td>
<td>5.5</td>
<td>16</td>
<td>25.1%</td>
</tr>
<tr>
<td>#2</td>
<td>1</td>
<td>3.5</td>
<td>5.5</td>
<td>19.2</td>
<td>9.4%</td>
</tr>
<tr>
<td>#2</td>
<td>1</td>
<td>3.5</td>
<td>7.25</td>
<td>24</td>
<td>73.6%</td>
</tr>
<tr>
<td>#2</td>
<td>1</td>
<td>3.5</td>
<td>7.25</td>
<td>32</td>
<td>13.2%</td>
</tr>
</tbody>
</table>

The design is adequate

![StruCalc Adequacy](image)

### 2.3.2 HEADER DESIGN

The length of the header in figure 2-4 is five feet six inches long. The unbraced length at the top of the header is 16 inches, the same as the joist spacing. Note: If there is a knee wall above the header, then the unbraced length would be the distance between supports, not 16 inches on center. The unbraced length of the bottom of the header is the distance between supports. The floor dead load is the combined weight of the floor materials and the joists, in this situation it is 15 pounds per square foot. Beam self-weight is taken into account by the program so it does not need to be included in the dead load you enter. The applicable building code determines the floor live load, for this case it will be 40 pounds per square foot. The header is supporting a floor tributary width of six feet. The roof live load and dead load is 25 psf and 15 psf, respectively. The header is supporting a roof tributary width of 6ft. The roof pitch is 4:12. The wall above the header is eight feet tall and it weighs approximately 10 psf/foot of wall height, therefore the total wall load on the header is 80 plf.

Note: Additional load is transferred to the lower bearing support when roof rafters have cantilevered eaves. So in this case we'll add an additional tributary width of 2 ft to side two.

To design the header using StruCalc perform the following steps:

1. Click the Combination Roof and Floor Beam button.
2. Change the **Navigation Toolbar** to the following:

![Navigation Toolbar Diagram]

3. Change the **Combination Roof and Floor Beam** form to the following:

![Combination Roof and Floor Beam Form]

4. ![Warning Icon] Click the **Autosize** button on the **Navigation Toolbar** on the left. Click on the #2 to highlight it in the AutoSize form.

![Autosize Button]

Select Grade and click auto-size
Select Structural
#1
#1 & Btr
#2
#3
Construction
Standard
Stud
Utility
2.3.3 MULTI-SPAN STEEL FLOOR BEAM DESIGN

The length of the continuous beam in the center of the floor system in figure 2-4 is 30 feet. A column supports the beam 20 feet from the left side of the residence. The beam consists of two spans: 20 feet and 10 feet. The unbraced length at the top of the beam is 16 inches due to the floor joists providing lateral support at that spacing. The unbraced length at the bottom of the beam is the distance between supports. The floor dead load is the combined weight of the floor materials and the joists, in this situation it is 15 pounds per square foot. The beam self-weight is taken into account by the program. The applicable building code determines the uniform floor live load, for this case it will be 40 pounds per square foot. There is a point load on the beam 15 feet from the left support. See figure 2-4. The live (snow) load is 5625 lbs and the dead load is 3851 lbs as shown in figure 2-6 below under “Vertical Reactions”.

![FIGURE 2-6 ROOF POST PRINTOUT](image)

To design this beam using StruCalc perform the following steps:

1. **Click the Multi-Span Floor Beam** button.
2. Change the **Navigation Toolbar** to the following:

![Navigation Toolbar](image1.png)

4. Change the **Multi-Span Floor Beam Module** to the following:

![Multi-Span Floor Beam Module](image2.png)

5. *Click* the **AutoSize** button on the **Navigation Toolbar** on the left. Hold down the Ctrl button and *Click* W8, W10, and W12.

![AutoSize button](image3.png)
6. Highlight the W12x30 selection and click Select and Return or double click the selection.

<table>
<thead>
<tr>
<th>Grade</th>
<th>Section Required</th>
<th>Adequacy</th>
</tr>
</thead>
<tbody>
<tr>
<td>W8</td>
<td>W8x31</td>
<td>19.1%</td>
</tr>
<tr>
<td>W10</td>
<td>W10x33</td>
<td>39.9%</td>
</tr>
<tr>
<td>W12</td>
<td>W12x30</td>
<td>0.2%</td>
</tr>
</tbody>
</table>

The design is adequate

2.3.4 STEEL COLUMN DESIGN
The length of the column supporting the multi-span floor beam is approximately nine feet. See figure 2-1. The unbraced length of the column is nine feet in both the X and Y direction. For this column design we will assume a pinned-pinned condition, therefore Ke is equal to one. The exact column bending coefficient (Cm) may be calculated or it may be assumed to be equal to one, which is a conservative assumption. The live load on the column is 15964 lbs and the dead load is 8477 lbs as shown in figure 2-7 under Reactions “Line B”. The load eccentricity on the post is zero in both directions. However it is recommended that a minimum eccentricity of one inch or 1/10th of the member dimension be used, whichever is larger. In this situation we will assume an eccentricity of ½” in both directions to be conservative. Note that eccentricity will significantly affect the column design; however its inclusion is not required by any of the building codes. The duration factor of the loads on the column is 1.00.

<table>
<thead>
<tr>
<th>REACTIONS</th>
<th>A</th>
<th>B</th>
<th>C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Live Load</td>
<td>4791 lb</td>
<td>15964 lb</td>
<td>2200 lb</td>
</tr>
<tr>
<td>Dead Load</td>
<td>2248 lb</td>
<td>8483 lb</td>
<td>-580 lb</td>
</tr>
<tr>
<td>Total Load</td>
<td>7039 lb</td>
<td>24448 lb</td>
<td>1620 lb</td>
</tr>
<tr>
<td>Uplift (1.5 F.S)</td>
<td>0 lb</td>
<td>0 lb</td>
<td>-3410 lb</td>
</tr>
<tr>
<td>Bearing Length</td>
<td>0.74 in</td>
<td>0.74 in</td>
<td>0.74 in</td>
</tr>
</tbody>
</table>

**FIGURE 2-7 MULTI-SPAN FLOOR BEAM REACTIONS**

To design this column using StruCalc perform the following steps:

1. Click the Column button.
2. Change the Navigation Toolbar to the following:
3. Change the **Column Module** to the following:

![Column Module Table]

6. **Click** the **AutoSize** button on the **Navigation Toolbar** on the left. Hold down the Ctrl button and **Click** HSS 3, 3/12 and 4.

![AutoSize Button]

7. **Highlight** the HSS 3 ¼ x 3 ¼ x 3/16 selection and **click Select and Return** or **double click** the selection.

![Highlight and Select]

The design is adequate

![Adequacy Check]

Congratulations! You have finished designing the floor framing members.
2.4 FOUNDATION DESIGN
To finish off this tutorial we will size a square footing to support the steel column that we just designed.

2.4.1 SQUARE FOOTING DESIGN
The footing is supporting a 3 ½” x 3 ½” x 3/16” tube steel column. The base plate is a 6” x 6” x ½” steel plate. The allowable soil bearing pressure in this case is 1500 psf. The live load and dead load on the footing are 15964 lbs and 8557 lbs, respectively. See figure 2-8. The concrete compressive strength used in this design is 2500 psi, therefore no special inspection is required. The steel reinforcement bars are #4’s with a yield strength of 40000psi.

<table>
<thead>
<tr>
<th>VERTICAL REACTIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Live Load: Vert-LL-Rxn = 15964 lb</td>
</tr>
<tr>
<td>Dead Load: Vert-DL-Rxn = 8557 lb</td>
</tr>
<tr>
<td>Total Load: Vert-TL-Rxn = 24521 lb</td>
</tr>
</tbody>
</table>

**FIGURE 2-8 STEEL COLUMN PRINTOUT**

To design this footing using StruCalc perform the following steps:

1. **Click the Footing button.**
2. Change the **Navigation Bar** to the following:

   ![Navigation Bar](image)

3. Change the **Footing Module** to the following:

   ![Footing Module](image)
4. Click the Calculate Button.

The footing size and development length are inadequate.

Although there are a number of factors the first step is to change the size of the footing to provide sufficient bearing area, most of the time this will solve all of the other issues.

As you can see above the footing width and footing area required are shown at the bottom of the input screen. The required footing width is 4.22 ft. Therefore, change the input box labeled “Footing Width” from 3 feet to 4.25 feet and click the calculate button again.

The footing is now adequate.

Congratulations! You have finished the StruCalc 8.0 tutorial.
3. USING THE PROGRAM

3.1 HELP

QUICKHELP
StruCalc provides on-screen help by displaying information on the Help Toolbar pertaining to the input that has focus. The contents of each subject can be viewed by either scrolling through the QuickHelp or by clicking on the Help Toolbar. If the Help Toolbar is turned off the user can access QuickHelp by pressing the F1 key.

HELP FILE
If more thorough help is required than is provided by the QuickHelp, the entire help file can be accessed by pressing the F1 key.

MORE HELP
Additional help and a FAQ can be found on our website at www.strucalc.com. Please feel free to email any questions to techelp@strucalc.com free of charge or you can purchase phone technical support; see section 1.4 of this manual.

3.2 AUTOSIZE

The AutoSize feature is a fast and efficient way to size structural members. Furthermore, it allows an easy comparison of several different adequate members so that the most efficient member may be selected.

1. After the loading information has been entered into the appropriate input boxes, click the AutoSize button on the Navigation Toolbar on the left.

2. Select the type of material to AutoSize (solid sawn, glulams, etc.) from the pull down box on the Navigation Toolbar.

3. Select the species or manufacturer (Douglas Fir-Larch, Boise Cascade, etc.) to AutoSize from the pull down box on the Navigation Toolbar.

4. Select the grade of material to AutoSize. Several different grades may be sized at the same time by either holding down the Ctrl key and clicking on the desired selection or by clicking and dragging the cursor across the ones you would like to size.

5. Click the AutoSize button in the top toolbar or double click the grade you wish to AutoSize.

6. When the AutoSizing is done the adequate members will be displayed. If there are no adequate members for a particular grade, N/A will be displayed. Choose one member and click the Select and Return button. The program will return to the module with the member chosen.

Note: The AutoSize feature automatically calculates and displays several adequate member sizes for each grade chosen. Therefore, the calculation intensive modules, such as the Multi-span and the Multi-loaded, can cause the AutoSize process to take a long time.
3.3 CALCULATE

The Calculate button on the Analysis Toolbar starts the calculation process to determine if the member selected is adequate. After the required inputs are entered or if a new member size is selected, click the Calculate button. If the member is not adequate, adjust the input parameters or increase the member size and click the Calculate button again. Pressing the Enter key on the keyboard can also perform the calculation.

3.4 STRESS VALUES

The stress values for the species chosen can be viewed by clicking the Stress Values button on the Navigation toolbar. The grade loaded in the left navigation toolbar can be changed by double clicking the grade in the stress values list. Stress values can also be viewed and modified using the Materials Database Editor, see section 4 of the manual for information.

3.5 LOADING DIAGRAM

Loading diagrams are available for all of the beam modules. To view the loading diagram for a particular analysis, click the Loading Diagram button on the Navigation Toolbar. The reactions and the loads on the beam are displayed in the Navigation Toolbar. A small loading diagram printed on each printout, a larger loading diagram can be printed by clicking the Print button on the bottom toolbar while viewing the loading diagram. StruCalc can also be configured to automatically print the loading diagram with the standard print-out, see section 3.11 for printing options.

3.6 SHEAR, MOMENT, AND DEFLECTION DIAGRAM

Shear, moment, and deflection diagrams are available for all of the beam modules. To view the diagrams for a particular analysis, click the VMD Diagrams button on the Navigation Toolbar.

Once the diagrams are displayed the user may scroll across any one of the diagrams and the shear, moment, and deflection at that point will be displayed in the boxes at the left of the screen. The shear, moment and deflection at any location along the beam can also be viewed by simply inputting the desired point in the box that is labeled "Location" and then pressing the "Calculate" button. The controlling shear, moment, and deflection diagrams are the default diagrams, but the user can scroll through the drop down box at the left of the screen to select the desired load combinations and the corresponding diagrams will be displayed. The Shear/Moment/Deflection Diagram can be printed after viewing by clicking the Print button. StruCalc can also be configured to automatically print the Shear/Moment/Deflection Diagram with the standard print-out, see section 3.11 for printing options.

3.7 CALCULATOR

StruCalc has the ability to open the Windows calculator by clicking the Calculator button on the Analysis Toolbar. If a different calculator is desired, rename the desired calculator to Scalc.exe and put it in the StruCalc directory.
3.8 MEMBER NOTES
User added member notes may be added to the bottom of the printouts of each member by clicking the Member Notes button on the Navigation Toolbar. The member file must be opened to add user notes. Optional AutoSize member sizes can be copied directly to the notes from the AutoSize form, see section 3.2.

3.9 CHANGING THE BUILDING CODE
The default building code can be changed by clicking the Settings button on the Analysis Toolbar and then selecting the desired code. The default code is the code that will be used when starting a new project. The NDS version can be selected separately if a different version is desired, check with your local building department to determine if they will accept a newer version for design. StruCalc automatically defaults to the NDS specified by the chosen building code.

The code for a particular member can be changed in the navigation toolbar by clicking on the Member Codes tab and choosing the desired code.

The code for an entire project can be changed in the project manager. After opening the project manager, highlight the desired project, click on the project button in the top toolbar and choose Change project.

3.10 PRINTING

3.10.1 PRINTING MEMBERS
Printing member information can be accomplished in several different ways and in several different formats.

PRINTING FROM THE MODULE
The current analysis can be sent to the printer by clicking the Print Analysis button on the Analysis Toolbar. The print dialog box will appear and the print parameters can be set as desired. Click on the Print button to print the analysis.

PRINTING MULTIPLE MEMBERS
Printing multiple members can only be accomplished in the Project Manager. See section 3.15.2.

PRINTING TO SCREEN
The current analysis can be displayed on the screen by clicking the Preview Analysis button on the Navigation Toolbar. The preview shows the layout of the member information that will be sent to the printer.

PRINTING TO FILE
The current analysis can be printed to a text file by first opening a print preview by clicking the Print Preview button on the Navigation Toolbar. The preview can be saved in several different file formats by clicking the save button in the top toolbar.
3.10.2 CHANGING THE PRINTING SETTINGS

MARGINS
The default margins used for the printouts can be changed by adjusting the size of the margins for the left, right, top and bottom under the Printing tab of the Settings form. The margin widths are in inches and they are in addition to the required hard margins defined by the print area of the specific printer in use. The margins for a particular print out can be adjusted in the Navigation Toolbar on the left while viewing the print preview.

FONTS
The default font and font size can be changed under the Printing tab in the settings form. The font and font size for a particular print out can be changed in the navigation toolbar while viewing the print preview. Note that not all fonts will format correctly in StruCalc. We recommend that you not change the font from the default font.

STRU CALC PRINTER
The choose printer button opens the printer setup dialog box. The printer settings are unique to the printer in use and use the same printer drivers used by other Windows applications. If there is difficulty printing from StruCalc please check other Windows applications. If word processors, spreadsheet programs, and etc. are not printing, then the printing problem is not a result of the StruCalc program. In these cases please refer to the Windows manual.

3.11 CHANGING THE USER INFORMATION/ COMPANY LOGO
The user name, company name, address and company logo that appear on the printouts can be changed by clicking the Settings button on the Analysis Toolbar, and then choosing User from the tabs at the top of the form.

3.12 CHANGING THE USER OPTIONS/DEFAULT VALUES
The user options and default values can be changed by clicking the Settings button on the Analysis Toolbar and then choosing Options and Deflections from the tabs at the top of the form. All the modules in StruCalc use the design options/default values that are listed, therefore it is important that the preferred design options are selected and the correct default values are entered. See the pertinent part of the manual for specific information on individual options.

3.13 MODIFYING PROJECTS
StruCalc organizes members into projects, which are files containing all of the individual member designs. Most modification of projects and files is done using the Project Manager. Projects can be opened, printed, deleted, renamed, copied and code changed using the Project Manager. Although the Project Manager is a powerful tool it was not designed to replace the Windows File Manager, so its functions are limited.

3.13.1 SAVING MEMBERS
Members are saved in each individual module by clicking the Save button on the Analysis Toolbar. The location text box is used to identify the member within the project. If the project already exists and is open, the member will be added to the project file. If a project is not currently open you will be
prompted to open an existing project or to start a new project. A .pr8 extension is automatically added to the project name.

3.13.2 PROJECT MANAGER

The Project Manager is used to manipulate project files (files with .pr8 extensions). A project file contains all of the design members for a given project. Your current projects should be listed under the Projects heading at the left side of the project manager. Click on the project you want to view and the members in the project will appear to the right. The control buttons at the top of the Project Manager can be used to perform various functions depending on which list box has focus.

CREATING A NEW PROJECT

New projects can be created by clicking Project and then New Project in the Project Manager and entering a new project name in the Project dialog box or by specifying a new project when saving a member from an analysis. The new project becomes the default project for any subsequently saved members.

CHANGING THE DIRECTORY

You can change the directory by clicking Change Directory in the toolbar and browsing to the new directory and then choosing the project file you want to open.

RETRIEVING AN EXISTING MEMBER FROM A PROJECT

To open an existing member click Open in the Project Manager after the desired member in the member list box has been highlighted. The design analysis specified will be opened. Double clicking on the desired member name will also open the design analysis.

COPYING PROJECTS

To copy projects click Copy in the Project Manager after the project to be copied is highlighted in the project list box. The copy dialog box will appear. Members can also be copied by opening the desired member and renaming it before saving.

RENAMEING PROJECTS AND MEMBERS

To rename projects and members right click on the member or project and choose Rename. Type the new name into the input box.

DELETING PROJECTS AND MEMBERS

To delete a project or member rightclick and choose Delete while the desired project or member is highlighted in the Project or Member list box.

PRINTING MULTIPLE MEMBERS

Multiple members in a project can be printed from the Project Manager. Select the desired members to be printed on the right hand side and click Print Project under the project heading. Select the desired format and click Ok. Several members can be selected by holding down the Ctrl key and clicking on the members to be printed.

An entire project can be printed by clicking to highlight the desired project and then choosing Print Project from the project heading in the top toolbar.

After the print preview appears click the print button in the top toolbar.
PRINTING MULTIPLE MEMBERS TO FILE
Multiple members in a project can be printed to file by using the Project Manager. Follow the directions for printing Multiple Members above and instead of printing the file click the Save button in the top toolbar.

CHANGING THE BUILDING CODE
To change the building code for an entire project click the desired project on the left hand side of the project manager, then click the Change Code button Under Project in the top toolbar. Choose the desired code and NDS and click Ok. The code will be changed for all of the members in the project.

IMPORTING FILES FROM PREVIOUS VERSIONS OF STRUCALC
To import projects from previous versions of StruCalc open the Project Manager, click on Project in the top toolbar and choose Update Wizard. Choose Batch Processing to update multiple files at one time or choose Individual Project in order to update a single project. The new StruCalc 8.0 project will be created in the same folder as the previous project.

3.14 TOOLBARS

3.14.1 ANALYSIS TOOLBAR
The Analysis Toolbar is a useful way to open the different StruCalc modules and quickly perform the common program functions. When the mouse pointer is moved over a toolbar button, a description of that button’s function appears on the QuickHelp toolbar above.

The first eleven buttons on the left are used to quickly access StruCalc’s eleven design modules. To open a new module, simply click on the icon representing the desired module.

The next button is the Calculate Button, click this to run an analysis.

The last six buttons control the following functions: Save Analysis, Settings, Project Manager, Materials Database Editor, Calculator, and Print.

NAVIGATION TOOLBAR
The Navigation Toolbar at the left contains the information describing the type of member to be used for the design and the design parameters.

The material type, species/manufacturer, and grade are chosen from the three pull down boxes. See section 4.2.1 for commentary. The label below the grade drop down box indicates the material group being used for solid sawn lumber. The button on the left toggles between Wet and Dry use conditions. See section 4.2.10 for commentary.
The Navigation Toolbar contains information about the physical size of the member including number of laminations, width, depth, spacing for joist and rafters, and notch depth.

The live load and total load deflection criteria as well as the duration factor and camber adjustment factor inputs are Navigation Toolbar. All of these inputs are not relevant for every material type and design module; therefore they will not always appear on the Factor Toolbar.

3.14.2 QUICKHELP TOOLBAR
The QuickHelp Toolbar displays a quick description and help for a particular input box when you have given that input box focus by clicking on it or mousing over it. Click F3 to view the help summary if the information is longer than one line.

3.14.3 RESULTS TOOLBAR
The Results Toolbar displays adequacy information at the bottom of the screen after a calculation is performed. The color of the icon on the pop up toolbar displays a visual reminder of the design adequacy. The Adequacy Icon is green if the inputs are within bounds, yellow if they are questionable or an extra note is included on the output, and red if the inputs are out of bounds or the design is inadequate.

The controlling factor and percentage adequate or inadequate is shown in the main bar. When the results toolbar is popped up it also displays the other design factors and percentages adequate or inadequate as well as a green or red indicator coloring.
4. MATERIAL’S DATABASE EDITOR

4.1 Material’s Database Editor Overview

The Materials database editor is used to view, modify or add new material types to StruCalc as well as turn Species, grades and sizes on and off, and set the default grades that appear in the program.

4.1.1 MAIN FORM

![Diagram of the main form of the database editor]

4.1.2 TOOLBAR

The toolbar is relatively simple. The Add Wizard allows you add new species and grades. The add Size button allows the addition of new sizes. The Delete button allows you to delete materials. The Set Default button allows you to set the default materials to show up initially in the program. The Save Changes button is enabled when changes to a stress value need to be saved. The Video Tutorial button launches a video overview on how to use the database editor. Watching the video is the recommended method of learning the how to use the powerful Materials Database Editor.

Each individual button has two settings, enabled and disabled. You cannot click on a disabled button. The disabled buttons are always shown in grayscale and upon enabling they become full color. Clicking an enabled, full color toolbar button will execute that buttons function on the current selection. This will bring up new forms for review or actually perform the function on the list box.
4.1.3 LIST BOXES & MODULE SELECTIONS
There are four list boxes as shown in figure 4.1.1. These list boxes work in a left to right manner. This means as you make a selection in a list box the list box to its immediate right will load with the selections that your current selection supports.

The module selection buttons allows you to load the lists that correspond to the design modules in the program. There are three main types of design modules: beams, columns and joists/rafters. All changes made to each type in the Database Editor only affect the design modules of that design type. For example turning off a material for beams only turns it off in the beam modules and does not affect its appearance in the joist or column module.

4.1.4 CUSTOM VS. DEFAULT SELECTIONS
The Database Editor has a pre-set database of materials with species, grades, sizes etc. already added to the program. These pre-defined types are the default values in the Database Editor. These values can be modified or deleted by an end-user in the Database Editor. Extreme caution should be used prior to deleting or modifying any of the pre-defined values. These values have been taken from the appropriate nationally recognized agencies and realistically should not need to be modified.

Anything that an end-user adds to StruCalc using the Database Editor is added as a custom species, section, or grade. The highest level of this custom defined type till have – Custom appended to its name. If you have a selection appended with – Custom all selections linked at a lower level to this selection are also custom.

4.2 USING THE ADD WIZARD

The process for adding a new material, steel shape or steel grade is very similar. The Add Wizard will walk you through the steps required. See below for a step by step example of adding a new wood species.

4.2.1 ADDING A NEW WOOD SPECIES EXAMPLE

The following is the process for adding a new solid sawn species and grade to StruCalc.

1. Click the Add Wizard button in the top toolbar.
2. Choose Solid Sawn and click Next.
3. At this point you can either add a new name for your custom species or add a new grade to an existing species by choosing one from the drop down menu. For this example we will create a new species. Type in a name for your new wood species. For this example we used “Example Wood Species”.

![Image showing name input for a new species]

Click Next.

4. The next step is choosing which module type to add the new species too. For most cases it makes sense to add it too all the module types.

![Image showing module type selection]

Click Next.

5. Enter the Species self weight.

![Image showing self weight input]

Click Next.

6. Choose the size factors to be applied for the new species. For this example we will be using Western factors.

![Image showing size factor selection]

Click Next.
7. Enter a name for the first grade for the new species. For this example we will use #2. Enter the design values for each size category. If values for certain categories do not exist you can enter zeros for these values. All of the values for a specific size category must be filled in for the calculations to be valid.

![Table with design values](image)

Click Next.

8. You can now choose to add another grade or continue on. To add another custom grade click yes to the pop up message box.

Click No.

9. Congratulations you have added a new custom solid sawn species. Standard sizes are automatically added.

4.2.2 ADDING A NEW WOOD SIZE

Adding a new size to any solid sawn, glulam or manufactured lumber material is easy using the Add Size wizard.

ADDING A CUSTOM SIZE EXAMPLE

1. Click on the species or grade you want to add a custom size too by clicking on it.

2. Click the Add Size button in the top toolbar.

3. Enter the new width, depth and size factors.

4. Click the Add Button.

5. You can then add additional sizes or click Finish to exit and add the sizes.
4.3 SETTING THE DEFAULT SELECTION IN STRUCALC

Every wood species can have a default grade selection. Every steel section can have a default shape and also a default grade selected. Just highlight the desired default and click the Set Default button in the top toolbar.

The Database Editor is shipped with default selections for all types by StruCalc's engineers but these are just suggestions. Custom or non-custom types can all have defaults changed by the end user. When a default is set a light gray border appears around the selection signifying that it is the current default. When a new module is opened or a species changed in StruCalc this grade or default selection will be automatically selected. Removing the need for the end-user to select a species and then a grade each time.
5. DESIGN OVERVIEW

5.1 DESIGN PROCESS
Designing a structural member with StruCalc can be accomplished in two different ways, by either trial and error, or by using the AutoSize feature. The basic design process is as follows:

1. Determine all of the design parameters and code requirements, such as beam size and tributary widths and code required live loads.
2. Input all of the loading information.
3. Click the AutoSize tab in the navigation toolbar and have StruCalc size several adequate members or continue to step 4.
4. Choose a material, type, and grade.
5. Enter a trial size in the width and depth pull down boxes.
6. Click the Calculate button.
7. Inspect the results toolbar on the bottom to determine if the member is adequate.
8. If the member is adequate, attempt to reduce the size to optimize the material use.
   If the member is inadequate, modify the inputs until the member is adequate.

5.2 DESCRIPTION OF BASIC BEAM INPUTS

5.2.1 MATERIALS
The materials used for design are selected using the three pull down boxes at the top of the Navigation Toolbar on the left of the screen. The contents of the boxes change depending on the module and code chosen.

MATERIAL TYPE
The material type differentiates between solid sawn lumber, glulams, structural composite, steel, tube steel, flitch beam, and I-joists.

SPECIES/MANUFACTURER
The species box differentiates between the different species for solid sawn lumber and glulams, the structural composite/I-joist manufactures, and steel/tube steel shapes.

GRADE
The grade box differentiates between the grades of solid sawn lumber, the combination symbol of glulams, the specific steel/tube steel shapes, and the specific manufactured structural composite or I-joist.
5.2.2 MEMBER DESCRIPTION

LAMINATIONS
StruCalc allows the lamination of dimensional lumber and structural composite. The program includes the total area of the laminated members in all calculations.

A professional should design the connection of the laminated members. Care should be taken to insure that any unbalanced loading of the members is properly transferred through all laminations to avoid a shear failure. Consult the manufacturer for structural composite connection information.

WIDTH/DEPTH
The width and depth are entered using pull down list boxes. These boxes are loaded with the enabled sizes for the material chosen in the Database Editor. Note that the program uses the actual net sizes of the lumber not the nominal size (i.e. a 2x4 is entered as 1.5”x 3.5”). The program will automatically switch the depth increments to the correct size when changing from dimensional lumber to beams and stringers or posts and timbers. The width and depth can be overridden by typing over the value in the box. You can design a solid sawn member for flat use by typing in a width greater then the depth in the two size input boxes. StruCalc will automatically design the member flat and apply the correct flat use factor.

SPACING
The spacing of joists and rafters is chosen from the values in the pull down box on the section toolbar. These values can be overridden by manually typing in the desired spacing.

NOTCH DEPTH
StruCalc allows the notching of wood members for bearing over a support within the limits set by the NDS. See figure 5-1. For solid sawn lumber, the maximum allowable notch depth is d/4. For glulam beams the maximum allowable notch depth is not to exceed d/10. Also the 01 and new NDS reduce the allowable shear parallel to grain for all notched Glulams. Please note that these notches should be avoided whenever possible.

![Figure 5-1 Sample Notch Depth](image-url)
SPAN
The span length (L1, L2, and L3) is the plan view projected length measured from center to center of supports. The length shown in the short summary is the plan view length of all spans. The actual length is the total of the slope or pitch adjusted lengths of all portions of the beam (see section 4.2.7 for slope adjustment commentary).

Note: The output from StruCalc should not be used for a materials list without first accounting for end bearing length and conditions.

TRIBUTARY WIDTH
The tributary width (TW) is the loaded area bearing on the beam. This is usually half the joist or rafter span.

ROUND MEMBERS
Click the Round checkbox in the navigation toolbar in order to design using a round section. StruCalc will automatically calculate the equivalent square section and apply the form factor adjustment factor as required. Round member can also be specifically entered in the Materials Database Editor and can only be used as a round member.

FLAT USE
Click the Flat Use checkbox below the depth input box on the Navigation toolbar in order to design a solid sawn member in the flatwise direction. StruCalc will apply the flat use factor automatically.

5.2.3 DEFLECTION CRITERIA
The inputs for the live and total load deflection limits are in pull down boxes, which list the common values for roof and floor loading. The listed inputs can be overridden manually by inputting the desired value. The default values for floor beam loading are L/360 live load and L/240 dead load and for roof loading are L/240 live load and L/180 total load.

RECOMMENDED DEFLECTION CRITERIA
The default live and dead load deflection limits listed above are based on the code required minimum values and should be considered the minimum used for ideal conditions. Higher values should be used in situations where deflections are critical. This determination is highly subjective and requires professional judgment. For example, the code minimum values for floor joists may result in a “bouncy” feel for long spans, therefore the default values, in StruCalc, for floor joist loading are L/480 live load and L/360 dead load.

The allowable deflection is calculated based on the inputted deflection limits and the interior span length. The actual deflection ratio is printed next to the deflections on the output.

DEFLECTION LIMIT CALCULATIONS FOR INTERIOR SPANS
The maximum allowable live load deflection limit:

\[ TLD_{\text{max}} = \frac{L}{360} \text{(in/ft)} \]

The actual live load deflection ratio:
The actual total load deflection ratio:

\[
L_{LD_{ratio}} = \frac{L \cdot \left(\frac{12 \text{ in}}{\text{ft}}\right)}{L_{LD}}
\]

\[
T_{LD_{ratio}} = \frac{L \cdot \left(\frac{12 \text{ in}}{\text{ft}}\right)}{T_{LD}}
\]

**DEFLECTION LIMIT OPTIONS FOR CANTILEVERS**

The cantilever deflection criteria can be changed from 1x the span length to 2x the span length. Open the Settings form by clicking the **Settings** button on the Analysis Toolbar and then choose “the **Options** tab” from the tabs at the top of the form. To illustrate how the allowable live load deflection limit would be changed the new equation has been shown below:

\[
L_{LD_{max}} = \frac{2L \cdot \left(\frac{12 \text{ in}}{\text{ft}}\right)}{L_{LD_{criteria}}}
\]

**Note:** All of the beam modules in StruCalc will use the cantilever deflection option that is selected therefore it is important that the preferred option is chosen.

**5.2.4 DURATION FACTOR**

The inputs for the duration factor are in pull down boxes, which list the common code values for typical loading. The listed inputs can be overridden manually by inputting the desired value. The default values for each module are 1.15 for roof snow loads, 1.25 for roof non-snow loads, 1.33 for wind or seismic loads, and 1.0 for floor loading.

**CODE REQUIRED DURATION FACTOR VALUES**

Due to the elastic properties of wood, adjustments are made in the program to consider the duration factor in moment and bending calculations. NDS\textsuperscript{5,9} Appendix B specifies modification factors for loading of other than "Normal load duration," Normal load duration is defined in section B.1.1 as applying the maximum tabulated stresses for approximately ten years, either continuously or cumulatively. A duration factor corresponding to the load of shortest duration is to be applied to each load case. Dead and total load cases should be reviewed with the appropriate duration factor to ensure that the maximum stresses are allowed for in design. Common values of the duration factor (Cd) are shown below per NDS\textsuperscript{5,9}

<table>
<thead>
<tr>
<th>DURATION</th>
<th>TYPE OF LOAD</th>
<th>DURATION FACTOR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent</td>
<td>Dead load</td>
<td>0.90</td>
</tr>
<tr>
<td>Normal</td>
<td>Occupancy live load</td>
<td>1.00</td>
</tr>
<tr>
<td>Two months</td>
<td>Snow load</td>
<td>1.15</td>
</tr>
<tr>
<td>Seven Days</td>
<td>Construction/Roof load</td>
<td>1.25</td>
</tr>
<tr>
<td>Ten Minutes</td>
<td>Wind/Seismic load</td>
<td>1.60</td>
</tr>
<tr>
<td>One Second</td>
<td>Impact load</td>
<td>2.00</td>
</tr>
</tbody>
</table>

**CONTROLLING DURATION FACTOR DETERMINATION**

The controlling duration factor for most roof and floor loading is the inputted live load value. In some cases where the dead load contributes the majority of the load, the permanent duration factor will control. StruCalc checks the inputted value against the permanent duration factor to determine the controlling duration factor. An example of this process is illustrated below.
If \[
\frac{\text{Dead Load}}{0.90} > \frac{\text{Total Load}}{\text{Inputted Cd}}
\]

Then the controlling duration factor (Cd) will be 0.90 and the controlling load will be the inputted dead load.

If \[
\frac{\text{Dead Load}}{0.90} < \frac{\text{Total Load}}{\text{Inputted Cd}}
\]

Then the controlling duration factor (Cd) will be the inputted duration factor and the controlling load will be the total load.

If the inputted duration factor is equal to 0.90 then the controlling load will be the total load.

### 5.2.5 SNOW/NON-SNOW

The roof beam and roof rafter modules allow the user to choose either a snow or non-snow region. For snow regions the live load is determined by the local building official and is entered by the user. For non-snow regions the program will determine the code required minimum live loads based on the method chosen. The alternate non-snow method allows the user to enter a live load other than what is determined by the code.

The different building codes use method 1, method 2, or both methods. See Table 5-1. The live loads for method one are based on the roof pitch and area loaded. The live loads for method two are also based on roof pitch and area loaded but they are determined as a percentage of the base load.

<table>
<thead>
<tr>
<th>ROOF SLOPE</th>
<th>METHOD 1</th>
<th>METHOD 2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>TRIBUTARY LOADED AREA IN SQUARE FEET FOR ANY STRUCTURAL MEMBER</td>
<td>UNIFORM LOAD</td>
</tr>
<tr>
<td></td>
<td>0 TO 200</td>
<td>201 TO 600</td>
</tr>
<tr>
<td>1. Flat or rise less than 4 in. per ft.</td>
<td>20</td>
<td>16</td>
</tr>
<tr>
<td>2. Rise 4 in. per ft. to less than 12 in. per ft.</td>
<td>16</td>
<td>14</td>
</tr>
<tr>
<td>3. Rise 12 in. per ft. and greater</td>
<td>12</td>
<td>12</td>
</tr>
</tbody>
</table>

**TABLE 5-1 MINIMUM ROOF LIVE LOADS**

Method 2 only applies for members supporting roof areas greater than 150 square feet.

Roof load reduction percentage (when applicable) based on roof loaded area bearing on beam:

\[ R1 = r \cdot (RLA - 150) \]

Where RLA is the roof loaded area in square feet and r is the rate of reduction from the minimum roof live loads table.

### 5.2.6 UNBRACED LENGTH

Determination and understanding of the unbraced length is a very important part of member design. The adequacy of a particular beam can be dramatically affected by the lateral support provided. The
unbraced length for simple span beam modules refers to the top of the beam, which in all simply supported gravity loaded cases is in compression.

Multi-Span beams and cantilevers, due to their loading geometry, generally have moment reversals, which cause the bottom portion of the beam to be in compression. In StruCalc the unbraced length of the bottom of multi-span beams and cantilever beams defaults to the distance between supports and the distance from the cantilever end to the support; the reason being that unless bracing is specifically called out on plans the bottom of beams are usually always unbraced. The unbraced length of the bottom of multi-span joists also defaults to the distance between supports unless some sort of sheathing (i.e. plywood, gwb, etc.) is applied to the joist’s bottom. If sheathing is applied to the bottom of multi-span joists, the box labeled “Bracing Applied to Bottom of Joists” (in the floor joist module) needs to be checked to provide an accurate joist design.

For wood design, the NDS allows two different methods for the determination of lateral stability in beams. StruCalc uses the empirical method of reducing the allowable bending stress for large unbraced lengths. The “rule of thumb” method is also allowed by the NDS and can be used by setting the unbraced length to zero and hand checking the stability requirements.

For steel beam design, the unbraced length is much more critical than it is in wood design since steel members generally have very slender portions in compression. As a result, the proper determination of the unbraced length is very critical for insuring a safe and practical design.

Engineering judgment is always required to determine how and when a beam is braced and what is considered fully braced.

5.2.7 SLOPE ADJUSTMENTS

The program adjusts the live load, dead load, and beam length to account for roof pitch and beam slope. As the roof pitch increases, a greater roof surface area is bearing on the supporting members thus increasing the dead loads. In addition, when beams are sloped (i.e., a difference in elevation between beam ends) the live load, dead load, and beam length are adjusted.

ROOF BEAM-ROOF PITCH ADJUSTMENTS

Live loads are not adjusted for roof pitch since the tributary width (TW) of the load does not change. Dead loads are increased due to roof pitch since an increase in pitch increases the rafter length for a given rafter tributary width. See Figure 5-2.

\[ RA = \tan^{-1}\left( \frac{RP}{12} \right) \]

\[ TWadj = \frac{TW}{\cos(RA)} \]

\[ wD \text{ (adjusted for pitch)} = TWadj \cdot wD \]

Where:
RA is the roof angle.
RP is the roof pitch in inches per foot.
TWadj is the adjusted rafter tributary width.
TW is the rafter tributary width.

Example: Rafter tributary width of 10 feet, roof pitch 6:12, roof dead load 20 psf.
RA = \tan^{-1}\left(\frac{6}{12}\right) = 26.56^\circ

TWadj = \frac{10\text{ft}}{\cos 26.56^\circ} = 11.2 \text{ ft}

wD (adjusted for pitch) = 11.2 \text{ ft} \cdot 20 \text{ psf} = 223 \text{ plf}

**FIGURE 5-2 SLOPE ADJUSTED BEAM LOADING**

**ROOF BEAM-SLOPE AND ROOF PITCH ADJUSTMENTS**
The tributary dead loads are calculated to account for the roof pitch (see calculations above). However, no adjustment is made for the live loads. As the slope of the beam increases, the actual length of the beam increases but the total vertical load projected on the beam does not increase. The same vertical load consequently is spread over a longer beam length therefore moment and deflection calculations for beams with a high pitch can be significantly affected.
Therefore, to avoid an overly conservative design and to calculate the beam live and dead loads more accurately, an adjustment is made to consider only the components of the loads that are acting perpendicular to the centroidal axis of the beam. See Figure 5-3.

The following commentary and equations below illustrate how the loads are adjusted for a beam that is sloped.

$BA = \tan^{-1}\left( \frac{EL}{L} \right)$

$L_{adj} = \frac{L}{\cos(BA)}$

Where:
BA is the beam angle.
L is the beam span (horizontal projected length).
EL is the beam end elevation difference.
Ladj is the slope adjusted beam length.

**Figure 5-3 Adjusted Loads on Sloped Beam**

Live load adjustments:

$BLL' = TW \cdot RLL$
\[ TLL = BLL' \cdot L \]

\[ BLL_{adj} = \frac{TLL}{L_{adj}} \]

\[ wL = BLL_{adj} \cdot \cos(BA) \]

Where:
- \( BLL' \) is the beam live load acting over span \( L \).
- \( TW \) is the rafter tributary width.
- \( RLL \) is the roof live load.
- \( TLL \) is the total live load bearing on the beam.
- \( BLL_{adj} \) is the total live load spread over \( L_{adj} \) (slope adjusted beam).
- \( wL \) is the linear live load acting perpendicular to the beam which is used in beam design calculations.

Dead load adjustments:

\[ BDL' = wD(\text{adjusted for pitch}) \text{ See section 5.2.7.} \]

\[ TDL = BDL' \cdot L \]

\[ BDL_{adj} = \frac{TDL}{L_{adj}} \]

\[ wD = BDL_{adj} \cdot \cos(BA) \]

Where:
- \( BDL' \) is the beam dead load acting over span \( L \).
- \( TDL \) is the total dead load bearing on the beam.
- \( BDL_{adj} \) is the total dead load spread over \( L_{adj} \) (slope adjusted beam).
- \( wD \) is the actual linear dead load acting perpendicular to the beam which is used in beam design calculations.

**ROOF RAFTER-PITCH ADJUSTMENTS**

The adjustments for a roof rafter are similar to the adjustments for a roof beam with an end elevation difference. The dead load is increased since the actual length of the rafter is longer while the vertical component of the live load is spread over a longer span. See figure 5-4.

The adjusted rafter length is calculated as follows:

\[ L_{adj} = \frac{L}{\cos(\text{RA})} \]

\[ \text{Leave}_{adj} = \frac{\text{Leave}}{\cos(\text{RA})} \]

Where:
- \( \text{RA} \) (roof angle) = \( \arctan \left( \frac{\text{RP}}{12} \right) \).
The loads on a rafter are adjusted for roof pitch as shown:

\[ w_{L\text{ (adjusted for pitch)}} = w_L \cdot \left( \frac{L}{L_{adj}} \right) \cdot \cos(RA) \]

\[ w_{D\text{ (adjusted for pitch)}} = w_D \cdot \cos(RA) \]

### 5.2.8 LOADING

The most critical part of the design process is to properly assess all of the physical parameters involved in a member design. This includes such parameters as the length and size of a member as well as the end use and loading conditions. The loading of a member requires professional judgment and knowledge of not only the actual loading conditions but also the code requirements of the local building department.

### LOAD COMBINATIONS

StruCalc does not check the different load combinations required by code (except for column design). It is the responsibility of the user to determine which combinations will control the design. The user should check live, dead, and wind/seismic loads.

### DEAD LOADS

Dead loads are permanent loads due to the actual physical weight of a structure. There are several sources that can be used to determine the dead load of a design. Care should be taken to investigate the actual dead loads since material weights can vary considerably. StruCalc automatically includes
LIVE LOADS

Live loads are loads imposed on a structural member, which depend upon the use of the area supported by the beam or column. The local building department usually designates the minimum live load but the actual loading conditions should be investigated and accounted for in the design.

5.2.9 FLOOR LIVE LOAD REDUCTION

Many of the building codes used in StruCalc allow reductions in the required unit live load for certain occupancy types and large areas of loading. The reasoning for this reduction is that large areas typically are not loaded to capacity over their entire area simultaneously. It is the responsibility of the designer to apply this reduction only when allowed by the building code in effect and when its application will not result in possible overstress of the structural member being designed.

REDUCTIONS FOR 94 AND 97 UBC, 2000, 2003 AND 2006 IBC

Floor live load reductions are allowed for any member supporting an area greater than 150 square feet, except in places of public assembly and for live loads greater than 100 psf.

Floor load reduction percentage is based on the loaded floor area supported by the beam being designed.

Reduction for 94 and 97 UBC:

\[ R1 = 0.08 \cdot (\text{FLA} - 150) \quad \text{UBC}^{2.8} \left(1607.5\right) \]

Where:

- FLA is the floor loaded area.

The reduction R is limited to R2 based on the dead load to live load ratio, which is determined by the formula below:

\[ R2 = 23.1 \cdot \left(1 + \frac{\text{DL}}{\text{LLavg}}\right) \quad \text{UBC}^{2.8} \left(1607.5\right) \]

Where:

- LLavg is the average floor live load.

R is also limited to R3, which is the maximum reduction allowed by code. Note that R3 is 0.40 for beams with floor loads from one level and 0.60 for all other cases.

R is the controlling reduction factor, which is the minimum of R1, R2 and R3.

REDUCTIONS FOR 94, 97 AND 99 SBC

Floor live load reductions are allowed for any member supporting an area greater than 150 square feet, except in places of public assembly and for live loads greater than 100 psf.

Floor load reduction percentage is based on the loaded floor area supported by the beam being designed.

Reduction for 94, 97, and 99 SBC:

\[ R1 = 0.08 \cdot \text{FLASBC}^{10,14,15} \left(1604.2\right) \]

Where:
FLA is the floor loaded area.

The reduction $R$ is limited to $R_2$ based on the dead load to live load ratio, which is determined by the formula below:

$$R_2 = 23.1 \left(1 + \frac{DL}{LL_{avg}}\right)$$

Where:

LL$_{avg}$ is the average floor live load.

$R$ is also limited to $R_3$, which is the maximum reduction allowed by code. Note that $R_3$ is 0.40 for all loading cases.

$R$ is the controlling reduction factor, which is the minimum of $R_1$, $R_2$ and $R_3$.

**REDUCTIONS FOR 93, 96 AND 99 BOCA**

Floor live load reductions are allowed for members with influence areas of 400 square feet or more and live loads less than 100 psf.

Reduction for 93, 96, and 99 BOCA:

$$R_1 = 0.25 + \frac{15}{\sqrt{Ai}}$$

Where:

Ai is the influence area in square feet.

The influence area is two times the floor loaded area for beams ($Ai = 2 \times FLA$).

R2 is the maximum reduction allowed by code for live loads of 100 psf or more. R2 is 0.20.

R3 is the maximum reduction allowed by code for live loads of 100 psf or less. Note that R3 is 0.50 for beams with floor loads from one level and 0.60 for all other cases.

**5.2.10 CAMBER AMPLIFICATION FACTOR**

The amplification factor for dead load deflection in glulam beams is used to determine the required camber ($C$) to counteract long term dead load creep (plastic deformation). In most cases, a factor of 1.5 is used.
6. DESIGN CALCULATIONS

6.1 GENERAL DESIGN CALCULATIONS
StruCalc uses the moment distribution method to calculate the moment, shear and deflection along the beam length. Each span of the member is broken up into one hundred sections and the moment, shear and deflection are calculated at each section. The program uses the critical shear, moment and deflection to design the structural member.

6.2 WOOD DESIGN

6.2.1 MATERIAL PROPERTIES
The program uses the following properties specified by the NDS\textsuperscript{5,9} for solid sawn and glulam members. Structural composite lumber and I-joists use the manufacturer's published data.

- \( F_b \) = Tabulated bending stress, in psi.
- \( F_{b\_cpr} \) = Tabulated bending stress for the compression face in tension of glulam, in psi.
- \( F_v \) = Tabulated value of horizontal shear stress, in psi.
- \( F_{c\_perp} \) = Tabulated compression stress perpendicular to the grain, in psi.
- \( E \) = Tabulated modulus of elasticity, in psi.
- \( E_{\_min} \) = Tabulated adjusted modulus of elasticity, in psi.
- \( E_x \) = Tabulated modulus of elasticity about the X-axis for glulams, in psi.
- \( E_y \) = Tabulated modulus of elasticity about the Y-axis for glulams, in psi.

6.2.2 ADJUSTMENT FACTORS
The tabulated allowable stress values are modified by several adjustment factors specified in the NDS\textsuperscript{5,9} before they are used to calculate available section properties for the member. These adjustments include the duration factor, size factor, volume factor, stability factor, repetitive use factor, incising factor, and the wet use factor.

For the duration factor adjustment commentary see section 5.2.4.

SIZE FACTOR ADJUSTMENT
The NDS\textsuperscript{5,9} specifies a modification factor for the tabulated bending stress values for dimensional lumber and timbers exceeding 12 inches in depth per the NDS\textsuperscript{5,9} (5.3.2). Structural composite members are also adjusted for size and each manufacturer specifies a denominator for the size factor adjustment equation.

See table 4A in the NDS\textsuperscript{5,9} supplement for a listing of the size factors for dimensional lumber. The size factors for dimensional lumber are included in the Materials Database.

Bending stress adjustment for solid sawn timbers or glulams with depths greater than 12 inches:
Bending stress adjustment for structural composite lumber with \( n \) specified by the manufacturer:

\[
C_f = \left( \frac{12}{d} \right)^{\frac{1}{n}}
\]

**VOLUME FACTOR ADJUSTMENT**

The NDS\(^{5,9}\) specifies an adjustment to the maximum bending stress for glulams. The volume factor is not applied simultaneously with the beam stability factor, the lesser of these two adjustments is used.

The volume factor adjustment for glulams:

\[
C_v = K_l \cdot \left( \frac{21}{L} \right) \left( \frac{12}{d} \right) \left( \frac{5.125}{w} \right)
\]

Where:

\( x = 20 \) for southern pine.

\( x = 10 \) for all other species.

\( K_l \) = the loading condition coefficient which is conservatively taken as 1.0.

For the above equation, \( L \) is defined as the distance between points of zero moment. StruCalc conservatively uses the span length.

**STABILITY FACTOR ADJUSTMENT**

The NDS\(^{5,9}\) states that when the depth of a bending member exceeds its width lateral support may be required. If the unbraced length of the beam is greater than zero then the stability factor is calculated and applied to the bending stress. For glulam beams the stability factor is not applied simultaneously with the volume factor, the lesser of these two adjustments is used.

The stability factor is calculated as follows:

\[
C_l = \frac{1 + \left( \frac{F_{be}^{*}/F_{b}^{*}}{19} \right)}{\left[ 1 + \left( \frac{F_{be}^{*}/F_{b}^{*}}{19} \right) \right]^{2} \left( \frac{F_{be}^{*}/F_{b}^{*}}{0.95} \right)}
\]

Where:

\( F_{b}^{*} \) = Tabulated bending design value multiplied by all applicable adjustment factors except \( C_v \) and \( C_l \).

\( F_{be} = \frac{K_{be} \cdot E'}{R_{b}^2} \) use \( E' \) for Structurally glued laminated timber beams. (01 and older NDS)

\( K_{be} = 0.438 \) for visually graded lumber.

\( K_{be} = 0.609 \) for Structurally glued laminated timber beams.
F_{be} = \frac{1.2 \cdot E - \text{Min}'}{Rb^2} \quad \text{use } E_y\text{'min' for Structurally glued laminated timber beams. (05 NDS)}

Slenderness ratio:

\[ R_b = \left( \frac{L_e \cdot d}{w^2} \right)^{\frac{1}{2}} \]

Rb cannot exceed 50.

The effective unbraced length is calculated as follows:

When \( \frac{L_u}{d} < 7 \):

\[ L_e = 2.06 \cdot L_u \left( \frac{12 \text{in}}{\text{ft}} \right) \]

When \( 14.3 \geq \frac{L_u}{d} \geq 7 \):

\[ L_e = 1.63 \cdot L_u \left( \frac{12 \text{in}}{\text{ft}} \right) + 3 \cdot d \]

When \( \frac{L_u}{d} > 14.3 \):

\[ L_e = 1.84 \cdot L_u \left( \frac{12 \text{in}}{\text{ft}} \right) \]

**REPETITIVE MEMBER FACTOR**

The bending stress is adjusted by the repetitive member factor (Cr) when three or more dimensional members are in contact or are spaced not more than 24 inches on center. The repetitive member factor is equal to 1.15. This factor is automatically applied for laminated members, joists, and rafters that meet this requirement. It can also be manually applied in the multi-loaded beam module in order to correctly model an individual joist or rafter.

**INCISING FACTOR**

The incising factor is new for the NDS\textsuperscript{5,9,20} and it is found in the 1997 and 2001 edition. It is used to reduce the tabulated design values for solid sawn lumber when the member is incised to increase penetration of preservatives. The incising factor should be applied to all designs of solid sawn lumber that will be incised.

Note: The wet use factor should be applied along with the incising factor.

<table>
<thead>
<tr>
<th>97 NDS</th>
<th>01/05 NDS</th>
<th>DESIGN VALUE</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.85</td>
<td>0.80</td>
<td>F_b</td>
<td>Bending stress adjustment</td>
</tr>
<tr>
<td>0.85</td>
<td>0.80</td>
<td>F_c</td>
<td>Compression stress parallel to grain adjustment</td>
</tr>
<tr>
<td>0.95</td>
<td>0.95</td>
<td>E</td>
<td>Modulus of Elasticity adjustment</td>
</tr>
</tbody>
</table>

**WET USE FACTOR**

The program adjusts the member properties for moist service conditions as specified in the NDS\textsuperscript{5,9}. Wet service conditions are warranted when the moisture content of solid sawn lumber exceeds 19%
or 16% for Glulams. Structural composite members should not be used in wet service conditions unless specified by the manufacturer. To apply the wet use factor click the wet/dry button at the top left corner of the upper toolbar.

### WET USE (Cm) ADJUSTMENT FACTOR FOR SOLID SAWN LUMBER

<table>
<thead>
<tr>
<th>FACTOR</th>
<th>DESIGN VALUE</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.85</td>
<td>Fb</td>
<td>Bending stress adjustment if Fb &gt;1150 psi</td>
</tr>
<tr>
<td>1.00</td>
<td>Fb</td>
<td>Bending stress adjustment if Fb &lt;1150 psi</td>
</tr>
<tr>
<td>0.97</td>
<td>Fv</td>
<td>Horizontal shear stress adjustment</td>
</tr>
<tr>
<td>0.67</td>
<td>Fc_perp</td>
<td>Compression stress perpendicular to grain</td>
</tr>
<tr>
<td>0.90</td>
<td>E</td>
<td>Modulus of Elasticity adjustment</td>
</tr>
</tbody>
</table>

### WET USE (Cm) ADJUSTMENT FACTOR FOR GLULAMS

<table>
<thead>
<tr>
<th>FACTOR</th>
<th>DESIGN VALUE</th>
<th>DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.80</td>
<td>Fb</td>
<td>Bending stress adjustment</td>
</tr>
<tr>
<td>0.875</td>
<td>Fv</td>
<td>Horizontal shear stress adjustment</td>
</tr>
<tr>
<td>0.53</td>
<td>Fc_perp</td>
<td>Compression stress perpendicular to grain</td>
</tr>
<tr>
<td>0.833</td>
<td>E</td>
<td>Modulus of Elasticity adjustment</td>
</tr>
</tbody>
</table>

### 6.2.3 ADJUSTED ALLOWABLE STRESS VALUES

#### ADJUSTED BENDING STRESS – Fb’

The adjusted allowable bending stress for solid sawn and structural composite members:

\[
F_{b'} = F_b \cdot C_d \cdot C_m \cdot C_l \cdot C_f \cdot C_r \cdot C_i
\]

The adjusted allowable bending stress for glulams with only the smaller of Cv or Cl applied:

\[
F_{b'} = F_b \cdot C_d \cdot C_m \cdot C_l \cdot C_f \cdot C_v \cdot C_r \cdot C_i
\]

#### ADJUSTED SHEAR STRESS – Fv’

The adjusted allowable shear stress for solid sawn, glulams and structural composite lumber:

\[
F_{v'} = F_v \cdot C_d \cdot C_m \cdot C_i
\]

#### ADJUSTED COMPRESSION PARALLEL TO GRAIN – Fc’

The adjusted allowable compression parallel to grain for solid sawn lumber:

\[
F_{c'} = F_c \cdot C_d \cdot C_m \cdot C_f \cdot C_i
\]

The adjusted allowable compression parallel to grain for glulams:

\[
F_{c'} = F_c \cdot C_d \cdot C_m \cdot C_f
\]

#### ADJUSTED COMPRESSION PERPENDICULAR TO GRAIN – Fc_perp’

The adjusted allowable compression perpendicular to grain for solid sawn lumber, glulams and structural composite lumber:

\[
F_{c_perp'} = F_{c_perp} \cdot C_m \cdot C_i
\]
ADJUSTED MODULUS OF ELASTICITY– $E'$
The adjusted allowable modulus of elasticity for solid sawn lumber, glulams and structural composite lumber:

$$E' = E \cdot C_m \cdot C_i$$

6.2.4 WOOD SECTION PROPERTIES
To determine adequacy, the program compares the area, section modulus, and moment of inertia of the member with the required area, section modulus, and moment of inertia.

AREA
StruCalc adjusts the beam area for notches when applicable. The area is calculated using the following equations:

91 and 97 NDS:
$$A = w \cdot (# \ of \ Laminations) \cdot (d – nd) \cdot \frac{(d – nd)}{d}$$

Note that this is a transformation of the following equation:

$$V = \left( \frac{2w \cdot d \cdot Fv'}{3} \right) \left( d – nd \right)$$. NDS\(^9\) (3.44)

01/05 NDS:
$$A = w \cdot (# of Laminations) \cdot \left( \frac{(d – nd)}{d} \right)^2$$

Note that this is a transformation of the following equation:

$$V = \left( \frac{2w \cdot (d – nd) \cdot Fv'}{3} \right) \left( d – nd \right)^2$$. NDS\(^{20}\) (3.43)

StruCalc will also automatically ignore loads within a distance equal to the depth of the member from the member end for shear calculations based on NDS section 3.42. This can greatly affect the size of short heavily loaded members but only applies if the load is at the top of the beam and the support is on the bottom. You can turn off this feature in the Settings under the options tab.

SECTION MODULUS
The section modulus of a wood member:

$$S = \frac{w \cdot (# \ of \ Laminations) \cdot d^2}{6}$$

MOMENT OF INERTIA
The moment of inertia of a wood member:

$$I = \frac{w \cdot (# \ of \ Laminations) \cdot d^3}{12}$$
6.2.5 REQUIRED WOOD SECTION PROPERTIES

AREA REQUIRED

The area required is based on the maximum shear in the beam and the allowable shear stress of the member:

\[ A_{\text{req}} = \frac{15 \cdot V}{F_v'} \]

SECTION MODULUS REQUIRED

The section modulus required is based on the critical moment in the beam and the allowable bending stress of member:

\[ S_{\text{req}} = \frac{M \cdot \left( \frac{12 \text{in}}{\text{ft}} \right)}{F_b'} \]

MOMENT OF INERTIA REQUIRED

The moment of inertia required is based upon the allowable live and total load deflection limits inputted. The equation used is a derivation of the ratio of the moment of inertia, moment of inertia required, deflection and deflection limits.

\[ I_{\text{req}} = \frac{L \cdot \left( \frac{12 \text{in}}{\text{ft}} \right)}{\text{Controlling deflection} \cdot \text{Deflection Limit}} \]

6.2.6 ADEQUACY

For wood members the program compares the moment of inertia, section modulus, and area of the member with the required moment of inertia, section modulus and area to determine adequacy. The percent adequacy and the controlling factor are printed on the adequacy bar and on the printouts. The controlling factor is the criteria with smallest percentage of adequacy.

6.3 STEEL DESIGN

The steel design calculations and steel properties are taken from the 13th edition of the Manual of Steel Construction Allowable Stress Design, ASD. The 9th edition calculations are also available.

We have chosen not to reproduce the 13th edition calculations here. They can be found in the Manual of Steel Construction 13th Edition which can be downloaded for free at www.aisc.org.

Below are the steel calculations for the 9th edition ASD.

6.3.1 MATERIAL PROPERTIES

TABULATED PROPERTIES

The tabulated properties are loaded into the program for each steel shape as required for calculations. Some of the following are for tube steel members only.

\[ d = \text{Tabulated depth, in inches.} \]

\[ tw = \text{Tabulated web thickness, in inches.} \]

\[ bf = \text{Tabulated width of flange, in inches.} \]
tf = Tabulated thickness of flange, in inches.

k = Tabulated distance from the outside edge of the flange to the toe of the fillet, in inches.

Ix, Iy = Tabulated moment of inertia about the X-axis and Y-axis, in inches$^4$.

Sx, Sy = Tabulated section modulus about the X-axis and Y-axis, in inches$^3$.

rt = Tabulated radius of gyration of the flange and one third of the web, in inches.

Wt = Tabulated weight of the beam in pounds per linear foot.

A = Tabulated area of the member, in inches$^2$.

6.3.2 DESIGN FOR BENDING

The moment capacity of a steel beam is based upon the physical shape of the member and the lateral support of the beam.

COMPACT SECTION CRITERIA

Steel shapes are classified by the flange and web buckling ratio into compact and non-compact sections. See ASD\textsuperscript{4} (Table B5.1).

The flange buckling ratio is:

$$\text{FBR} = \frac{bf}{2 \cdot tf} \text{ for wide flange members.}$$

$$\text{FBR} = \frac{\text{dia}}{t} \text{ for round tube steel members.}$$

$$\text{FBR} = \frac{dx}{t} \text{ for square and rectangular tube steel members.}$$

$$\text{FBR} = \frac{d}{t} \text{ for steel angles.}$$

The maximum allowable flange buckling ratio for a compact section is:

$$\text{AFBR} = \frac{65}{\sqrt{F_y}} \text{ for wide flange and angle (eq 5-1a) steel members.}$$

$$\text{AFBR} = \frac{76}{\sqrt{F_y}} \text{ for angle steel members eq 5-1b.}$$

$$\text{AFBR} = \frac{3300}{F_y} \text{ for round tube steel members.}$$

$$\text{AFBR} = \frac{190}{\sqrt{F_y}} \text{ for square and rectangular tube steel members.}$$
The web buckling ratio is:

\[ \text{WBR} = \frac{d}{tw} \]

The maximum allowable web buckling ratio for a compact section is:

\[ \text{AWBR} = \frac{640}{\sqrt{F_y}} \]

**UNBRACED LENGTH**

Due to standard steel nomenclature the unbraced length for steel design is defined as Lb not Lu. The limiting unbraced lengths (Lc) and (Lu) are calculated as shown below.

The limiting unbraced length for compact "I" shaped members and channels for \( F_b = 0.66 \cdot F_y \) is taken as the smaller of the following two equations:

\[ Lc = \frac{76 \cdot \text{bf}}{(12 \text{in/ft})^{\frac{F_y}{12}}} \text{ ASD}^4 (F1-2) \]

Or:

\[ Lc = \frac{20,000}{(12 \text{in/ft}) \left( \frac{d}{Af} \right) \cdot F_y} \text{ ASD}^4 (F1-2) \]

For tube steel:

\[ Lc = \frac{1200 \cdot \left( \frac{dy}{F_y} \right)}{12 \text{in/ft}} \text{ Minimum for simplicity.} \]

The limiting unbraced length for "I" and "C" shaped members for \( F_b = 0.60 \cdot F_y \):

\[ Lu = \frac{20,000}{(12 \text{in/ft}) \left( \frac{d}{Af} \right) \cdot F_y} \]

**ALLOWABLE BENDING STRESS**

The allowable bending stress for "I" shaped compact members with an unbraced length (Lb) less than the limiting unbraced length (Lc):

\[ F_b = 0.66 \cdot F_y \text{ ASD}^4 (F1-1) \]

The allowable bending stress for "I" shaped members with non-compact flanges, \( F_y \) less than 65 ksi, and an unbraced length (Lb) less than the limiting unbraced length (Lc):

\[ F_b = F_y \left( 0.79 - 0.002 \cdot \frac{bf}{2 \cdot tf} \cdot \sqrt{F_y} \right) \text{ ASD}^4 (F1-3) \]
The allowable bending stress for non-compact members not included above with an unbraced length (Lb) less than the limiting unbraced length (Lu):

\[ F_b = 0.60 \cdot F_y \cdot ASD^4 \quad (F1-5) \]

The allowable bending stress for compact or non-compact members with an unbraced length (Lb) greater than the limiting unbraced length (Lu) is calculated as follows.

When the unbraced length (Lb) is less than the elastic limit for equation F1-6 (EL1-6) then the allowable bending stress (Fb) is equal to the larger of equations F1-6 or F1-8. When the unbraced length (Lb) is greater than the elastic limit for equation F1-6 then the allowable bending stress (Fb) is equal to the larger of equations F1-7 or F1-8. The allowable bending stress (Fb) for “C” shaped members is equal to equation F1-8.

The elastic limit for equation F1-6 is:

\[ EL_{1-6} = rt \cdot \sqrt{\frac{510,000 \cdot C_b}{F_y} \cdot \frac{1}{(12\text{in/ft})}} \]

Equation F1-6:

\[ F_{1-6} = \frac{2}{3} \left( \frac{F_y \cdot \left( \frac{L_b \cdot (12\text{in/ft})}{rt} \right)^2}{1,530,000 \cdot C_b} \right) \leq 0.60 \cdot F_y \]

Equation F1-7:

\[ F_{1-7} = \frac{170,000 \cdot C_b}{\left( \frac{L_b \cdot (12\text{in/ft})}{rt} \right)^2} \leq 0.60 \cdot F_y \]

Equation F1-8:

\[ F_{1-8} = \frac{12,000 \cdot C_b}{L_b \cdot (12\text{in/ft}) \cdot \left( \frac{d}{A_f} \right)} \leq 0.60 \cdot F_y \]

The bending coefficient (Cb) is a measure of the moment gradient for the section of the beam in question. It is conservative to take Cb as 1.0. You can change the Cb value used in design using the drop down box in the section toolbar.

The allowable stress for compact tube steel members with an unbraced length Lb less than Lc is:

\[ F_b = 0.66 \cdot F_y \]
For non-compact tube steel members or $L_b$ greater than $L_c$:

$$F_b = 0.6 \cdot F_y$$

See the manual of Steel Construction supplement: Specification for Allowable Stress Design of Single-Angle members for formula and explanation of single angle design for bending.

**NOMINAL MOMENT CAPACITY**

The moment capacity of the member is:

$$M_n = F_b \cdot S_x$$

**6.3.3 DESIGN FOR SHEAR**

**ALLOWABLE SHEAR STRESS**

The allowable shear stress is limited by the web height to thickness ratio below:

$$\frac{h}{t_w} \text{ limit} = \frac{380}{\sqrt{F_y}}$$

The web height to thickness ratio:

$$\frac{h}{t_w} = \frac{d - 2k}{t_w}$$

For tube steel substitute $d_x$ for $d$, $t$ for $t_w$, and $t$ for $k$.

When the web height to thickness ratio ($h/t_w$) is less than or equal to the limiting web height to thickness ratio ($h/tw$-limit) then the allowable shear stress is:

$$F_v = 0.40 \cdot F_y \cdot \text{ASD} (F4-1)$$

When the web height to thickness ratio ($h/tw$) is greater than the allowable web height to thickness ratio then the allowable shear stress is:

$$F_v = \frac{F_y \cdot K_v}{2.89 \cdot C_v} \cdot \text{ASD}^4 (F4-2)$$

When $C_v$ is less than or equal to 0.8 then $C_v$ is equal to:

$$C_v = \frac{45,000 \cdot K_v}{F_y \cdot \left(\frac{h}{t_w}\right)^2}$$

When $C_v$ is greater than 0.8 then $C_v$ is equal to:

$$C_v = \frac{190 \cdot C_v}{\left(\frac{h}{t_w}\right) \cdot \sqrt{\frac{K_v}{F_y}}}$$

When $\frac{L}{d - 2k}$ is less than 1.0:
\[K_v = 4.00 + \frac{5.34}{\left(\frac{L}{d-2k}\right)^2}\]

When \(\frac{L}{d-2k}\) is greater than 1.0:

\[K_v = 5.34 + \frac{4.00}{\left(\frac{L}{d-2k}\right)^2}\]

The allowable shear stress for steel angles is:

\[F_v = 0.40 \cdot F_y\]

An additional shear stress is added to the normal beam shear stress in a steel angle due to the torsional moment. A steel angle is not normally loaded directly over the vertical leg so there is a rather substantial moment created by this eccentric loading. StruCalc assumes the loads on the angle are through the center of the horizontal leg.

The additional shear stress is equal to:

\[f_v = \frac{M_t \cdot t}{J} \quad \text{Eq C3-2)}\]

Where \(M_t = \) moment from the load at 1/2 the depth of the angle

**NOMINAL SHEAR CAPACITY**

The shear capacity of the member is:

\[V_n = F_v \cdot d \cdot t_w\]

For tube steel:

\[V_n = F_y \cdot 2 (dx - 2 \cdot t)\]

**6.3.4 BEARING LENGTH REQUIRED**

The bearing length required for wide flange steel member is taken as the minimum of the distance \(k\) or the length required to prevent local web yielding or web crippling.

**LOCAL WEB YIELDING**

For simple beam modules and cantilevers with CS1 < d or CS2 < d then

\[BL \text{ due to web yielding} = \frac{R}{tw \cdot 0.66 \cdot F_y} - 2.5k \ \text{ASD}^4 (K1-3)\]

For cantilevers with CS1 > d or CS2 > d then:

\[BL \text{ due to web yielding} = \frac{R}{tw \cdot 0.66 \cdot F_y} - 5k \ \text{ASD}^4 (K1-2)\]

For tube steel members:
WEB CRIPPLING
For wide flange steel members:

For end supports:

\[
\text{BL due to web crippling} = \left( \frac{R}{34 \cdot tw^2 \cdot Fy \cdot \frac{tf}{tw}} \right) - 1 \cdot \frac{d}{3 \cdot \left( \frac{tw}{tf} \right)^{1.5}} \quad \text{ASD}^4 \ (K1-5)
\]

For interior supports then:

\[
\text{BL due to web crippling} = \left( \frac{R}{6.75 \cdot tw^2 \cdot Fy \cdot \frac{tf}{tw}} \right) - 1 \cdot \frac{d}{3 \cdot \left( \frac{tw}{tf} \right)^{1.5}} \quad \text{ASD}^4 \ (K1-4)
\]

For tube steel members:

For end supports:

\[
\text{BL due to web crippling} = \left( \frac{R}{34 \cdot t^2 \cdot Fy \cdot \frac{t}{t}} \right) - 1 \cdot \frac{dx}{3 \cdot \left( \frac{t}{t} \right)^{1.5}} \quad \text{Modified ASD}^4 \ (K1-5)
\]

For interior supports:

\[
\text{BL due to web crippling} = \left( \frac{R}{6.75 \cdot t^2 \cdot Fy \cdot \frac{t}{t}} \right) - 1 \cdot \frac{dx}{3 \cdot \left( \frac{t}{t} \right)^{1.5}} \quad \text{Modified ASD}^4 \ (K1-4)
\]

BEARING LENGTH CAPACITY
For wide flange steel members StruCalc takes the smaller of the bearing lengths required due to web yielding or web crippling and compares them to the distance \( k \). The maximum value is then taken as the bearing length required. For tube steel beams the minimum of the web yielding or web crippling values are used.

6.3.5 STEEL BEAM ADEQUACY
For steel members the program compares the moment of inertia of the member with the moment of inertia required, the allowable shear of the member with the maximum shear, and the moment capacity of the member with the maximum moment to determine adequacy. The percent adequacy and the controlling factor are printed on the adequacy bar and on the printouts.

6.4 FLITCH BEAM DESIGN
StruCalc can design a flitch beam (which is a steel plate sandwiched between two pieces of lumber) in any of the beam modules. Choose the type of flitch beam from the left toolbar (solid sawn or structural composite), click the Flitch Plate checkbox, and then choose the thickness and the number of the steel plate(s). StruCalc uses the transformation method to convert the steel plate into an equivalent wood section.
6.4.1 TRANSFORM CALCULATIONS

The size of the transformed wood member is based on the ratio of the modulus of elasticity of the steel and the modulus of elasticity of the wood. The relative stiffness of the two materials dictates the amount of load each type of material resists.

\[ n = \frac{E_{\text{Steel}}}{E'_{\text{Wood}}} \]

The depth of the transformed section remains constant, a transformed width is determined based on the modular ratio:

\[ w_{\text{trans}} = w_{\text{wood}} \cdot (\# \text{ of laminations}) + (w_{\text{steel}} \cdot n) \]

The stress in each material must be limited to less than the allowable. In most cases the steel would be overstressed and as a result the allowable wood stress is limited. The allowable stress for A36 steel is:

\[ F_{b}(\text{steel}) = F_{y} \cdot 0.66 \]
\[ F_{b}(\text{steel}) = 36,000 \text{ psi} \cdot 0.66 = 21,600 \text{ psi} \]

The allowable stress in the wood portion of the beam is limited:

If \[ \frac{21,600}{n} < F_{b}'(\text{wood}) \] then \( F_{b}'(\text{wood}) \) is limited to \[ \frac{21,600}{n} \]

StruCalc will only design flitch beams with full lateral support for the compression portion of the beam. For a simple span beam with positive loading this requires the top of the beam to be adequately tied into the floor/roof system to prevent out of plane buckling. For a multi-span or cantilever beam both the top and the bottom of the beam must be restrained.

The shear strength of the steel plate is calculated as a steel beam with a h/w ratio equal to the height of the plate divided by the width. See section 6.33 for steel shear calculations. The resultant allowable shear stress of the plate is then compared to the allowable shear stress of the wood portion of the beam and is limited in the same way as the bending stress above.

The bearing length required for the flitch beam is based on the wood portion of the beam only. The bearing length can be reduced if a steel plate is provided to spread the load from the steel portion of the beam to the support. This calculation is not provided by StruCalc.

6.4.2 FLITCH BEAM CONNECTIONS

Flitch beams must be connected together to transfer the loads to the wood and steel portions of the beam in proportion to the relative stiffness of the member. StruCalc does not provide this calculation but we have provided two sample methods below for determining this connection.

The first is an empirical method based on what has worked well in the past. Use a regular bolting pattern of 1/2" or 5/8" bolts at 16" o.c.. Stagger the bolts and provide minimum edge distance of 2 1/2" from the edge of the beam.

The rational method is to actually calculate the load transfer between the steel and wood members using the code referenced NDS. The load carried by the steel plate can be determined by multiplying
the percentage of the load carried by the steel plate shown on the StruCalc printout by the load on the beam and designing the bolts to transfer this load.

Example Calculation:

Beam length = 10 ft
(2) 2x12 #2 Doug Fir Larch and (1) 3/8" Steel Plate
Total Uniform load on the beam = 800 plf
Percentage of Load Carried by Steel Plate = 81.9%

Load needed to be transferred to plate = 800 plf \cdot 0.819 = 655 plf

Check NDS capacity of 1/2" bolts perpendicular to grain. For simplicity we will use the double shear capacity from table 11F. A higher value can be calculated using the six yield equations.

Bolt capacity = 730 lbs

Bolt spacing required = \frac{730}{655} = 1.11 \text{ ft}

\therefore \text{Space bolts at 12" o.c.}

Calculate end reaction at plate:

R = 655 \cdot 10 \text{ ft} = \frac{6550}{2} = 3275 \text{ lb}

End bolts required to transfer steel plate load to wood members for bearing are required unless the steel plate bears on a steel bearing plate.

Number of bolts = \frac{3275 \text{ lb}}{730 \text{ lb}} = 4.48

\therefore \text{Use five bolts at the end of the beam.}

This is just one example of how to design the bolting for a flitch beam; there certainly are other valid methods and assumptions that will provide an adequate design.
7. SIMPLE BEAM MODULES

7.1 UNIFORMLY LOADED FLOOR BEAM

7.1.1 DESCRIPTION

The uniformly loaded floor beam module is used to design beams loaded by floor joists or trusses. Different live loads are allowed on each side of the beam in order to model different occupancy requirements. Different dead loads are also allowed on each side of the beam to account for any weight differences in floor materials. See figure 7-1.

StruCalc sets the load deflection criteria to the code required live load (L/360) and total load (L/240) deflection limits as well as setting the duration factor to 1.0 for normal loading duration.

See section 5.2.9 for floor live load reduction commentary.
See section 6.2.2 for the incising factor commentary.

7.1.2 LOADING

The floor dead load and live load can be inputted for both tributary widths and they are in pounds per square foot. The wall load is a uniform dead load that is applied along the entire length of the beam.
7.2 ROOF BEAM

7.2.1 DESCRIPTION
The roof beam module is used to design roof beams loaded by roof rafters or trusses. The roof beam can be designed with an end elevation difference and also a roof pitch for the attached rafters or trusses. See figure 7-2.

![Figure 7-2 Roof Beam Example](image)

StruCalc sets the load deflection criteria to the code recommended live load (L/240) and total load (L/180) deflection limits as well as setting the duration factor to 1.15 for snow loading duration. For non-snow regions, the duration factor is set at 1.25.

See section 5.2.5 for snow/non-snow commentary.
See section 5.2.7 for slope and pitch commentary.
See section 6.2.2 for the incising factor commentary.

7.2.2 HIP/VALLEY ROOF BEAM DESIGN
Roof hip beams and valley beams can be easily designed using the StruCalc Roof Beam module by simply clicking on the box labeled “Hip/Valley Beam”. A picture of the hip/valley beam will appear and the user can then define the tributary width of roof to be carried by the beam. See figure 7-3.

Note: If the rafter length is manually inputted into the Hip/Valley section of the module, the span length at the top of the roof beam module will automatically be updated by StruCalc to reflect any changes.
The hip/valley beam module can design a simple or multi-span hip/valley beam as well as a cantilevered hip beam. Just check the Intermediate Support checkbox and specify the distance from the left end of the beam to the middle support. The program will automatically adjust the inputs to match the actual physical beam parameters. It may take some iteration of the lengths, tributary widths, and roof pitch to accurately model the actual design. The A support can be removed for a cantilevered hip/valley beam, and a point load can be added to the cantilevered end.

7.3 COMBINATION ROOF AND FLOOR BEAM

7.3.1 DESCRIPTION
The combination roof and floor beam module is used to design beams carrying both roof loads and floor loads. See figure 7-4.

StruCalc sets the load deflection criteria to the code recommended live load (L/360) and total load (L/240) deflection limits for floor loading since they are more conservative than the roof recommended values and will usually provide a “stiffer” beam design. The duration factor is set to 1.15 for the roof portion of the loading and 1.0 for the floor portion of the loading. For non-snow regions, the duration factor is set to 1.25 for the roof portion of the loading.

See section 5.2.5 for snow/non-snow commentary.
See section 5.2.7 for roof pitch commentary.
See section 5.2.9 for floor live load reduction commentary.
See section 6.2.2 for the incising factor commentary.
8. MULTI-SPAN BEAM MODULES

8.1 MULTI-SPAN ROOF BEAM

8.1.1 DESCRIPTION
The multi-span roof beam module provides the user with the ability to design roof beams with one, two, or three spans with up to two of those spans being cantilevers. A point load can be applied on all three spans of the beam as well as uniform loads and wall loads. See figure 8-1.

![Figure 8-1 Multi-Span Roof Beam Example](image)

**FIGURE 8-1 MULTI-SPAN ROOF BEAM EXAMPLE**

StruCalc sets the default load deflection criteria to the code required live load (L/240) and total load (L/180) deflection limits as well as setting the duration factor to 1.15 for snow loading duration. For non-snow regions, the duration factor is set to 1.25.

See section 5.2.5 for snow/non-snow commentary.
See section 5.2.7 for roof pitch commentary.
See section 6.2.2 for the incising factor commentary.

**UNBALANCED LOADS**
When the unbalanced load checkbox is not checked StruCalc applies all the loads simultaneously for the live, dead and total load cases. The beam stresses can be substantially lower than would occur if the “Check Unbalanced Loads” box were checked, therefore unbalanced loads should always be considered in a beam design. Furthermore, care should be taken so that field conditions always match the design loading condition otherwise dangerous stresses or uplift forces at the reactions can occur.
UPLIFT FACTOR OF SAFETY
The uplift factor of safety reduces the uniform dead load during the minimum reaction calculations. The purpose of the dead load reduction is to avoid an underestimated uplift force. The uplift factor of safety does not affect other calculations. This setting can be found in the settings under the options tab.

8.2 MULTI-SPAN FLOOR BEAM

8.2.1 DESCRIPTION
The multi-span floor beam module provides the user with the ability to design floor beams with one, two, or three spans with up to two of those spans being cantilevers. A point load can be applied to all three spans of the beam as well as uniform loads and wall loads. See figure 8-2.

![FIGURE 8-2 MULTI-SPAN FLOOR BEAM EXAMPLE](image)

StruCalc sets the load deflection criteria to the code recommended live load (L/360) and total load (L/240) deflection limits as well as setting the duration factor to 1.0 for normal loading duration.

See section 5.2.9 for floor live load reduction commentary.
See section 6.2.2 for the incising factor commentary.

8.3 MULTI-LOADED BEAM

8.3.1 DESCRIPTION
The multi-loaded beam module provides the user with the ability to design most beams with one, two, or three spans with up to two of those spans being cantilevers. Six point loads, a uniform load, and four partial uniform/trapezoidal loads can be applied to all three spans. See figure 8-3 for an example application of the multi-loaded beam module.
FIGURE 8-3 MULTI-LOADED BEAM EXAMPLE

Each span is loaded independently by clicking on the button at the center of the screen indicating the span desired. Each load location is then measured from the left end of the individual span. For example, to apply the point load P1 on span L3 to the beam shown in Figure 8-3, first click on the picture of the left span and input the live and dead load. The location would be inputted as the distance X-1 above. Note that to apply a uniform load to all three spans it is necessary to input the load three times, one time for each span.

StruCalc sets the load deflection criteria to the code recommended live load (L/360) and total load (L/240) deflection limits as well as setting the duration factor to 1.0 for normal loading duration. Always check the deflection criteria and duration factor to make sure that they are set correctly for the specific member that is being designed.

See section 7.2.2 for the incising factor and repetitive use factor commentary.
9. JOISTS AND RAFTERS

9.1 FLOOR JOIST

9.1.1 DESCRIPTION
The floor joist module is used to design simple span, multi-span, and cantilevered joists. A uniform load, a wall load (perpendicular to joists), and a partially distributed load can be applied to each span of the joists. See figure 9-1 for an example application.

![FIGURE 9-1 EXAMPLE JOIST APPLICATION](image)

StruCalc sets the load deflection criteria to the code recommended live load (L/480) and total load (L/360) deflection limits for floor loading and the duration factor to 1.0 for normal loading conditions.

LOADING
The code required concentrated load is required by several building codes. The load is applied over an area 30 inches square at the most critical location on the joist. The shear and moment produced by the concentrated load are compared with the values from the full uniform loading of the joist, the greater value controls.

The wall loading in the floor joist module is entered in pounds per linear foot and it is applied across the floor joists and then it is converted into a point load on each individual joist for calculation.

I-JOISTS
The program does not allow the design of cantilevered I-joists. This in no way implies that I-joists are unsuited for cantilever loading situations, but due to the various manufacturer specifications and requirements the design is best handled by the particular joist manufacturer or their technical representative. In addition, StruCalc does not include the option of glued and screwed plywood for I-
joists. The design of I-joists with plywood sheathing requires a proprietary method for each I-joist manufacturer.

The I-joists designed by StruCalc are for very specific loading conditions, if the design does not match the actual joist loading or span conditions in any way, contact the joist manufacturer for design verification.

StruCalc will check for the need for web stiffeners over supports if the manufacturer provided the values in their literature.

PLYWOOD DECKING
For solid sawn joists, StruCalc will consider the affect of glued and screwed plywood in deflection calculations. The plywood combines with the joists to form a “T beam” which can significantly increase the moment of inertia and thereby reduce deflection. The use of glued and screwed plywood may be beneficial when joist spans are long and deflection is the controlling factor.

9.1.2 CALCULATIONS

I-JOIST CALCULATIONS
StruCalc compares the published moment, shear, reaction, and deflection capacities against the calculated values of the joist to determine adequacy. Note that the program does not include the addition of web stiffeners in any of the calculations or design. The addition of web stiffeners or increasing the bearing length can greatly increase the reaction capacity of the I-joists.

PLYWOOD CALCULATIONS
The addition of plywood to the joists increases the moment of inertia but does not affect section modulus and area calculations. Only the area of the plywood parallel to the joists is considered for moment of inertia calculations which is a conservative approximation since the ply’s perpendicular to the joist will also add to the “T beam” effect.

![Figure 9-2 Plywood Moment of Inertia Dimensions](image)

**FIGURE 9-2 PLYWOOD MOMENT OF INERTIA DIMENSIONS**

The area of plywood parallel to the joist:
where:

\[ T_{ply} = \text{the area of the plywood parallel to the joist per foot}. \]

This calculation assumes that the plywood is applied with the face grain perpendicular to the joists.

The centroid of the plywood and joists (see figure 9-2):

\[ c = \frac{\left( \frac{A_{joist} \cdot d}{2} \right) + A_{ply} \left( \frac{d + t}{2} \right)}{A_{joist} + A_{ply}} \]

The combined moment of inertia:

\[ I_{comb} = \frac{w \cdot d^3}{12} + A_{joist} \left( \frac{d}{2} - c \right)^2 + A_{ply} \left( \frac{d + t}{2} - c \right)^2 \]

9.2 ROOF RAFTER

9.2.1 DESCRIPTION

The roof rafter module is used to design rafters with overhangs. There is an option to design the rafters with a double live load on the cantilever portion of the rafter. This is required by some codes to account for the possibility of an ice dam forming on the eaves. See figure 9-3.
See section 5.2.7 for commentary on roof pitch adjustments.

The program prints out the equivalent tributary width, which can be used to input directly into other modules.

\[
UT_{\text{eq}} = \frac{L}{2}
\]

\[
LT_{\text{eq}} = \frac{(L + \text{Leave})^2}{2 \cdot L}
\]
10. COLUMNS

10.1 INPUTS AND DESCRIPTION

DESCRIPTION
The column module will design axially loaded columns and studs with eccentricity and lateral loads. The lateral loads can only be applied to one face of the column at a time and are intended for wind and seismic loads.

FIGURE 10-1 COLUMN DIMENSIONS

UNBRACED LENGTH
The unbraced length of a column (Lx, Ly) indicates the distance between points of support for each bending axis (see figure 10-1). If the column is restrained from bending in one direction then the unbraced length in that direction can be set to zero. An example would be a standard 2x6 stud framed into a wall, as shown in figure 10-2.
In this case the stud is restrained from bending about the Y-axis and the unbraced length in the Y direction is zero (Ly = 0) while the unbraced length in the X direction would be the length of the stud. Care must be taken not to overestimate the amount of lateral support since it has a critical effect on the adequacy of the column.

COLUMNS DIMENSIONS
The column dimensions are set with the width equal to dy and the depth equal to dx (See figure 10-1). StruCac always treats the x-axis as the strong axis and the y-axis as the weak axis.

COLUMN END CONDITION
The column end condition (Ke for wood design, K for steel) is used to determine the effective length of the column. The effective length of a column can exceed its actual length by as much as a factor of two, which can greatly affect the column's capacity. The value of K or Ke is dependent on the end fixity and lateral translation of the column, and as a result requires professional judgment to determine the restraint conditions. Theoretical and recommended values for K and Ke are shown in table 10-1.

It should be noted that it is a common, yet conservative practice to set K and Ke greater than or equal to 1.0 even if the bracing conditions justify a lesser value.

COLUMN BENDING COEFFICIENT
The column bending coefficient (Cm) is only available in steel column design and is used to reduce the effect of second order moment amplification due to eccentricities. This value may be conservatively taken as 1.0. See chapter H in the Manual of Steel Construction Allowable Stress Design, ninth edition (AISC4) for a thorough explanation of the calculation of the reduction factor.

LOAD ECCENTRICITY
The addition of a load eccentricity (ex, ey) allows for the possibility that the loading is not a pure axial load (see figure 10-1). Any eccentricity in the loading develops a bending moment in the column, which decreases the adequacy of a given member. It is recommended that a minimum eccentricity of the larger of 1 inch or 1/10th of the member dimension be used. It should be noted that eccentricity can significantly affect the column design, and that the inclusion of eccentricity is not required by any of the building codes.

LATERAL LOADS
Lateral loading on a column is generally a wind/seismic lateral load transferred to the column by the wall sheathing or framing. StruCac allows the user to apply three point loads and one uniform load to one face of the column. This load is applied using the load combinations required by the building code chosen.

LAMINATED COLUMNS
StruCac only allows built up columns of multiple 2x solid sawn members. The narrow face of the laminated member is always in the dy direction. Please note that for the design calculations to be
valid the members must be nailed or bolted per requirements of the National Design Specifications for Wood Construction section 15.3.3 and 15.3.4, (NDS\textsuperscript{5,5}).

<table>
<thead>
<tr>
<th>Buckled shape of column is shown by dashed line</th>
</tr>
</thead>
<tbody>
<tr>
<td>![Diagram of buckled shapes]</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Theoretical K, Ke value</th>
<th>0.5</th>
<th>0.7</th>
<th>1.0</th>
<th>1.0</th>
<th>2.0</th>
<th>2.0</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Recommended design value when ideal conditions are approximated</th>
</tr>
</thead>
<tbody>
<tr>
<td>K (Steel)</td>
</tr>
<tr>
<td>Ke (Wood)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>End condition legend</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rotation fixed and translation fixed</td>
</tr>
<tr>
<td>Rotation free and translation fixed</td>
</tr>
<tr>
<td>Rotation fixed and translation free</td>
</tr>
<tr>
<td>Rotation free and translation free</td>
</tr>
</tbody>
</table>

**TABLE 10-1 COLUMN LENGTH FACTOR TABLE**

**MACHINE STRESS RATED LUMBER**

The Machine Stress Rated Lumber checkbox is only available for wood columns. This checkbox sets the Euler buckling coefficient (Kce) to 0.418. Note that StruCalc does not use the MSR designated stress values listed in the NDS; therefore when designing for MSR lumber, the bending stress and modulus of elasticity values used in the design should represent the actual MSR designation that will be used in construction. In addition, MSR lumber only comes in 2x members and StruCalc does not disable the checkbox for larger members. MSR lumber is usually only available by special order. You can designate MSR/MEL lumber in the Materials Database Editor.

**STUD DESIGN**

The Stud Design checkbox is located in the section toolbar. StruCalc will automatically distribute the loads to the stud based on spacing and will apply the repetitive use factor if allowed.

**CANTILEVER COLUMN**

To design a cantilever column, click the cantilever column checkbox on the column form. The K(e) factor will be adjusted for a cantilever situation. The proper length of the cantilever column for design is at the point of fixity below the ground to the top of the column. Unless the base is constrained by a slab or some other method this will rarely be at the surface. StruCalc does not calculate the depth of embedment required to resist the lateral loads.
10.2 COLUMN CALCULATIONS

DEFLECTION
Deflection is calculated based on the lateral loading only. The actual deflection for an axially and laterally loaded column may be larger than indicated on the printout due to the additional deflection caused by the P-Delta effect. The live load deflection limit can be set to limit the amount of deflection allowed for an adequate design.

SHEAR
This program does not check the shear capacity of a laterally loaded column. In the majority of situations the bending and compression capacity of the column will be the controlling factor. If you are designing a very short heavily loaded column you should check its capacity in shear by applying the same loads in one of the beam modules.

MOMENT
The bending moment about the X-axis due to eccentric loading is:

\[ M_{x-ex} = PT \cdot ex \]

The bending moment about the Y-axis due to eccentric loading is:

\[ M_{y-ey} = PT \cdot ey \]

The Lateral Load Moment is calculated for the axial direction in which the lateral loads are applied.

Uniform lateral load (w):

\[ M@Y = \left( \frac{w \cdot Y}{2} \right) (L - Y) \]

Where:
Y is the distance along the column where the moment is calculated.

Lateral Point load (P):

The moment at point Y due to a point load at a distance X from the left support:

When \( Y < X \):

\[ M@Y = \frac{P \cdot Y \cdot (L - X)}{L} \]

When \( Y > X \):

\[ M@Y = \frac{P \cdot Y \cdot X}{L} \]

LOAD COMBINATIONS
The axial loads and lateral loads are applied based on the pertinent load combination for the code used. The following combinations are used:

DL + LL Only
DL + Lateral Loads
DL + LL + Lateral loads (non wind/seismic)
DL + .75 * [LL + Lateral Loads] (wind/seismic)

10.3 WOOD COLUMNS

10.3.1 PROPERTIES AND CALCULATIONS

TABULATED WOOD COLUMN DESIGN STRESSES

$F_b =$ the column bending stress.

$F_c =$ the column compressive stress parallel to grain, in psi.

$E =$ the column modulus of elasticity, in psi.

$F_{bx}$ and $F_{by} =$ the column bending stresses in the X and Y direction, in psi.

ADJUSTED DESIGN STRESSES

The design stresses are adjusted based on the 1991, 1997, 2001 or 2005 NDS depending on the building code used. See sections 5.1 and 5.2 for commentary on code specific adjustment factors not covered in this section.

The column module allows the user to manually add the repetitive use factor for design. This allows the correct design of laterally loaded studs and should only be used when dimensional members are spaced not more than 24 inches on center or when three or more dimensional members are in contact. The repetitive member factor is equal to 1.15.

DURATION FACTOR AND LOAD CASES

The load combinations specified by the building code are discussed in section 9.2. The basic load combinations are supplemented by additional load cases determined by the chosen duration factors.

The following load cases are checked:

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Loads</th>
<th>Duration Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead Load Only</td>
<td>DL</td>
<td>0.9</td>
</tr>
<tr>
<td>Dead + Live</td>
<td>LL + DL</td>
<td>Axial Load</td>
</tr>
<tr>
<td>Dead + Lateral</td>
<td>DL + Lateral</td>
<td>Lateral load</td>
</tr>
<tr>
<td>Dead + Live + Lateral</td>
<td>DL + LL + Lateral Or DL + .75* (LL + Lateral)</td>
<td>Larger of Axial or Lateral Load</td>
</tr>
</tbody>
</table>

SECTION MODULUS

The section modulus about the X-axis is:

$$S_x = \frac{dy \cdot dx^2}{6}$$

The section modulus about the Y-axis is:
\[ Sy = \frac{dx \cdot dy^2}{6} \]

**COMPRESSION SLENDERNESS RATIO**
The column compression slenderness ratio is a measure of the column's resistance to lateral buckling and is used to calculate the column capacity. The slenderness ratio is limited to 50 in both directions.

The column compression slenderness ratio about the X-axis is:

\[ \frac{Lex}{dx} = \frac{Ke \cdot Lx \cdot \left(\frac{12\text{in}}{\text{ft}}\right)}{dx} \]

The column compression slenderness ratio about the Y-axis is:

\[ \frac{Ley}{dy} = \frac{Ke \cdot Ly \cdot \left(\frac{12\text{in}}{\text{ft}}\right)}{dy} \]

**COMPRESSIVE STRESS**
The Compression stress due to pure axial loading is:

\[ f_c = \frac{PT}{A} \]

**BENDING STRESS**
The column bending stress about the X-axis is:

\[ f_{bx} = \frac{Mx \cdot \left(\frac{12\text{in}}{\text{ft}}\right)}{Sx} \]

The column bending stress about the Y-axis is:

\[ f_{by} = \frac{Mx \cdot \left(\frac{12\text{in}}{\text{ft}}\right)}{Sy} \]

**BASE REACTIONS**
The base reactions are the reactions transferred from the column into the supporting framework or footing.

The base live load, in pounds is:

\[ RL = PL \]

The base dead load including the column self weight, in pounds is:

\[ RD = PD + \left( L \cdot A \cdot \left(\frac{\text{ft}^2}{144\text{in}^2}\right) \cdot \left(\frac{35\text{lb}}{\text{ft}^3}\right) \right) \]

The base total load in pounds is:
RT = RL + RD

**ADJUSTMENT FACTORS**

The NDS\(^5,9\) applies a stability factor to the allowable compressive stress that provides a smooth transition between failure due to compression and failure due to buckling. This factor is applied to the compressive stress along with the duration factor (Cd), the wet use factor (Cm) and the size factor (Cf). All of the factors covered in section 6.2 are applied to the bending stress (Fb), however the flat use factor is only applied to bending about the Y-axis.

**COLUMN STABILITY FACTOR**

The column stability factor is calculated for the axis with the largest compression slenderness ratio and will reduce to 1.0 for a fully braced column.

\[
C_p = \frac{1 + \left( \frac{F_{ce}}{F_{c}^*} \right)}{2c} - \sqrt{1 + \left( \frac{F_{ce}}{F_{c}^*} \right)^2 - \frac{F_{ce}}{F_{c}^* c}} \quad \text{NDS}^{5,9,20} (3.7.1)
\]

Where:

- \(F_{c}^* = \) Tabulated Fc value multiplied by all applicable adjustment factors except Cp.

\[
F_{ce} = \frac{K_{ce} \cdot E'}{(\text{larger of Ley/} \text{dy or Lex/dx})^2}
\]

\[
F_{ce} = \frac{0.822 \cdot E - \text{min'}}{(\text{larger of Ley/dy or Lex/dx})^2} \quad \text{for 05 NDS}
\]

- \(K_{ce} = 0.3\) for visually graded lumber.
- \(K_{ce} = 0.418\) for glued laminated timber and Machine Stress Rated lumber (MSR).
- \(c = 0.8\) for sawn lumber.
- \(c = 0.9\) for glued laminated timber and Machine Stress Rated lumber (MSR).

**BUILT UP COLUMNS**

The stability factor is multiplied by the Kf factor to account for the effectiveness of shear transfer between the individual laminations of built up columns. The Kf factor only applies to the axis parallel to the weak axis of individual laminations, in StruCalc it is always the Y-axis.

- \(K_{f} = 0.6\) for columns nailed per NDS\(^5,9,20\) section 15.3.3.
- \(K_{f} = 0.75\) for columns bolted per NDS\(^5,9,20\) section 15.3.4.

**COMBINED STRESS FACTOR**

The combined stress factor is the measure of adequacy of an eccentrically or laterally loaded column. The following formula is based on the X-axis being the strong axis. For some instances of built up columns where dy is greater than dx then the directions in the formula will be reversed. See NDS\(^5,9\) (15.4).
The column adequacy is presented as a percentage of the combined stress factor (CSF) for a laterally loaded column or eccentrically loaded column. For an axial load only the column adequacy percentage is equal to the percentage difference between \( f_c \) and \( F_c' \). Note that the combined stress factor can provide erroneous results when the column is inadequate, i.e. the percentage inadequate will fluctuate as the load increases. This is a result of the rather large number of parameters contained in the formula.
10.4 STEEL COLUMNS

The steel design calculations and steel properties are taken from the 13th edition of the Manual of Steel Construction Allowable Stress Design, ASD. The 9th edition calculations are also available.

We have chosen not to reproduce the 13th edition calculations here. They can be found in the Manual of Steel Construction 13th Edition which can be downloaded for free at www.aisc.org.

Below are the steel calculations for the 9th edition ASD.

10.4.1 PROPERTIES AND CALCULATIONS

TABULATED STEEL COLUMN DESIGN PROPERTIES

Fy = the column steel yield stress, in psi.
E = the modulus of elasticity of the column, in psi.
t = the wall thickness of the column, in inches.
A = the net area of the column, in square inches.
Ix and Iy = the moment of inertia about the X and Y-axis, in in^4.
Sx and Sy = the section modulus about the X and Y-axis, in in^3.
rx and ry = the radius of gyration about the X and Y-axis, in inches.

COMPRRESSIVE STRESS

The actual column compressive stress is:

\[ f_a = \frac{P_T}{A} \]

COLUMN SLENDERNESS RATIO

The column slenderness ratio in the X and Y direction is:

\[ \frac{K_{lx}}{rx} = \frac{Ke \cdot Lx \cdot \left(12\text{in}/\text{ft}\right)}{rx} \]
\[ \frac{K_{ly}}{ry} = \frac{Ke \cdot Ly \cdot \left(12\text{in}/\text{ft}\right)}{ry} \]

The column slenderness ratio separating elastic and inelastic buckling is:

\[ C_c = \sqrt{\frac{2\pi^2 \cdot E}{Fy}} \]

ALLOWABLE COMPRESSIVE STRESS

When the maximum of \(K_{lx}/rx\) and \(K_{ly}/ry\) is less than \(C_c\) than:
Where:

\[ \frac{1 - \left( \frac{KL/r}{r} \right)^2}{2Cc^2} \cdot F_y \]

\[ \frac{5}{3} \cdot \frac{3(\frac{KL/r}{r})}{8Cc} \cdot \frac{(\frac{KL/r}{r})^3}{8Cc^3} \]

AISC^4 (E2-1)

\[ \frac{12\pi^2 \cdot E}{23(\frac{KL/r}{r})^2} \]

AISC^4 (E2-2)

Where:

\[ KL/r = \text{the maximum of } KLx/rx \text{ and } Kly/ry. \]

When the maximum of \( KLx/rx \) or \( Kly/ry \) is greater than \( Cc \) then:

\[ \frac{12\pi^2 \cdot E}{23(\frac{KL/r}{r})^2} \]

AISC^4 (E2-2)

Where:

\[ KL/r = \text{the maximum of } KLx/rx \text{ and } Kly/ry. \]

**BENDING STRESS**

Column bending stress in the X direction is:

\[ f_{bx} = \frac{M_x}{S_x} \]

Column bending stress in the Y direction is:

\[ f_{by} = \frac{M_y}{S_y} \]

**ALLOWABLE BENDING STRESS**

See section 5.4 for allowable bending stress calculations.

**EULER’S STRESS**

The Euler’s stress in the X direction used for combined stress calculations is:

\[ F_{ex}' = \frac{12\pi^2 \cdot E}{23(KLx/rx)^2} \]

AISC^4 (H)

The Euler’s stress in the Y direction used for combined stress calculations is:

\[ F_{ey}' = \frac{12\pi^2 \cdot E}{23(Kly/ry)^2} \]

AISC^4 (H)

**COLUMN COMBINED STRESS CALCULATIONS**

The following two equations must be satisfied for column adequacy:
CSF(1) = \frac{fa}{Fa} + \left(1 - \frac{fa}{Fex}\right) \cdot \frac{Cm \cdot fbx}{Fbx} + \left(1 - \frac{fa}{Fey}\right) \cdot \frac{Cm \cdot fby}{Fby} \leq 1.0 \quad \text{AISC}^4 \quad (H1-1)

CSF(2) = \frac{fa}{0.60Fy} + \frac{fby}{Fby} \leq 1.0 \quad \text{AISC}^4 \quad (H1-2)

When \frac{fa}{Fa} \leq 0.15 \text{ then the following equation may be used in lieu of CSF(1) and CSF(2):}

CSF(3) = \frac{fa}{Fa} + \frac{fby}{Fby} \leq 1.0 \quad \text{AISC}^4 \quad (H1-3)

**BASE REACTIONS**
Base live load:

RL = PL

Base dead load:

RD = PD + L \cdot A \cdot \left(\frac{ft^2}{144\text{in}^2}\right) \cdot 490

Base total load:

RT = RL + RD

**COLUMN ADEQUACY**
The column adequacy is presented as a percentage of the controlling combined stress factor (CSF).
11. FOOTINGS

11.1 DESCRIPTION

The footing module sizes square, rectangular, round and continuous footings, with reinforcement or without, based on the design requirements of the chosen building code and the ACI\textsuperscript{7,18,19}. See figure 11-1 for a typical square/rectangular footing application.

![Figure 11-1 Typical Square/Rectangular Footing Application]

**FIGURE 11-1 TYPICAL SQUARE/RECTANGULAR FOOTING APPLICATION**

11.1.1 SQUARE/RECTANGULAR FOOTING INPUTS

COLUMN TYPE

The type of column that will be supported on the footing should be selected. The critical sections in the footing, at which the maximum moment is calculated, differ depending on the type of column selected, therefore it is important to select the correct column type. See ACI\textsuperscript{7,18,19} (15.4).

Note: If a "wood" or "other" column type is selected, the critical section, at which the maximum moment is calculated, is located at the center of the footing.

LOADING

The service (unfactored) live load (LL) and the service dead load (DL) are to be entered into the input boxes. The program applies the code required factors to the loads and the ultimate factored load (Pu) is calculated.
Note: To calculate the total concentrated load on the footing using the program "Load calculator"; check the "Load Calculator" box and input the appropriate service live loads, service dead loads, and tributary areas.

MATERIAL PROPERTIES
fc' = The concrete compressive strength, in psi.
fy = The reinforcing steel yield strength, in psi.

SOIL BEARING PRESSURE
The allowable soil bearing pressure (Qs) should be determined by a thorough soil analysis of the site by a registered professional. If a soil report is not available then use the values specified by the local building code.

The "Allow Bearing Pressure Increase" checkbox is available when using the 94 and 97 UBC. The code allows a 20% increase in the allowable soil bearing pressure for each additional foot of width or depth greater than 12 inches. This increase is limited to three times the inputted value and is not allowed for clay, sandy clay, silty clay, or clayey silt. See 1994 UBC (Table 18-I-A) or 1997 UBC (Table 18-I-A).

The "Calculate Soil Weight Above Footing" checkbox is used to calculate the total overburden weight of any soil on top of the footing; this value is then subtracted from the allowable soil bearing pressure (Qs).

REINFORCEMENT
The program allows the selection of #3 through #11 reinforcement bars.

Note: Check the "Equal Bar Spacing" box to have the program design a rectangular footing with equal bar spacing throughout the "center band width W" and the "outer bands". See figure 11-2 below. This is generally a conservative design in that the required bar spacing for the "center band width W" is also specified for the "outer bands." (This applies to rectangular footings only)
CONCRETE COVER
The concrete cover (c) is the distance between the edge of the lowest layer of reinforcement and the concrete surface in inches. See figure 11-3 below. The code minimum cover for concrete cast against and permanently exposed to earth is 3 inches. See ACI 7.18 (7.7). The required reinforcement development length is a function of concrete cover; therefore it may be beneficial in some cases to specify a larger cover than the required minimum.

![Figure 11-3 Typical Square/Rectangular Footing Section]

PLAIN CONCRETE
To design footings without steel reinforcement check the “Plain Concrete” box. The program does not limit the size of plain concrete footings. However, when designing large footings special attention should be made to the possibility of shrinkage/temperature cracking. Furthermore, plain concrete footings are not recommended in any situation due to the possibility of sudden failure or strength loss due to overloading or cracking.

COLUMN DIMENSIONS
m = column width in inches.

n = column depth in inches.

F = The distance between the edge of the column and the edge of the baseplate in inches.

See figure 11-4.

BASEPLATE
The column baseplate specifies the area of loading for a steel column supported by the footing. The baseplate is centered on the footing as shown in figure 11-4. The width (bsw) and length (bsl) of the baseplate are entered in inches. Regarding circular columns ACI 7.18.19 (15.3) states, “For location of critical sections for moment, shear, and development of reinforcement in footings, it shall be permitted to treat circular or regular polygon shaped concrete columns or pedestals as square members with the same area.”
FIGURE 11-4 FOOTING DIMENSIONS

FOOTING DIMENSIONS
W = The footing width, in feet.
L = The footing length, in feet.
Depth = The total footing depth, in inches.

Note: StruCalc automatically subtracts two inches from the inputted depth of plain footings to account for soil contamination of the concrete per ACI\textsuperscript{7,18,19} (22.4.8).

11.1.2 CONTINUOUS FOOTING INPUTS

STEMWALL TYPE AND WEIGHT
The type of stemwall that will be supported on the footing should be selected. The critical sections in the footing, at which the maximum moment is calculated, differ depending on the type of stemwall selected, therefore it is important to select the correct column type. See ACI\textsuperscript{7,18,19} (15.4). Depending
on the type of stemwall that is selected; StruCalc will automatically enter a default weight in pounds per cubic foot. The user can change this value if the actual weight differs from the default value.

Note: If a “wood” or “other” stemwall type is selected, the critical section, at which the maximum moment is calculated, is located at the center of the footing. In addition, if a “wood” stemwall type is selected the weight input box will disappear.

11.1.3 FOOTING DESIGN PROCESS
Sizing a footing using StruCalc is an iterative process. Several attempts are required to optimize the footing design for each unique situation. The following steps can be used as a guideline for most designs:

1. Determine all of the design parameters and code requirements.
2. Input all of the known parameters and trial parameters. The calculated width required and length required (when applicable) will be displayed at the bottom of the input boxes and the adequacy statement(s) will be displayed on the adequacy bar.
3. Inspect any inadequacy statements (see section 11.1.4) and adjust the necessary parameters and calculate again.
4. Repeat step 3 until the footing is adequate.
5. Optimize the footing by selecting the smallest size with the least expensive materials that provides an adequate design.

11.1.4 INADEQUACY TROUBLESHOOTING

FOOTING SIZE INADEQUATE
The footing size inadequate message will appear when the footing size is not sufficient to support the load on the footing. This is a factor of the allowable soil bearing pressure, allowable bearing pressure increase, depth, and the length/width. The allowable bearing pressure is not normally a parameter that can be changed since it is site specific. The depth is a factor due to the increased self-weight of the footing but usually is not changed due to a footing size inadequacy.

If the allowable bearing pressure increase checkbox is checked then the footing size required is a factor of the footing size. In this case several different sizes may need to be checked until the footing size required is smaller but close to the footing size. If the bearing pressure increase checkbox is not checked or not available in the code that is in effect, then choose a footing length/width larger than the required footing length/width required.

FOOTING DEPTH INADEQUATE
The footing depth inadequate message will appear when the footing depth is not sufficient to meet the code allowed minimums. Increase the depth until the depth is no longer inadequate.

ONE-WAY SHEAR FAILURE
The one-way shear failure message will appear when the footing fails in one-way shear. Increase the depth of footing until the shear strength of the footing is adequate.

PUNCHING SHEAR FAILURE
The punching shear failure message will appear when the footing fails in punching shear. Increase the depth of footing until the shear strength of the footing is adequate.
FOOTING WEIGHT EXCEEDS BEARING PRESSURE
The footing weight exceeds bearing pressure message will appear when the weight of the footing exceeds the allowable soil bearing pressure. Increase the allowable bearing pressure and/or decrease the depth of the footing.

BEARING INADEQUATE (SQUARE/RECTANGULAR/ROUND FOOTING ONLY)
The bearing inadequate message will appear when the baseplate is not sufficient to transfer the column load into the footing without concrete compression failure. Increase the baseplate length and/or width until the baseplate adequately transfers the load into the footing.

Note: StruCalc does not check the adequacy of the baseplate thickness the baseplate itself must be designed by the user.

FAILURE IN BENDING
The failure in bending message will appear when the tension reinforcement is not adequate. This is either the result of insufficient strength, a calculated spacing smaller than the code minimum, or less than the minimum amount of steel for code. Increase the steel size until the reinforcement is adequate.

DEVELOPMENT LENGTH INADEQUATE
If the reinforcement development length is inadequate then increase the development length by increasing the footing size. StruCalc will not display this message if the footing would be adequate in bending without any reinforcement.

PLAIN CONCRETE FLEXURE FAILURE
If the plain concrete checkbox is checked and the plain concrete flexure failure message appears then the footing is failing in bending due to a tension failure in the concrete. Either increase the depth or add reinforcement.

11.2 CALCULATIONS

11.2.1 GENERAL CALCULATIONS

EFFECTIVE SOIL BEARING CAPACITY
Effective soil bearing capacity (Qe) is determined by reducing the allowable soil bearing capacity by the weight of the footing and the soil overburden weight (when applicable).

\[
Q_e = Q_s - \left( \frac{D}{12\text{in./ft}} \right) \left( 150\text{lb/ft}^3 \right) + (\text{SoilH} \cdot \text{SoilW})
\]

SoilW = The weight of the soil, in pounds per cubic foot.
SoilH = The depth of the soil on top of the footing, in feet.

Note: The concrete weight used by the program is 150 pcf.

LOADING
The total load (TL) and the ultimate factored load (Pu) are calculated as shown:

\[
TL = LL + DL
\]
Pu = \( (1.6 \cdot LL) + (1.2 \cdot DL) \) \text{ ACI}^{7,18,19} (9.2)

**FOOTING SIZE**

The footing area:

\[
A = W \cdot L
\]

Required footing area:

\[
\text{Areq} = \frac{TL}{Qe}
\]

Minimum footing size based on area:

\[
\text{Lreq} = \sqrt{\text{Areq}}
\]

**FOOTING DEPTH**

The effective footing depth is calculated based on footing depth, concrete cover, and reinforcement thickness. Note that the effective depth is based on the upper layer of reinforcement. See figure 10-3.

\[
d = \text{Depth} - c - (1.5 \cdot t)
\]

Reinforcement size, in inches:

\[
t = \frac{\text{Bar size}}{8}
\]

Effective footing depth for plain footings, in inches:

\[
d = \text{Depth} - 2
\]

**11.2.2 BEARING CALCULATIONS (SQUARE/RECTANGULAR FOOTING ONLY)**

**CONCRETE BEARING STRENGTH**

The bearing capacity of concrete is calculated per ACI^{7,18,19} (10.15, 10.17, 10.17).

\[
Y = \phi \cdot 0.85 \cdot fc' \cdot (m \text{ or bsw}) \cdot (n \text{ or bsl}) \cdot X
\]

Where \( \phi \) is equal to 0.65 for reinforced footings and plain concrete footings. The multiplier \( X \) accounts for the difference between the loaded area and the concrete area and cannot be greater than 2 or less than 1.

\[
X = \sqrt{\frac{A \cdot \left(\frac{144 \text{in}^2}{\text{ft}^2}\right)}{(m \text{ or bsw}) \cdot (n \text{ or bsl})}}
\]

The concrete bearing capacity is compared to the ultimate factored load to determine adequacy.

\( Pu \leq Y \) for adequacy
11.2.3 **PUNCHING SHEAR CALCULATIONS (SQUARE/RECTANGULAR FOOTING ONLY)**

**CRITICAL PERIMETER**

Critical perimeter \((Bo)\) is used to calculate the average perimeter length upon which the concrete will fail due to the punching shear. See points \(qrst\) on figure 10-4.

\[
Bo = [2(m + F + d)]+ [2(n + F + d)]ACI^{7,18,19} \quad (11.12.1.2)
\]

**PUNCHING SHEAR**

Punching shear:

\[
Vu_1 = Qu \cdot [A - Acp]
\]

Where \(Acp\) is the area bounded by the critical perimeter and is located along points \(qrst\) in figure 10-4.

\[
Acp = ((m + F + d) \cdot (n + F + d)) \cdot \left(\frac{ft^2}{144in^2}\right)
\]

Punching shear stress:

\[
uu_1 = \frac{Vu_1}{\phi \cdot Bo \cdot d}
\]

Where \(\phi\) is the strength reduction factor and is equal to 0.75 for reinforced footings and plain concrete footings.

**ALLOWABLE PUNCHING SHEAR STRESS**

\(vc_1\) is equal to the smallest of \(vc_{1_a}\), \(vc_{1_b}\), and \(vc_{1_c}\):

\[
vc_{1_a} = \left(2 + \frac{4}{\beta_c}\right)\sqrt{fc'} \quad ACI^{7,18,19} \quad (11.12.2.1-a)
\]

Where \(\beta_c\) is the ratio of the long side to the short side of the column baseplate.

\[
vc_{1_b} = \left(\frac{\alpha_s \cdot d}{Bo} + 2\right)\sqrt{fc'} \quad ACI^{7,18,19} \quad (11.12.2.1-b)
\]

Where \(\alpha_s\) is 40 for interior columns.

\[
vc_{1_c} = 4 \cdot \sqrt{fc'} \quad ACI^{7,18,19} \quad (11.12.2.1-c)
\]

**PUNCHING SHEAR ADEQUACY**

The punching shear stress is compared to the allowable punching shear stress to determine adequacy.

\[Vu_1 \leq vc_1\] for adequacy
11.2.4 BEAM SHEAR CALCULATIONS

BEAM SHEAR STRESS

Beam shear:

\[ Vu_2 = Qu \cdot W \cdot E \]

Where \( E \) is the width of the critical section for beam shear. The area that is used to determine the beam shear is bounded by points \( novu \) in figure 10-4. Note that the least dimension of a rectangular column dimension is used to determine the maximum beam shear.

\[ E = \frac{L}{2} - \left( \frac{\text{min. of } m \text{ or } n}{2} \right) - (d + F / 2) \]

Beam shear stress:

\[ vu_2 = \frac{Vu_2}{\phi \cdot L \cdot d \left( \frac{12\text{in.}}{\text{ft}} \right)} \]

Where \( \phi \) is the strength reduction factor and is equal to 0.75 for reinforced footings and plain concrete footings.

ALLOWABLE BEAM SHEAR STRESS

\[ vc_2 = 2 \cdot \sqrt{f_{ct}} \quad ACI^{7,18,19} (11.3.1.1) \]

BEAM SHEAR ADEQUACY

The beam shear stress is compared to the allowable beam shear stress to determine adequacy.

\[ vu_2 \leq vc_2 \text{ for adequacy} \]

11.2.5 REINFORCEMENT CALCULATIONS

STEEL REQUIRED BASED ON MOMENT

The footing strength, factored moment is \((Mu)\) in in-lbs. The bending moment is calculated by summing the moments about a plane which is projected along the face of a concrete pedestal, column, or wall, halfway between the face of a column and steel baseplate, halfway between the middle and edge of a masonry wall, or the center of the footing for a wood wall. See figure 11-4 for an example of a footing with steel column/baseplate. The soil pressure exerted against the footing base causes the moment and the area acting is located around points \(yxpm\).

For example, the bending moment for a steel column with baseplate is:

\[ Mu = Qu \cdot W \left( \frac{L}{2} - \left( \frac{\text{min. of } m \text{ or } n + F}{24} \right) \right)^2 \cdot 6 \]

Minimum steel required to resist the bending moment stresses is:
$$As(1) = \frac{Mu \cdot (12 \text{in} / \text{ft})}{0.85 \cdot Fy \cdot \left( d - \frac{a}{2} \right)}$$

Where:

- $a$ = the concrete compressive block depth.

The depth of the equivalent rectangular compressive stress block is:

$$a = \frac{As(1) \cdot fy}{0.85 \cdot Fc' \cdot W \cdot (12 \text{in} / \text{ft})}$$

**MINIMUM STEEL REQUIREMENTS**

StruCalc allows the user to select the specific code requirements from which the minimum steel is calculated. To change the requirements open the Settings form by clicking the Settings button on the Analysis Toolbar and then choose Options from the tabs at the top of the form.

Minimum steel requirements based on ACI\textsuperscript{18,19} (10.5.1) shall not be less than:

$$As(2) = \frac{3\sqrt{f_{c'}}}{f_y} b_w \text{Depth}$$

And not less than:

$$As(2) = \frac{200 \cdot \text{Depth} \cdot b_w}{f_y}$$

Area of steel reinforcement based on ACI\textsuperscript{18,19} (10.5.4) shall provide at least the following ratios of reinforcement area to gross concrete area, but not less than .0014:

a) .0020 for Grade 40 or 50 deformed bars
b) .0018 for Grade 60 deformed bars are used
c) .0018 x 60,000/fy for reinforcement w/ a yield stress exceeding 60,000 psi measured at a yield strain of 0.35 percent

Note: The minimum steel requirements for footings is specified in ACI\textsuperscript{18,19} (10.5.4). The minimum steel requirements based on ACI\textsuperscript{18,19} (10.5.1) is a more conservative alternative based on flexure.

**SPACING REQUIREMENTS**

The minimum clear spacing is $d_b$ but not less than 1 inch per ACI\textsuperscript{7,18,19} section 7.6.1. Maximum spacing is 18 inches. $d_b$ is the diameter of reinforcement.

The bar spacing required to satisfy steel requirements is given to the nearest inch. For bar spacing less than 8 inches on center, the exact spacing is specified.

**REINFORCEMENT PROVIDED**

The controlling reinforcement required ($As_{\text{reqd}}$) is determined by taking the maximum of $As(1)$ and $As(2)$.

Selected reinforcement steel area, in square inches per foot:
BAND WIDTH REINFORCEMENT (RECTANGULAR FOOTINGS ONLY)
A portion of the total reinforcement equal to the equation below shall be distributed uniformly over the “center band width W” equal to the length of the short side “W” of the footing. See figure 11-2.

Reinforcement required in center band width is:

\[
\frac{\text{Reinforcement in band width}}{\text{Total reinforcement in short direction}} = \frac{2}{\beta + 1}
\]

Where \(\beta\) is the ratio of the length of the long side to the width of the short side of the footing.
The remainder of reinforcement that is required in the short direction is to be distributed uniformly in the outer bands.

11.2.6 REINFORCEMENT DEVELOPMENT LENGTH CALCULATIONS
The development length is the length of embedment required to develop the full strength of the reinforcing steel.

DEVELOPMENT LENGTH REQUIRED (ACI 318-89) (Revised 1992)
The basic development length in inches:

\[
L_d = \frac{0.04 \cdot A_b \cdot f_y}{\sqrt{f'_c}}
\]

ACI (12.2.2)

Where \(A_b\) is the area of selected reinforcement and \(\sqrt{f'_c}\) cannot exceed 100 psi.

The following adjustment factors are applied to the basic development length:

1. If spacing > (5 \( \cdot \) \(d_b\)) and cover (c) is > (2.5 \( \cdot \) \(d_b\)) then \(L_d\) is multiplied by 0.8.
2. If spacing > (3 \( \cdot \) \(d_b\)) and cover (c) is > (2 \( \cdot \) \(d_b\)) then \(L_d\) is equal to the basic development length.
3. If spacing < (2 \( \cdot \) \(d_b\)) or cover (c) is < \(d_b\) then \(L_d\) is multiplied by 2.
4. If the spacing and concrete cover do not fall into one of the specifications above then multiply \(L_d\) by 1.4.

The minimum development length after all adjustments are made must not be less than 12 inches or:

\[
L_d = \frac{0.03 \cdot d_b \cdot f_y}{\sqrt{f'_c}}
\]

ACI (12.2.3.6)

DEVELOPMENT LENGTH REQUIRED (ACI 318-95, 99, 02, 05)
The basic development length is determined from the equation below, but \(L_d\) shall not be less than 12 inches:
\[
\frac{f_d}{d_b} = \frac{3}{40} \frac{f_y}{f_c'} \left( \frac{\alpha \beta \gamma \lambda}{c + K_{tr}} \right)
\]

ACI\textsuperscript{18,19} (12.2.3)

Where:
- \(\alpha\) = the traditional reinforcement location factor.
- \(\beta\) = a coating factor reflecting the effects of epoxy coating.
- \(\gamma\) = the reinforcement size factor.
- \(\lambda\) = the lightweight concrete factor.
- \(K_{tr}\) = the transverse reinforcement index.

The term \((c + K_{tr})/d_b\) shall not be taken greater than 2.5.

**DEVELOPMENT LENGTH PROVIDED**

The development length of reinforcement is measured from the critical section (varies depending on wall or column type) that is established for moment calculations to the edge of the footing minus a minimum of one inch for concrete cover. See figure 11-3.

**11.2.7 PLAIN CONCRETE CALCULATIONS**

**STRENGTH IN BENDING**

Plain concrete design is generally controlled by concrete failure in tension due to bending moment. The section modulus of the footing is calculated based on the total footing depth.

\[
Sc = \frac{W \cdot (12\text{in./ft}) \cdot \text{Depth}^2}{6}
\]

The allowable moment capacity of the selected footing:

\[
M_n = \phi \cdot Sc \cdot 5 \cdot \sqrt{f_c'}
\]

Where \(\phi\) is equal to 0.55.
12. COLLAR TIE

12.1 INPUTS AND DESCRIPTION

DESCRIPTION
The collar tie module designs a double rafter and collar tie system. It checks the rafters for bending, shear and deflection and the collar tie for tension as well as specifying nailing requirements.

LOADING
The loading on the collar tie system is just the vertical live and dead loads on the rafters inputted in pounds per square foot. There is an option to design the rafters with a double live load on the cantilever portion of the rafters. This is required by some codes to account for the possibility of an ice dam forming on the eaves. See section 5.2.7 for commentary on roof pitch adjustments.

COLLAR TIE
The collar tie can be either solid sawn or composite lumber. Choose the material type and the size of the collar tie in the collar tie input section below the collar tie input diagram. The collar tie is designed for tension only and the tension capacity is set in the Settings under the Options tab.
The collar tie is positioned between the top plate and the bottom of the rafters at the peak of the roof. You can automatically lock it to the bottom plate by checking the Lock Collar Tie to Top Plate checkbox.

**BRACING**
The rafters can be braced above and below the collar tie by checking the two checkboxes on the main form. Depending on the length of the rafters this may be necessary due to a negative moment in the rafters.

**DESIGN PROCESS**
Tension in the collar tie rarely controls the design of the collar tie/rafter system. In most cases it is the bending moment in the rafters at the location of the collar tie that controls the connection design. The design is a trial and error process, use different rafter sizes and spacing and move the collar tie up and down until an adequate situation is achieved.

**CONNECTION DESIGN**
StruCalc will automatically provide nailing requirements for three different types of 16d nails. Note that if the rafter or collar tie is wider than 1 ¾" the nailing requirement is not valid and will not be shown. Bolts can be used but must be hand calculated to determine that adequate edge spacing, bolt spacing and size are provided. Note that the number of nails shown may not be

**12.2 COLLAR TIE CALCULATIONS/ASSUMPTIONS**
The moment, shear, and deflection as well as the tension in the collar tie are determined using basic statics. The maximums are determined and the design calculations are done based on chapter 6.

The location for all calculations is based on the top of the rafter and the center line of the collar tie. It is assumed that the walls do not provide any horizontal support for the rafters and all of the horizontal resistance to movement is provided by the tension in the collar tie and compression at the ridge.
### APPENDIX A- SOLID SAWN SPECIES CLASSIFICATIONS

<table>
<thead>
<tr>
<th>Species combination</th>
<th>Species that are Included in the Combination</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alaska Spruce</td>
<td>Alaska Sitka Spruce</td>
</tr>
<tr>
<td></td>
<td>Alaska White Spruce</td>
</tr>
<tr>
<td>Aspen</td>
<td>Big Tooth Aspen</td>
</tr>
<tr>
<td>Quaking Aspen</td>
<td></td>
</tr>
<tr>
<td>Beech-Birch-Hickory</td>
<td>American Beech</td>
</tr>
<tr>
<td>Bitternut Hickory</td>
<td></td>
</tr>
<tr>
<td>Mockernut Hickory</td>
<td></td>
</tr>
<tr>
<td>Nutmeg Hickory</td>
<td></td>
</tr>
<tr>
<td>Pecan Hickory</td>
<td></td>
</tr>
<tr>
<td>Pignut Hickory</td>
<td></td>
</tr>
<tr>
<td>Shagbark Hickory</td>
<td></td>
</tr>
<tr>
<td>Shellbark Hickory</td>
<td></td>
</tr>
<tr>
<td>Sweet Birch</td>
<td></td>
</tr>
<tr>
<td>Water Hickory</td>
<td></td>
</tr>
<tr>
<td>Yellow Birch</td>
<td></td>
</tr>
<tr>
<td>Douglas Fir-Larch</td>
<td>Douglas Fir</td>
</tr>
<tr>
<td></td>
<td>Western Larch</td>
</tr>
<tr>
<td>Douglas Fir-Larch (North)</td>
<td>Douglas Fir</td>
</tr>
<tr>
<td></td>
<td>Western Larch</td>
</tr>
<tr>
<td>Eastern Hemlock- Balsam Fir</td>
<td>Balsam Fir</td>
</tr>
<tr>
<td></td>
<td>Eastern Hemlock</td>
</tr>
<tr>
<td>Tamarack</td>
<td></td>
</tr>
<tr>
<td>Eastern Hemlock-Tamarack</td>
<td>Eastern Hemlock</td>
</tr>
<tr>
<td></td>
<td>Tamarack</td>
</tr>
<tr>
<td>Eastern Softwoods</td>
<td>Balsam Fir</td>
</tr>
<tr>
<td></td>
<td>Black Spruce</td>
</tr>
<tr>
<td></td>
<td>Eastern Hemlock</td>
</tr>
<tr>
<td></td>
<td>Eastern White Pine</td>
</tr>
<tr>
<td></td>
<td>Jack Pine</td>
</tr>
<tr>
<td></td>
<td>Norway (Red) Pine</td>
</tr>
<tr>
<td></td>
<td>Pitch Pine</td>
</tr>
<tr>
<td></td>
<td>Red Spruce</td>
</tr>
<tr>
<td></td>
<td>Tamarack</td>
</tr>
<tr>
<td></td>
<td>White Spruce</td>
</tr>
<tr>
<td>Eastern Spruce</td>
<td>Black Spruce</td>
</tr>
<tr>
<td></td>
<td>Red Spruce</td>
</tr>
<tr>
<td></td>
<td>White Spruce</td>
</tr>
<tr>
<td>Hem-Fir</td>
<td>California Red Fir</td>
</tr>
<tr>
<td>Grand Fir</td>
<td></td>
</tr>
<tr>
<td>Noble Fir</td>
<td></td>
</tr>
<tr>
<td>Pacific Silver Fir</td>
<td>Amabilis Fir</td>
</tr>
<tr>
<td>--------------------</td>
<td>-------------</td>
</tr>
<tr>
<td>Western Hemlock</td>
<td>Western Hemlock</td>
</tr>
<tr>
<td>White Fir</td>
<td></td>
</tr>
</tbody>
</table>

| Hem-Fir (North)    | All Oak Species |
|--------------------|-----------------
| Mixed Maple        | Black Maple    |
| Red Maple          |                |
| Silver Maple       |                |
| Sugar Maple        |                |

| Mixed Oak          |                |
| Mixed Southern Pine|                |
| All species in the Southern Pine | Species Combination plus |
| Pond Pine          |                |
| Virginia Pine      |                |

<table>
<thead>
<tr>
<th>Northern Pine</th>
<th>Jack Pine</th>
</tr>
</thead>
<tbody>
<tr>
<td>Northern Red Oak</td>
<td>Black Oak</td>
</tr>
<tr>
<td>Scarlet Oak</td>
<td>Pin Oak</td>
</tr>
<tr>
<td>Northern Species</td>
<td>Any Species graded under NLGA rules except</td>
</tr>
<tr>
<td></td>
<td>Red Alder, White Birch, and Norway Spruce</td>
</tr>
</tbody>
</table>

| Red Oak            | Black Oak |
| Cherrybark Oak     |            |
| Laurel Oak         |            |
| Northern Red Oak   |            |
| Pin Oak            |            |
| Scarlet Oak        |            |
| Southern Red Oak   |            |
| Water Oak          |            |
| Willow Oak         |            |

<table>
<thead>
<tr>
<th>Southern Pine</th>
<th>Loblolly Pine</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shortleaf Pine</td>
<td>Longleaf Pine</td>
</tr>
<tr>
<td>Slash Pine</td>
<td></td>
</tr>
</tbody>
</table>

| Spruce-Pine-Fir    | Alpine Fir |
| Balsam Fir         |            |
| Black Spruce       |            |
| Engelmann Spruce   |            |
| Jack Pine          |            |
| Lodgepole Pine     |            |
| Red Spruce         |            |
| White Spruce       |            |

| Spruce-Pine-Fir (South) | Balsam Fir |
| Black Spruce           |            |

100
Jack Pine
Norway (Red) Pine
Red Spruce
White Spruce
Engelmann Spruce
Lodgepole Pine
Sitka Spruce

Western Cedars
Incense Cedar
Port Orford Cedar
Western Red Cedar
Western Cedars (North)

Western Woods
(South) Species Combination plus
Alpine Fir
Idaho White Pine
Mountain Hemlock
Ponderosa Pine
Sugar Pine

White Oak

Alaska Cedar
Pacific Coast Yellow Cedar
Western Red Cedar
All Species in the Douglas Fir-Larch, Douglas Fir-South, Hem Fir, and Spruce-Pine-Fir
Bur Oak

Chestnut Oak
Live Oak
Overcup Oak
Post Oak
Swamp Chestnut Oak
Swamp White Oak
White Oak
APPENDIX B-BOLTING AND NAILING REQUIREMENTS FOR LAMINATED COLUMNS

The calculations provided by StruCalc are applicable only if the following specifications are followed during the fabrication of the built-up column. The bolting and nailing apply to columns with 2 to 5 laminations that meet the following criteria:

1. all laminations are 1 1/2” or greater in width (StruCalc only allows 1 1/2” laminated members)
2. all laminations have the same depth d
3. the faces of adjacent laminations are in contact
4. all laminations are the full column length

Also note that when individual laminations are of a different species select the species and grade that provide the weakest allowable compression parallel to grain and modulus of elasticity.

NAILED LAMINATED COLUMNS
A properly nailed built up member is laminated using the following specifications:

1. adjacent nails are driven from opposite sides of the column.
2. all nails penetrate at least 3/4 of the thickness of the last lamination
3. $15D \leq \text{End distance} \leq 18D$
4. $20D \leq \text{spacing between adjacent nails in a row} \leq 6w$
5. $10D \leq \text{spacing between rows of nails} \leq 20D$
6. $5D \leq \text{edge distance} \leq 20D$
7. 2 or more longitudinal rows of nails are required when $d > 3w$

Where:

$D =$ nail diameter
$d =$ depth of an individual lamination
$w =$ width of an individual lamination

When only one longitudinal row of nails is required, adjacent nails are to be staggered. When 3 or more longitudinal rows are required then nails in adjacent rows are to be staggered.

BOLTED LAMINATED COLUMNS
A properly bolted built up member is laminated using the following specifications:

1. A washer or metal plate is provided between the wood and the bolt head, and between the wood and the nut.
2. nuts are tightened to ensure that faces of adjacent laminations are in contact
3. for softwoods: $7D \leq \text{end distance} \leq 8.4D$
4. for hardwoods: $5D \leq \text{end distance} \leq 6D$
5. $4D \leq \text{spacing between adjacent bolts} \leq 6w$
6. $1.5D \leq \text{spacing between rows of bolts} \leq 10D$
7. $1.5D \leq \text{edge distance} \leq 10D$
8. 2 or more longitudinal rows of bolts are required when $d > 3w$
Where:
D = bolt diameter
\( d \) = depth of an individual lamination
\( w \) = width of an individual lamination
## APPENDIX C - NOMENCLATURE

StruCalc uses standard engineering nomenclature when possible:

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Area of member, in square inches.</td>
</tr>
<tr>
<td>Acp</td>
<td>Area bounded by the critical perimeter on footing.</td>
</tr>
<tr>
<td>Af</td>
<td>Flange area, in square inches.</td>
</tr>
<tr>
<td>AFBR</td>
<td>Allowable flange buckling ratio.</td>
</tr>
<tr>
<td>Ai</td>
<td>Influence area in square feet.</td>
</tr>
<tr>
<td>As</td>
<td>Reinforcement steel area, in square inches.</td>
</tr>
<tr>
<td>As_reqd</td>
<td>Required steel reinforcement area, in square inches.</td>
</tr>
<tr>
<td>Areq</td>
<td>The area required for adequacy, in square inches.</td>
</tr>
<tr>
<td>Areq</td>
<td>Footing area required, in square feet.</td>
</tr>
<tr>
<td>AWBR</td>
<td>Allowable web buckling ratio.</td>
</tr>
<tr>
<td>BA</td>
<td>Beam angle, in degrees.</td>
</tr>
<tr>
<td>BDL'</td>
<td>Beam dead load acting over span L.</td>
</tr>
<tr>
<td>bf</td>
<td>Flange width, in inches.</td>
</tr>
<tr>
<td>BL</td>
<td>Bearing length, in inches.</td>
</tr>
<tr>
<td>BLL'</td>
<td>Beam live load acting over span L.</td>
</tr>
<tr>
<td>Bo</td>
<td>Critical perimeter for punching shear calculations, in inches.</td>
</tr>
<tr>
<td>bsl</td>
<td>Baseplate length, in inches.</td>
</tr>
<tr>
<td>bsw</td>
<td>Baseplate width, in inches.</td>
</tr>
<tr>
<td>BSW</td>
<td>Beam self weight, in pounds per linear foot.</td>
</tr>
<tr>
<td>c</td>
<td>Location of centroid.</td>
</tr>
<tr>
<td>c</td>
<td>Concrete reinforcement cover, in inches.</td>
</tr>
<tr>
<td>CAF</td>
<td>Camber amplification factor</td>
</tr>
<tr>
<td>C</td>
<td>The camber required, in inches.</td>
</tr>
<tr>
<td>Cb</td>
<td>Bending coefficient for steel design.</td>
</tr>
<tr>
<td>Cc</td>
<td>Column slenderness ratio separating elastic and inelastic buckling.</td>
</tr>
<tr>
<td>Cd</td>
<td>Duration factor adjustment.</td>
</tr>
<tr>
<td>Cf</td>
<td>Size factor adjustment.</td>
</tr>
<tr>
<td>Cf_Fb'</td>
<td>Size factor adjusted bending stress, in psi.</td>
</tr>
<tr>
<td>Ci</td>
<td>Incising factor adjustment.</td>
</tr>
<tr>
<td>Ck</td>
<td>Limiting slenderness factor.</td>
</tr>
<tr>
<td>Cl</td>
<td>Stability adjustment factor.</td>
</tr>
<tr>
<td>Cm</td>
<td>Wet use adjustment factor.</td>
</tr>
<tr>
<td>Cp</td>
<td>Column stability factor.</td>
</tr>
<tr>
<td>Cr</td>
<td>Repetitive member factor.</td>
</tr>
<tr>
<td>Cs</td>
<td>Slenderness adjustment factor.</td>
</tr>
<tr>
<td>Cs_Fb'</td>
<td>Slenderness adjusted bending stress, in psi.</td>
</tr>
<tr>
<td>CSF</td>
<td>Combined stress factor.</td>
</tr>
<tr>
<td>Cv</td>
<td>Volume factor adjustment.</td>
</tr>
<tr>
<td>Cv</td>
<td>Web stress to shear yield stress ratio for steel design.</td>
</tr>
<tr>
<td>d</td>
<td>Beam depth, in inches.</td>
</tr>
<tr>
<td>d</td>
<td>Effective footing depth, in inches.</td>
</tr>
<tr>
<td>Depth</td>
<td>Footing depth, in inches.</td>
</tr>
<tr>
<td>d_s</td>
<td>Reinforcement bar diameter, in inches.</td>
</tr>
<tr>
<td>DL</td>
<td>Dead load, in psf.</td>
</tr>
<tr>
<td>DLD</td>
<td>Dead load deflection, in inches.</td>
</tr>
<tr>
<td>dx</td>
<td>Column depth, in inches.</td>
</tr>
<tr>
<td>dy</td>
<td>Column width, in inches.</td>
</tr>
</tbody>
</table>

105
ex  Column end eccentricity about the X-axis, in inches.
ey  Column end eccentricity about the Y-axis, in inches.
E    Modulus of elasticity, in psi.
E'   Adjusted Modulus of elasticity, psi.
EL   Beam end elevation difference.
Ex   Modulus of elasticity about the X-axis, in psi.
Ey   Modulus of elasticity about the Y-axis, in psi.
fa   Actual column compressive stress, in psi.
fa   Allowable axial compressive stress for steel, in psi.
Fb   Allowable bending stress, in psi.
Fb'  Adjusted allowable bending stress, in psi.
Fb_cpr Allowable bending stress of the compression face for glulams, in psi.
Fbe  Critical buckling design value, in psi.
Fbx  Allowable column bending stress about the X-axis, in psi.
Fbx' Allowable bending stress about the X-axis with adjustment values applied.
Fbx' Allowable bending stress about the X-axis, in psi.
Fby  Allowable column bending stress about the Y-axis, in psi.
Fby' Allowable bending stress about the Y-axis with adjustment values applied.
Fby' Column bending stress about the Y-axis, in psi.
FBR  Flange buckling ratio.
fC   Compressive stress due to pure axial loading for wood, in psi.
fC'  Concrete compressive strength, in psi.
fC   Compressive stress parallel to grain.
fC'  Adjusted allowable compressive stress, in psi.
fCe  Critical buckling design value for compression members, psi.
fCe  Critical buckling design value for compression members about the X-axis, psi.
fCe  Critical buckling design value for compression members about the Y-axis, psi.
fce  Critical buckling design value for compression members about the X-axis, psi.
fce  Critical buckling design value for compression members about the Y-axis, psi.
fC_perp' Adjusted compressive stress perpendicular to the grain, in psi.
fC_perp' Adjusted compressive stress perpendicular to the grain, in psi.
fex  Euler's stress in the X-direction used for combined stress calculations.
fey  Euler's stress in the Y-direction used for combined stress calculations.
FLA  Floor loaded area, in square feet.
fy   Steel reinforcing yield strength, in psi.
fV   Horizontal shear stress, in psi.
fV'  Factored horizontal shear stress, in psi.
fY   Minimum steel yield stress, in psi.
h   Web clear height, in inches.
h_tw Web height to thickness ratio.
h_tw-limit Limiting web height to thickness ratio.
l   Moment of inertia, in inches^4.
lx   Moment of inertia about the X-axis, in inches^4.
ly   Moment of inertia about the Y-axis, in inches^4.
k   Distance from the outside edge of the flange to the toe fillet, in inches.
K    Column end condition for wood design.
kbe  Euler buckling coefficient for beams.
kce  Euler buckling coefficient for columns.
ke   Column end condition for steel design.
kf   Column stability coefficient for bolted and nailed built-up columns.
kl   The loading condition coefficient.
Kix/rx Column slenderness ratio about the X-axis for steel.
Kly/ry Column slenderness ratio about the Y-axis for steel.
ktr  Transverse reinforcement index.
kv   Shear coefficient.
L    Beam span from center of support to center of support, in feet.
L    Footing length, in feet.
<table>
<thead>
<tr>
<th>Term</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ladj</td>
<td>Beam span adjusted for beam slope, in feet.</td>
</tr>
<tr>
<td>Lb</td>
<td>Unbraced length.</td>
</tr>
<tr>
<td>Lc</td>
<td>Limiting unbraced length for steel design, in feet.</td>
</tr>
<tr>
<td>Ld</td>
<td>Reinforcement development length required, in inches.</td>
</tr>
<tr>
<td>Ld-prov</td>
<td>Reinforcement development length provided, in inches.</td>
</tr>
<tr>
<td>Le</td>
<td>Effective unbraced length, in inches.</td>
</tr>
<tr>
<td>Lex/dx</td>
<td>Column compression slenderness ratio about the X-axis.</td>
</tr>
<tr>
<td>Ley/dy</td>
<td>Column compression slenderness ratio about the Y-axis.</td>
</tr>
<tr>
<td>LL</td>
<td>Live load, in psf.</td>
</tr>
<tr>
<td>Lu</td>
<td>Unbraced length, in feet.</td>
</tr>
<tr>
<td>Lx</td>
<td>Unbraced length about the X-axis, in feet.</td>
</tr>
<tr>
<td>Ly</td>
<td>Unbraced length about the Y-axis, in feet.</td>
</tr>
<tr>
<td>LLavg</td>
<td>Average live load, in pounds per linear foot.</td>
</tr>
<tr>
<td>LLD</td>
<td>Live load deflection, in inches.</td>
</tr>
<tr>
<td>LLD_max</td>
<td>Live load deflection limit, in inches.</td>
</tr>
<tr>
<td>LLD_ratio</td>
<td>The actual live load deflection ratio.</td>
</tr>
<tr>
<td>LLD_criteria</td>
<td>The inputted allowable live load deflection limit.</td>
</tr>
<tr>
<td>Lreq</td>
<td>Footing length required for adequacy, in feet.</td>
</tr>
<tr>
<td>m</td>
<td>Column width, in inches.</td>
</tr>
<tr>
<td>M</td>
<td>The moment in the beam, in ft-lb.</td>
</tr>
<tr>
<td>Mx</td>
<td>Moment about the X-axis due to eccentricity, in in-lbs.</td>
</tr>
<tr>
<td>My</td>
<td>Moment about the Y-axis due to eccentricity, in in-lbs.</td>
</tr>
<tr>
<td>Mn</td>
<td>Moment capacity of a steel beam, in ft-lb.</td>
</tr>
<tr>
<td>Mu</td>
<td>Ultimate factored moment for footing calculations, in in-lb.</td>
</tr>
<tr>
<td>n</td>
<td>Column depth, in inches.</td>
</tr>
<tr>
<td>nd</td>
<td>Beam notch depth, in inches.</td>
</tr>
<tr>
<td>PD</td>
<td>Point dead load, in pounds.</td>
</tr>
<tr>
<td>PL</td>
<td>Point live load, in pounds.</td>
</tr>
<tr>
<td>PT</td>
<td>Point total load, in pounds.</td>
</tr>
<tr>
<td>Pu</td>
<td>Ultimate factored load for footing design, in pounds.</td>
</tr>
<tr>
<td>Qe</td>
<td>Effective soil bearing pressure, in psf.</td>
</tr>
<tr>
<td>Qs</td>
<td>Allowable soil bearing pressure, in psf.</td>
</tr>
<tr>
<td>Qu</td>
<td>Ultimate soil bearing pressure, in psf.</td>
</tr>
<tr>
<td>r</td>
<td>Rate of reduction from the minimum roof live loads table.</td>
</tr>
<tr>
<td>RA</td>
<td>The roof angle.</td>
</tr>
<tr>
<td>Rb</td>
<td>Slenderness ratio.</td>
</tr>
<tr>
<td>RD</td>
<td>Reaction dead load, in pounds.</td>
</tr>
<tr>
<td>RL</td>
<td>Reaction live load, in pounds.</td>
</tr>
<tr>
<td>RLA</td>
<td>Roof loaded area, in square feet.</td>
</tr>
<tr>
<td>RLL</td>
<td>Roof live load.</td>
</tr>
<tr>
<td>RT</td>
<td>Reaction total load, in pounds.</td>
</tr>
<tr>
<td>rt</td>
<td>Radius of gyration of the flange and 1/3 of the web, in inches.</td>
</tr>
<tr>
<td>RP</td>
<td>Roof pitch.</td>
</tr>
<tr>
<td>rx</td>
<td>Radius of gyration about the X-axis.</td>
</tr>
<tr>
<td>ry</td>
<td>Radius of gyration about the Y-axis.</td>
</tr>
<tr>
<td>S</td>
<td>Section modulus, in inches$^3$.</td>
</tr>
<tr>
<td>Sc</td>
<td>Section modulus of concrete, in inches$^3$.</td>
</tr>
<tr>
<td>Sreq</td>
<td>The required section modulus for adequacy, in inches$^3$.</td>
</tr>
<tr>
<td>Sx</td>
<td>Section modulus about the X-axis, in inches$^3$.</td>
</tr>
<tr>
<td>Sy</td>
<td>Section modulus about the Y-axis, in inches$^3$.</td>
</tr>
<tr>
<td>t</td>
<td>Reinforcement size, in inches.</td>
</tr>
<tr>
<td>TDL</td>
<td>Total dead load.</td>
</tr>
<tr>
<td>tf</td>
<td>Flange thickness, in inches.</td>
</tr>
<tr>
<td>TLD</td>
<td>Total load deflection, in inches.</td>
</tr>
<tr>
<td>TLD_max</td>
<td>Total load deflection limit, in inches.</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
</tr>
<tr>
<td>TLD&lt;sub&gt;ratio&lt;/sub&gt;</td>
<td>The actual total load deflection ratio.</td>
</tr>
<tr>
<td>TLD&lt;sub&gt;criteria&lt;/sub&gt;</td>
<td>The inputted allowable total load deflection limit.</td>
</tr>
<tr>
<td>TL</td>
<td>Total load.</td>
</tr>
<tr>
<td>TLL</td>
<td>Total live load.</td>
</tr>
<tr>
<td>tw</td>
<td>Web thickness, in inches.</td>
</tr>
<tr>
<td>TW</td>
<td>Tributary width, in feet.</td>
</tr>
<tr>
<td>UTWeq</td>
<td>Upper equivalent tributary load, in feet.</td>
</tr>
<tr>
<td>LTWeq</td>
<td>Lower equivalent tributary load, in feet.</td>
</tr>
<tr>
<td>V</td>
<td>Beam shear, in pounds.</td>
</tr>
<tr>
<td>vc1</td>
<td>Allowable punching shear stress, in psi.</td>
</tr>
<tr>
<td>Vn</td>
<td>Shear capacity of a steel members, in pounds.</td>
</tr>
<tr>
<td>Vu1</td>
<td>Punching shear, in pounds.</td>
</tr>
<tr>
<td>vu1</td>
<td>Punching shear stress, in psi.</td>
</tr>
<tr>
<td>Vu2</td>
<td>Beam shear, in pounds.</td>
</tr>
<tr>
<td>vu2</td>
<td>Beam shear stress, in psi.</td>
</tr>
<tr>
<td>vc2</td>
<td>Allowable beam shear stress, in psi.</td>
</tr>
<tr>
<td>W</td>
<td>Footing width, in feet.</td>
</tr>
<tr>
<td>WBR</td>
<td>Web buckling ratio.</td>
</tr>
<tr>
<td>wL</td>
<td>Uniform live load, in pounds per linear foot.</td>
</tr>
<tr>
<td>wD</td>
<td>Uniform dead load, in pounds per linear foot.</td>
</tr>
<tr>
<td>wT</td>
<td>Uniform total load, in pounds per linear foot.</td>
</tr>
<tr>
<td>Wt</td>
<td>Tabulated weight of the beam, in pounds per linear foot.</td>
</tr>
<tr>
<td>wTcont</td>
<td>Controlling uniform load, in pounds per linear foot.</td>
</tr>
<tr>
<td>X</td>
<td>Distance from the left support, in feet.</td>
</tr>
<tr>
<td>Y</td>
<td>Bearing capacity of concrete, in psi.</td>
</tr>
<tr>
<td>α</td>
<td>The traditional reinforcement location factor.</td>
</tr>
<tr>
<td>β</td>
<td>A coating factor reflecting the effects of epoxy coating.</td>
</tr>
<tr>
<td>γ</td>
<td>Reinforcement size factor.</td>
</tr>
<tr>
<td>λ</td>
<td>Lightweight concrete factor.</td>
</tr>
</tbody>
</table>
REFERENCES


7. American Concrete Institute: Building Code Requirements for Reinforced Concrete ACI 318-89 (Revised 1992), 1992, 2005


18. American Concrete Institute: Building Code Requirements for Reinforced Concrete ACI 318-95, 1995

19. American Concrete Institute: Building Code Requirements for Reinforced Concrete ACI 318-99, 1999
INDEX

A
Adequacy, 52, 58
Adjusted bending stress. See Wood
Adjusted compression parallel to grain. See Wood
Adjusted compression perpendicular to grain. See Wood
Adjusted shear stress. See Wood
Analysis Toolbar, 26
Autosize, 21

B
Baseplate, 91
Bending Coefficient. See Column
Bending stress. See Steel or Wood
Building code. See Settings

C
Calculate, 22, 26
Calculator, 22
Camber, 46
Code required concentrated load. See Floor joists
Column
  Bending coefficient, 74
  Load combinations, 76
  Steel column design, 81
  Unbraced length, 73
  Wood column design, 77
Column End Condition, 74
Column stability factor, 79
Combined Stress Factor, 79
Company name. See Settings
Concrete cover. See Reinforcement
Copy projects, 25, See Project Manager

D
Deflection Criteria, 37
Deflection limit options for cantilevers, 38
Delete Member, 25, See Project Manager
Delete Project, 25, See Project Manager
Development Length, 87, 90, 95, 96
Duration Factor, 27, 38, 39, 47, 61, 62, 64, 65, 66, 67, 69, 79

E
Eccentricity, 74

F
Flange buckling ratio. See Steel flitch beam, 58
Floor joists
  Code required concentrated load, 69
Floor live load reduction. See Live Load
Footing
  Baseplate, 87
  Beam shear, 93
  Design calculations, 90
  Inadequacy, 89
  Minimum steel requirements, 94
  Plain concrete, 87, 96
  Punching shear, 92
  Reinforcement, 86, 93, 94
  Soil bearing pressure, 86

G
Grade, 21, 22, 26, 48, 79

I
I-Joists, 69, 70
Incising factor, 49

L
Laminated columns, 74
Lamination of members, 36
Live Load
  Floor live load reduction, 45
  Non-snow region, 39
  Snow region, 39
  Load combinations, 44, See Column
Loading, 44
Loading diagram, 22

M
Margins, 24
Material Properties, 47, 52, 86
Materials, 89
Member Toolbar, 26
MSR Lumber, 75

N
NDS (91,97), 47
Non-snow region. See Live Load

P
Plywood, 70
Printing, 23, 24, See Settings
Printing to file, 23
Printing to screen, 23
Project Manager, 23, 24, 25, 26
Q

Quickhelp, 21

R

Reinforcement. See Footing
Concrete cover, 87
Rename projects, 25, See Project Manager
Repetitive factor, 49
Roof Pitch, 40
round, 37

S

Saving, 24
Section properties. See Wood
Section Toolbar, 27
Settings
   Building code, 23
   Company name, 24
   Printing, 24
   User name, 24
shear, 51
Shear stress. See Steel
Shear, moment, and deflection diagrams, 22
Size factor, 47
Slope, beam, 40
Snow region. See Live Load
Soil bearing pressure. See Footing
Spacing, 36
Species, 22, 26, 48
Stability factor, 48
Steel
   Bending Stress, 54
   Flange buckling ratio, 53
   Shear Stress, 56
   Web buckling ratio, 53
   Web crippling, 58
   Web yielding, 57
   Stress values, 22, 47

T

Tributary Width, 37

U

Unbalanced Loads, 65
Unbraced length, 39, 40, 48, 49, 54, 55, See Column
Uplift factor of safety, 66
User name, 24, See Settings
User notes, 23

V

Volume factor, 48

W

Web buckling ratio. See Steel
Web crippling. See Steel
Web yielding. See Steel
Wet use factor, 49
Width/depth of members, 36
Wood
   Adjusted bending stress, 50
   Adjusted compression parallel to grain, 50
   Adjusted compression perpendicular to grain, 50
   Adjusted modulus of elasticity, 51
   Adjusted shear stress, 50
   Section properties, 51