Welcome to the 2005 NDS! See what’s new inside…
In this eCourse, you will see how the 2005 NDS is laid out, and then see what each chapter contains. Along the way, you'll see what has changed from previous editions of the document. And we’ll use worked examples to show how this document works for both ASD and LRFD processes.

There are a few changes and additions for the 2005 document.
In the 2005 NDS, chapters have maintained the same order as the 2001 NDS to provide a more comprehensive document for the design of wood products for building construction. Chapters are grouped in a logical fashion beginning with scoping/definitions/general provisions, then wood materials, connections, and special assemblies and provisions. The format is one that is currently being found in standards of other materials so that for the designer, ease of going from one standard to another is enhanced.
2005 NDS Appendix E maintains its importance for checking Local Stresses in Fastener Groups. Appendix N is the only mandatory appendix and is new. It contains all of the necessary tables to apply LRFD to the NDS.
The *NDS Supplement: Design Values for Wood Construction*, an integral part of the NDS, has also been updated to provide the latest design values for lumber and glued laminated timber.
Chapter 1 – Design Loads

• references loads in accordance with local codes, such as ASCE 7 – 02 or 05

Chapter 1 contains reference information to relevant load documents that are compatible with the 2005 NDS. This version of the NDS references both ASCE 7 – 02 or 05.

Chapter 1 also describes compatible design processes that the 2005 NDS uses: allowable stress design (ASD), and new for 2005, Load Resistance Factor Design (LRFD).
Overview of LRFD
…a digression

So what is LRFD? Here’s an overview.
Let’s begin with design process. The underlying basic philosophy for the process of structural design is that whatever demand is expected from a structural system must be met at least by its capacity.
The structural design process fundamentally breaks down into five key components. Others below the line are normally of secondary importance to safety and serviceability concerns. The demand features the type, magnitude, and placement of loads on the system and the resulting actions on interaction with the system’s formal geometry. The capacity of the system is provided in combination by the judicious choice of materials, section geometry, and an understanding of the way the system behaves under demand. The subject matter of this seminar will be dealing with the capacity side of the structural issue - featuring wood as the material.
LRFD Design Concepts

Two Limit State concerns:

• safety against failure or collapse
  want statistical protection against failure

• serviceability (performance in service)
  want real-world measurable behavior

A limit state is the point at which the structure fails to serve its intended purpose in some way. Two broad limit states can be identified for structures: safety, and serviceability.
LRFD - Serviceability

- Unfactored loads
- Mean material strength values

Serviceability limit states appraise the structure in terms of its everyday usefulness. For this reason, it is important to know how well the structure is actually performing. A way of seeing this, is to consider average material strength values in combination with real load magnitudes in the measure of actual performance.
LRFD - Safety

- Factored loads
- Material strength values

Safety on the other hand can be thought of in statistical terms - probability of failure, or conversely, survival. Using statistics, one can appraise the safety of a structure in terms of measurable probability. In the LRFD method, the tie to a statistical approach is achieved through the use of load factors and material reference strengths modified by reliability factors.
Let's look first at the capacity of side of the issue, specifically materials. Here is a symbolic representation of the structural property variability among a variety of wood products. The same statistical form shows up for all other building products as well. Plotted here is the relative frequency of occurrence against the actual property values from testing. Structural testing in specific modes is performed on these products to produce the data set that makes up these curves. Each curve (normal distributions shown here) can be described by its statistical measures: mean, standard deviation (a measure of the spread of the curve).
The normal distribution curve has the inherent property, that the area underneath it equals 1.0. This conveniently implies that the probability of occurrence equals 100%. From this, one can determine for example the structural property value that is appropriate for 5% of the sample population. It can also be determined how many standard deviations (the distance) it is away from the mean. Note that at the 5th% percentile, 5 percent of the samples fail at this property value, while 95 percent of the samples survive.
Let’s take two of these distributions: one for load (S), and one for resistance (R); and plot them together. Each of the curves has its own unique statistical description (mean and standard deviation values), and may or may not have the same distribution type. Normal distribution types are shown here, but there are many others, chosen to best fit or model the test sample data points. Note that the resistance curve is to the right of the load curve, and that curves overlap. The overlap implies the region where load is greater than resistance, hence failure.
The overlap, or failure zone, can be represented in a more useful way. If the load and resistance distributions respectively are normalized to the same type, then a performance distribution $Z$ can be created by subtracting the load distribution from the resistance distribution. The statistics of $Z$ are determined as seen in the slide, as well as $f_Z$ itself. In this plot, the area under the $f_Z$ distribution that falls in the region of property values less than zero, represents the probability of failure of the structure in this particular mode of testing. Now a measurable probability of failure is available. It can be further described in terms of the number of standard deviations away from the mean of the performance distribution. The greek letter $\beta$, known as the safety index, is used to describe this multiple. Thus, $\beta$ is directly tied to the probability of failure.
LRFD - Probability of Failure

\[ P_f = \text{one failure expected for } x \text{ number of structures designed and built with a given } \beta \]

<table>
<thead>
<tr>
<th>( \beta )</th>
<th>( P_f )</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.2</td>
<td>1 : 10,000,000</td>
</tr>
<tr>
<td>4.7</td>
<td>1 : 1,000,000</td>
</tr>
<tr>
<td>4.2</td>
<td>1 : 100,000</td>
</tr>
<tr>
<td>3.7</td>
<td>1 : 10,000</td>
</tr>
<tr>
<td>3.2</td>
<td>1 : 1,000</td>
</tr>
<tr>
<td>2.7</td>
<td>1 : 100</td>
</tr>
<tr>
<td>2.2</td>
<td>1 : 10</td>
</tr>
</tbody>
</table>

For large values of \( \beta \), the probability of failure is very small. For small \( \beta \) values, the probability of failure is much larger.
LRFD - Range on $\beta$

$\beta$ Range for Wood Strength

<table>
<thead>
<tr>
<th></th>
<th>Low</th>
<th>Typical</th>
<th>High</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\beta$</td>
<td>2.4</td>
<td>2.6</td>
<td>2.9</td>
</tr>
<tr>
<td>$P_f$</td>
<td>1 : 25</td>
<td>1 : 63</td>
<td>1 : 251</td>
</tr>
</tbody>
</table>

These are the typical $\beta$ values used in structural design in many materials, not just wood. It is interesting to note the corresponding probability of failure. These are levels that designers have been historically been designing buildings for.
How is $\beta$ actually used in design? $\beta$ is actually invisible in the design process. It is tied to two other factors: the reliability index $\phi$ (used on the capacity side of the equation), and the load factor $\alpha$ (used on the demand side of the equation). To design for any demand with any material to a target $\beta$, it is prudent to fix the value of the load factor $\alpha$ (standardized values for all materials), and derive reliability indices $\phi$ for various structural properties of various materials. This process is known as calibrating the reliability index.

Calibration needs to cover all of the relevant factors such as the load and variability of the member strength based on species, grade, and type of application. Generally, the 5th percentile of the strength data test data is used for the resistance side, while the load statistics are obtained from extensive studies of structures in all climatic zones and with different occupancies.
LRFD - Reliability Index Calibration

Example:

Bending strength resistance of 2x8 lumber subjected to Quebec City snow load

A calibration example: the bending strength of 2x8 lumber subjected to Quebec City snow load. What $\phi$ value would be appropriate for a target $\beta$ of 2.6?
LRFD - Reliability Index Calibration

Find range on reliability index $\phi$ such that for fixed factored loads, a target $\beta$ is achieved.

100 % data

In this $\beta$-$\phi$ ($\alpha$ fixed) correlation plot, the Quebec City snow load is modeled with a lognormal distribution, while the bending strength of 2x8 lumber is modeled with four different distributions that are fit as closely as possible to a complete data set of full-sized test results.

To give a target $\beta$ of 2.6, $\phi$ would range from 0.55 to 1.0 depending on which mathematical model is used for the resistance. This shows how sensitive $\beta$ is to the assumed distribution type.
LRFD - Reliability Index Calibration

Example:

Bending strength resistance of 2x8 lumber subjected to Quebec City snow load

Here is a cumulative probability plot of 2x8 bending strength. On the plot is the complete test data set of full-sized specimens (In-Grade) and two distribution models that are fit as closely as possible to the test data. The test data comes from the 5th percentile MOR’s.
Careful inspection of the test strength data reveals that, while the 100% distribution curve fits the complete data set reasonably well, the model doesn’t represent the lower end of the data set at all.

The lower tail is the most important portion of the test population since the low strength members are the ones most vulnerable to failure. Another distribution model can be chosen for use in the calibration to better represent the lower end of the test data set (the lower 15%). This will ultimately produce a much narrower range of $\phi$ values.
Find range on reliability index $\phi$ such that for fixed factored loads, a target $\beta$ is achieved.

15% data (lower tail)

$\phi = 0.85$
gives

$\beta = 2.6$ to $2.8$

Re-plotting the $\beta-\phi$ ($\alpha$ fixed) correlation using the lower tail model yields a more satisfying result. In this case, the value of $\phi = 0.85$ used for bending strength is consistent with that found in the design code equation.

The procedure to calibrate the code values with a probability analysis is mathematically sophisticated, and is not typically a design issue. It is useful however to be aware of the background to the design rules to gain a better understanding of the issues affecting safety and reliability, and in some cases to make rational decisions where the code does not provide direction.
What stays the same as ASD?

Well, how different is LRFD from ASD in terms of a design process? Do I have to re-learn design?
The engineering procedures for applying Allowable Stress Design methods to wood structures are published in ANSI/AF&PA NDS-2005. All model codes have used or referenced the NDS for design of wood for literally decades, so many designers are already familiar with its contents.

LRFD does not alter the familiarity. Many of the ASD features that designers have come to know have remained the same: basic equation format, adjustment factors, behavioral equations. The LFRD Manual has been eclipsed now by the 2005 NDS which includes LRFD and ASD design processes.

In terms of application of LRFD principles, design process does not change much. The demand side requires unfactored and factored (new) load calculations. The capacity side remains in the same form. Procedural steps are essentially the same as ASD for various structural components.
What changes from ASD?

Well then, what does change?
LRFD vs. ASD

• Three new notations - $\phi$, $\lambda$, and $K_F$
• Design loads (factored) for safety are bigger
• Design loads (unfactored) for serviceability are the same
• Material resistance values are bigger
• Load Duration factor changes to Time Effect Factor

These are some of the distinguishing features of LRFD. There is new notation (partly in Greek). Your calculations will develop bigger numbers as end-results. And there is a terminology change.
Here is why you get bigger numbers with LRFD in design calculations. The way safety is addressed in the two approaches is fundamentally different. ASD makes use of a theoretical safety margin that was applied to material stresses. Controversy often surrounded the theoretical safety margin and its lack of a rigorous basis of determination. ASD also features the comparison of stresses in the demand/capacity relations.
LRFD vs. ASD

LRFD features a statistical basis for a measurable probability of failure and thus insures a measurable level of safety. Factored load equations (with few exceptions) are standardized across all material groups. Resistance values are only modified by a reliability factor that varies by material and mode of use. In the demand/capacity relations, loads or moments are typically compared.
These are the eight fundamental factored load combinations used for safety analysis in LRFD. Reading into the symbology will yield what kind of environmental event the structure is being exposed to. Many more additional equations are derived from these when direction is taken into account.
LRFD Safety Design Equation

\[
\text{Demand} \leq \text{Capacity} \\
\sum_{i=1}^{n} \alpha Q \leq \lambda \phi R_n \\
\beta
\]

This is the basic form of the demand / capacity relation for LRFD.
The LRFD resistance factors (or reliability indices) for wood are shown here from new Appendix N for member properties and connections. The lower the number, the more vulnerable the material in the respective mode. Since these indices vary by material, and by mode of use, many designers exploit the reliability index factors from different materials to get the best from them.

<table>
<thead>
<tr>
<th>Application</th>
<th>Property</th>
<th>Symbol</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Member</td>
<td>$F_b$</td>
<td>$\phi_b$</td>
<td>0.85</td>
</tr>
<tr>
<td></td>
<td>$F_t$</td>
<td>$\phi_t$</td>
<td>0.80</td>
</tr>
<tr>
<td></td>
<td>$F_r, F_{as}, F_{a}$</td>
<td>$\phi_r$</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td>$F_{c}, F_{cl}$</td>
<td>$\phi_c$</td>
<td>0.90</td>
</tr>
<tr>
<td></td>
<td>$F_{min}$</td>
<td>$\phi_m$</td>
<td>0.85</td>
</tr>
<tr>
<td>Connections</td>
<td>(all)</td>
<td>$\phi_a$</td>
<td>0.65</td>
</tr>
</tbody>
</table>
NDS 2005 LRFD Specification

\( \lambda \) tied to ASCE 7-02 Factored Load Equations:

Table N3 Time Effect Factor, \( \lambda \) (LRFD Only)

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>( \lambda )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.2(D+F) + 1.6(H) + 0.5(L, or S or R)</td>
<td>0.6</td>
</tr>
<tr>
<td>1.2(D+F) + 1.6(L+H) + 0.5(L, or S or R)</td>
<td>0.7 when L is from storage</td>
</tr>
<tr>
<td></td>
<td>0.8 when L is from occupancy</td>
</tr>
<tr>
<td></td>
<td>1.25 when L is from impact</td>
</tr>
<tr>
<td>1.2D + 1.6(L, or S or R) + (L or 0.8W)</td>
<td>0.8</td>
</tr>
<tr>
<td>1.2D + 1.6W + L + 0.5(L, or S or R)</td>
<td>1.0</td>
</tr>
<tr>
<td>1.2D + 1.0E + L + 0.2S</td>
<td>1.0</td>
</tr>
<tr>
<td>0.9D + 1.6W + 1.6H</td>
<td>1.0</td>
</tr>
<tr>
<td>0.9D + 1.0E + 1.6H</td>
<td>1.0</td>
</tr>
</tbody>
</table>

1. Time effect factors, \( \lambda \), greater than 1.0 shall not apply to connections or to structural members pressure-treated with waterborne preservatives (see Reference 30) or fire retardant chemicals.
2. Load combinations and load factors consistent with ASCE 7-02 are listed for ease of reference. Nominal load shall be in accordance with N1.2.

LRFD introduces a new terminology for time effect, formerly known as load duration in ASD. The Time Effect factor \( \lambda \) is an adjustment for the effects of load duration and is calibrated to the primary load in a given load combination. LRFD also employs a new baseline of 10 minutes versus 10 years for \( \lambda = 1.0 \). Reduced to 3 general factors: 1.0 for short term, 0.8 for long term, 0.6 for permanent; this approach is consistent with international codes. By prescription, \( \lambda \) is tied to the LRFD load combination equation used.
Impact of Time Effect Factor

- accurate, but not “over-precise”
- easier for designers
  - 1.0 short term
  - 0.8 long term
  - 0.6 permanent
- consistent with International Codes

This method of dealing with time effect has advantages.
Even though the 2005 NDS allows both ASD and LRFD, it does so within the same document size as its predecessor. The factor $K_F$ tabled in Appendix N converts ASD material values in the 2005 Supplement for use with LRFD. This makes implementation of LRFD within the same document very straightforward.
Here is the $K_F$ table from Appendix N. It is dependent on the wood property, its reliability index, and application.
The question of why use LRFD for wood often arises, when ASD has served so well for many years. Many good reasons centered around changing times and evolution of standards documents suggest that there are advantages for LRFD: designing in multiple materials which may have an LRFD basis. There is a more rational treatment of loads with LRFD, and size advantages often result because of this.

Further, ASD load combinations have not been maintained in deference to LRFD load combinations.
Here’s an example comparing the two design processes. Consider a simple beam under uniform load, with given section properties. We have a displacement limit state (maximum) of span/360.

Both methods require determination of the safety and serviceability demand loads. Note the inclusion of the prescribed load factor(s) in the LRFD demand safety load. The serviceability loads are the same for both approaches.
Here we consider two safety limit states: shear and flexure. The demand / capacity relations for shear for this problem are shown.

ASD modifies the capacity with the $C_D$ factor for load duration. The LRFD capacity equation includes the time effect factor $\lambda$ and the reliability factor for shear $\phi_v$, as well as the $K_F$ format conversion factor. Note that factored LRFD loads are used in the demand.
Application - LRFD vs. ASD

Beam Example - UDL Simply Supported

Safety Limit State 2

<table>
<thead>
<tr>
<th>FLEXURE</th>
<th>LRFD</th>
<th>ASD</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\frac{wL^2}{8} \leq \lambda \phi_b K_F F'_b S$</td>
<td>$\frac{wL^2}{8} \leq F'_b C_D S$</td>
</tr>
</tbody>
</table>

Prime denotes inclusion of applicable C factors except $C_D$

The demand / capacity relations for flexure for this problem reveal much the same in comparison.

ASD modifies the capacity with the $C_D$ factor for load duration. The LRFD capacity equation includes the time effect factor $\lambda$ and the reliability factor for bending $\phi_b$, as well as the $K_F$ format conversion factor. Note again, that factored LRFD loads are used in the demand.
The serviceability limit state considered here is maximum displacement of span/360 under service load wₐ.

Note that both approaches use the same equation with very little difference. The important note here is that LRFD uses unfactored actual loads, just like ASD because you want a real measure of actual performance.

In summary, the design process for wood has not changed. LRFD requires the use of load and resistance factors that designers presently skilled in steel and concrete design using LRFD already are familiar with. But as will be seen, there are advantages to be gained with LRFD in final section determination, especially if the problem is governed by a safety limit state.
Chapter 1 – ASD and LRFD

- Behavioral equations are the same for both methods.
- Adjustment factor tables include applicable factors for determining adjusted ASD or LRFD design values.
- Appendix N – Mandatory Appendix for Load and Resistance Factor Design (LRFD), outlines requirements that are unique to LRFD, such as LRFD load combinations consistent with ASCE 7-02, and adjustment factors for LRFD, including $K_F$.

So in terms of design process, Chapter 1 accommodates both ASD and LRFD well. Behavioral equations don’t change. Adjustment factors are the same for both processes. Appendix N provides everything the designer needs to work with the NDS in an LRFD format.
Chapter 1 - Terminology

Basic requirements for checking strength are revised to use terminology applicable to both ASD and LRFD.

Example:

“3.3.1 The actual bending stress or moment shall not exceed the adjusted allowable bending design value.”

In equation format, this takes the standard form $f_b \leq F_{b'}$.

- “allowable” (typically associated with ASD) replaced by adjusted
  • to be more generally applicable to either ASD or LRFD and to better describe the approach of applying adjustment factors to reference design values.

- Reference design values ($F_{b'}, F_{t'}, F_{v'}, F_{c}, F_{c'}, E, E_{min}$) are multiplied by adjustment factors to determine adjusted design values ($F_{b'\prime}, F_{t'\prime}, F_{v'\prime}, F_{c'\prime}, F_{c'\prime}, E', E_{min'}$).

Chapter 1 also describes terminology used in the NDS. For 2005 because of the dual ASD and LRFD format, there are two changes. The old term “allowable” has given way to adjusted. And the base design values from the NDS Supplement are now called reference design values. Reference values are those without adjustment factors applied. Adjusted values are reference values with adjustment factors applied.

This concludes Chapter 1.
Chapter 2 – Adjustment Factors

- Applicable to ALL wood products
- Adjusts from reference to site conditions
  - $C_D$, $\lambda$: time-dependent
  - $C_M$: wet service
  - $C_t$: temperature

Chapter 2 deals with adjustment factors that are global in origin. These factors often are representative of the environment in which the wood structure is placed. Wet service, temperature, and load application duration may become critical issues depending on the environment.

The new feature here is the Time Effect Factor for LRFD notated as $\lambda$ found in Appendix N.
Chapter 2 – Adjustment Factors

• Wet Service Factor $C_M$

Let’s take a look at the wet service factor.
Design values tabulated in the NDS for sawn lumber apply to material surfaced in any condition and used in dry conditions of service. Such conditions are those in which the moisture content in use will not exceed a maximum of 19%. The graph, here, shows how wood in the right conditions of environmental temperature and relative humidity can reach equilibrium moisture content (EMC’s) of 19% or more. This >19% regime is not only problematic for some of the wood’s structural properties, but can also lead to early decay or other durability problems.
Here graphically, in somewhat general terms, is what happens to various structural properties of wood in the region of high EMC. Decreases in the structural properties are noted, especially for the crushing strength $F_{\text{cperp}}$. 
Wet Service Factor, $C_M$

- values found in the NDS Supplement for lumber

<table>
<thead>
<tr>
<th>Wet Service Factors, $C_M$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_b$</td>
</tr>
<tr>
<td>0.85*</td>
</tr>
</tbody>
</table>

* when $(F_b)(C_t) \leq 1,150$ psi, $C_M = 1.0$
** when $(F_v)(C_{c\perp}) \leq 750$ psi, $C_M = 1.0$

Wet service adjustment factors are provided for uses where the 19% EMC limit will be exceeded for a sustained period of time, or for repeated periods.

Applications in which structural members are regularly exposed directly to rain and other sources of moisture are typically considered wet conditions of service. Members that are protected from the weather by roofs or other means but are occasionally subjected to wind blown moisture are generally considered dry applications. The designer must use discretion. NDS Commentary 2.3.3 and 4.1.4 has additional information that can help.
Chapter 3 – Behavioral Equations

• ASD vs LRFD – adjusted stresses from reference

**ASD**  \[ F_n' = F_n C_D \]  x adjustment factors

**LRFD**  \[ F_n' = F_n K_F \phi_n \lambda \]  x adjustment factors

Chapter 3 describes the behavioral relations used in designing wood structures. Additionally, it describes the process of obtaining adjusted stresses from reference values. Between the ASD and LRFD processes, there is not much difference. There are a few additional factors for LRFD to deal with format conversion and safety issues. Adjustment factors are used to deal with wood in specific applications, for which values are found in the respective material chapters that follow.
Chapter 3 – Behavioral Equations

• Beams
  – $C_L$  beam stability

\[
C_L = \frac{1 + \left( \frac{F_{BE}}{F'_b} \right)}{1.9} - \sqrt{\left[ 1 + \left( \frac{F_{BE}}{1.9} \right) \right]^2 - \left( \frac{F_{BE}}{0.95} \right)} \quad (3.3-6)
\]

$F'_b$ = reference bending design value multiplied by all applicable adjustment factors except $C_v$, $C_n$, and $C_L$ (see 2.3)

$F_{BE} = \frac{1.20 E_{min}}{R_B^2}$

One of the changes in this chapter for 2005, is the restructuring of $C_L$ for beam stability. The Euler buckling term $F_{BE}$ is rewritten as a function of $E_{min}$, the fifth-percentile MOE value, sometimes referred to as “the strength $E$”.

Chapter 3 – Behavioral Equations

• Beams
  – $F_{be}^\prime$ equivalence

\[
F_{be}^\prime = \frac{1.20E_{\text{min}}^\prime}{R_b^2} = \frac{K_{be}E^\prime}{R_b^2}
\]

<table>
<thead>
<tr>
<th></th>
<th>2005 NDS</th>
<th>2001 NDS</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_{\text{min}}^\prime$</td>
<td>$K_{be}$ = $0.745 - 1.225(COV_e)$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$= 0.439$ for visually graded lumber</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$= 0.561$ for machine evaluated lumber (MEL)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$= 0.610$ for products with $COV_e \leq 0.11$</td>
<td></td>
</tr>
</tbody>
</table>

– $E_{\text{min}}^\prime$ adjusted for safety for both ASD and LRFD processes

$E_{\text{min}}^\prime$ now adjusts safety for both ASD and LRFD processes and is tabulated for ease of use in the 2005 NDS Supplement in the various wood products reference value tables.
Chapter 3 – Behavioral Equations

• Columns
  - \( C_P \) column stability

\[
C_P = \frac{1 + \left( \frac{F_{cE}}{F_c^*} \right)}{2c} - \sqrt{\left[ \frac{1 + \left( \frac{F_{cE}}{F_c^*} \right)}{2c} \right]^2 - \frac{F_{cE}}{F_c^*} \frac{c}{c}} \quad (3.7-1)
\]

- \( F_c^* = \) reference compression design value parallel to grain multiplied by all applicable adjustment factors except \( C_i \) (see 2.3)
- \( C_i = 0.8 \) for sawn lumber
- \( C_i = 0.85 \) for round timber poles and piles
- \( C_i = 0.9 \) for structural glued laminated timber or structural composite lumber

Similarly for columns, the Euler buckling term \( F_{cE} \) is rewritten for ease of use.
Chapter 3 – Behavioral Equations

• Columns
  – $F_{cE}$ equivalence
    $$F_{cE} = \frac{0.822E_{\min}'}{\left(\frac{l_e}{d}\right)^2} = \frac{K_{cE}E'}{\left(\frac{l_e}{d}\right)^2}$$
    
    2005 NDS  2001 NDS
    $K_{cE} = 0.510 - 0.839(COV_e')$
    = 0.3 for visually graded lumber
    = 0.384 for machine evaluated lumber (MEL)
    = 0.418 for products with $COV_e \leq 0.11$ (see Appendix F.2)

  – $E_{\min}$ adjusted for safety for both ASD and LRFD processes

$E_{\min}$ now adjusts safety for both ASD and LRFD processes and is tabulated for ease of use in the 2005 NDS Supplement in the various wood products reference value tables.
Chapter 3 – Behavioral Equations

- $E_{\text{min}}$ – $F_{cE}$ equivalence

$$E_{\text{min}} = \frac{1.03E(1 - 1.645(\text{COV}_E))}{1.66}$$

- $E$ = reference MOE
- 1.03 = adjustment factor to convert $E$ to a pure bending basis (shear-free) (use 1.05 for glulam)
- 1.66 = factor of safety
- $\text{COV}_E$ = coefficient of variation in MOE (NDS Appendix F)

Rather than use the tabled value for $E_{\text{min}}$, one could calculate it using the above relation.
Chapter 3 – Column Equations

Column Example – Axial Load only

Safety Limit State

<table>
<thead>
<tr>
<th>LRFD</th>
<th>ASD</th>
</tr>
</thead>
<tbody>
<tr>
<td>( P \leq P' )</td>
<td>( P \leq P' )</td>
</tr>
</tbody>
</table>


demand | capacity | demand | capacity

Here’s a numerical column example comparing ASD and LRFD design processes. Both ASD and LRFD methods require determination of the safety demand loads. Note the inclusion of the prescribed load factor(s) in the LRFD safety demand load.

ASD modifies the compression capacity with the \( C_D \) factor for load duration. The LRFD capacity equation includes the time effect factor \( \lambda \) and the reliability factor for compression \( \phi_c \). Note that factored LRFD loads are used in the demand.
Chapter 3 – Column Equations

Column Example

Dead Load = 5500 lbs
Live Load = 31500 lbs
Normal Time Duration

L = 16 ft (each direction)
Ends pinned

Consider a pinned column under axial load, with given section properties. This 16 foot unbraced column has applied dead and live loads.
Chapter 3 – Column Equations

Column Example

<table>
<thead>
<tr>
<th>DEMAND LOADS</th>
<th>LRFD</th>
<th>ASD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Safety</td>
<td>[ P = \Sigma \alpha Q ]</td>
<td>[ P = \Sigma Q ]</td>
</tr>
<tr>
<td></td>
<td>[ = 1.2 D + 1.6 L ]</td>
<td>[ = D + L ]</td>
</tr>
<tr>
<td></td>
<td>[ = 1.2 \times 5500 + 1.6 \times 31500 ]</td>
<td>[ = 5500 + 31500 ]</td>
</tr>
<tr>
<td></td>
<td>[ = 57000 \text{ lbs} ]</td>
<td>[ = 37000 \text{ lbs} ]</td>
</tr>
</tbody>
</table>

First we compute the demand loads for design for safety. LRFD uses load factors applicable to the load type and typically results in a larger numerical result than ASD.
Chapter 3 – Column Equations

Column Example

Try 6 3/4" x 9" Glulam visually graded western species, 16F-1.3E

GEOMETRY

<table>
<thead>
<tr>
<th>Section</th>
<th>X-X</th>
<th>Y-Y</th>
</tr>
</thead>
<tbody>
<tr>
<td>d = 9 in</td>
<td>K_\text{ed} = 1.0</td>
<td>K_\text{eb} = 1.0</td>
</tr>
<tr>
<td>b = 6.75 in</td>
<td>L_\text{ed} = 16 ft</td>
<td>L_\text{eb} = 16 ft</td>
</tr>
<tr>
<td>A = 61 in^2</td>
<td>L_\text{d} = K_\text{ed} L_\text{d}</td>
<td>L_\text{b} = K_\text{eb} L_\text{b}</td>
</tr>
</tbody>
</table>

Slenderness = \frac{L_\text{ed}}{\min(b, d)} = \frac{L_\text{eb}}{\min(b, d)} 

= 28

For this design, we try a 6 ¾” x 9” glulam (combination symbol 1). From the section geometry, the cross-sectional area is found. The column can buckle through the X-X or Y-Y directions depending upon bracing present in each direction. It is important to check bracing geometry and its relationship to section dimension. Here, the column is unbraced over its entire height, so the column could buckle in the direction of least section dimension (in the X-X plane here). Checking the slenderness ratio gives us an appreciation for this. We want to use the larger of the slenderness ratio with respect to the b and d section dimensions. In this case, the least dimension direction, b, governs, with a slenderness ratio of 28. This value is in line with what has been called a “long” or skinny column.
Next, we consider the column’s environment. Any changes from the base environment for the wood must be reflected in the adjustment factors. Note the adjustment factors for load duration are different numerically for LRFD and ASD; the LRFD version being prescribed by the load combination equation used.
Chapter 3 – Column Equations

<table>
<thead>
<tr>
<th>MATERIALS</th>
<th>LRFD</th>
<th>ASD</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_c$</td>
<td>1550 psi</td>
<td>1550 psi</td>
</tr>
<tr>
<td>$E$</td>
<td>1,500,000 psi</td>
<td>1,500,000 psi</td>
</tr>
<tr>
<td>$E_{\text{min}}$ (Glulam)</td>
<td>780,000 psi</td>
<td>780,000 psi</td>
</tr>
<tr>
<td>$\phi_c$ (compression)</td>
<td>0.90</td>
<td>0.9</td>
</tr>
<tr>
<td>$\phi_s$ (stability)</td>
<td>0.85</td>
<td>0.85</td>
</tr>
<tr>
<td>$K_F$ compression</td>
<td>$2.16 / \phi_c = 2.40$</td>
<td></td>
</tr>
<tr>
<td>$K_F$ stability</td>
<td>$1.5 / \phi_s = 1.76$</td>
<td></td>
</tr>
</tbody>
</table>

For material design values, we can go to the 2005 NDS Supplement and extract the reference values for compression and MOE. Note that the values are the same for both ASD and LRFD, although they are really the ASD values. For LRFD, we want the “buckling”, or “stability”, or “strength” $E$ (it has been called any of these) that is found in the 5th percentile $E$ column, $E_{\text{min}}$. We also need the $K_{CE}$ conversion factor for glulam (which part of a combination produces a 5th percentile $E$ value from a mean $E$). It is convenient now to also pull the LRFD resistance factors for column compression and stability, and the format conversion factors $K_F$ from Appendix N of the standard.
Chapter 3 – Column Equations

Column Example

<table>
<thead>
<tr>
<th>CAPACITY</th>
<th>LRFD</th>
<th>ASD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crushing</td>
<td>$F_{c^*} = F_c K_F \lambda \phi_c C_m C_t$</td>
<td>$F_{c^*} = F_c C_D C_m C_t$</td>
</tr>
<tr>
<td></td>
<td>$(1,550)(2.40)(0.8)(0.9)(1.0 \text{ all})$</td>
<td>$(1,550)(1.0)(1.0 \text{ all})$</td>
</tr>
<tr>
<td></td>
<td>$= 2,678 \text{ psi}$</td>
<td>$= 1,550 \text{ psi}$</td>
</tr>
<tr>
<td>$P_0 = A F_{c^*}$</td>
<td>$= (61)(2,678)$</td>
<td>$= (61)(1,550)$</td>
</tr>
<tr>
<td></td>
<td>$= 163,382 \text{ lbs}$</td>
<td>$= 94,550 \text{ lbs}$</td>
</tr>
</tbody>
</table>

The first limit state for columns is crushing: characteristic of short stocky geometries. We compute the crushing strength in stress form as $F_{c^*}$. Note that the ASD formulation includes the load duration factor $C_D$, while LRFD does not. The crushing load is also computed as shown.
Chapter 3 – Column Equations

Column Example

<table>
<thead>
<tr>
<th>CAPACITY</th>
<th>LRFD</th>
<th>ASD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Buckling</td>
<td>( E'<em>\text{min} = E</em>{\text{min}} K_F \phi_s C_M C_t )</td>
<td>( E'<em>\text{min} = E</em>{\text{min}} C_M C_t )</td>
</tr>
<tr>
<td></td>
<td>( = (780,000)(1.76)(0.85)(1.0) )</td>
<td>( = 780,000 \text{ psi} )</td>
</tr>
<tr>
<td></td>
<td>( = 1,166,880 \text{ psi} )</td>
<td>( )</td>
</tr>
<tr>
<td></td>
<td>( F_{cE} = \frac{0.822E'_\text{min}}{(\text{Slenderness})^2} )</td>
<td>( F_{cE} = \frac{0.822E'_\text{min}}{(\text{Slenderness})^2} )</td>
</tr>
<tr>
<td></td>
<td>( = \frac{(0.822)(1166880)}{(28)^2} )</td>
<td>( = \frac{(0.822)(780000)}{(28)^2} )</td>
</tr>
<tr>
<td></td>
<td>( = 1,223 \text{ psi} )</td>
<td>( = 818 \text{ psi} )</td>
</tr>
</tbody>
</table>

The second limit state is buckling: characteristic of slender geometries. The column buckling equation is derived from the familiar Euler formulation simplified further here for rectangular sections. Note that all these expressions are in terms of stress units – a change for designers who formerly used the 1996 LRFD Manual (it was load-based). Also note that the buckling stress is dramatically higher for LRFD than for ASD.
Chapter 3 – Column Equations

### Column Example

<table>
<thead>
<tr>
<th>CAPACITY</th>
<th>LRFD</th>
<th>ASD</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\alpha_c$ Ratios</td>
<td>$F_{cE}/F'_c$ = 1223/2678</td>
<td>$F_{cE}/F'_c$ = 818/1550</td>
</tr>
<tr>
<td></td>
<td>= 0.46</td>
<td>= 0.53</td>
</tr>
</tbody>
</table>

The slenderness of our column lies somewhere between the crushing and buckling limit states. To find out where, we compute the $C_p$ ratio. $C_p$ factors down the crushing strength based on the slenderness of our column. Again, the forms of the buckling-crushing ratio $\alpha_c$ are basically the same for both ASD and LRFD.
Chapter 3 – Column Equations

Column Example

<table>
<thead>
<tr>
<th>CAPACITY</th>
<th>LRFD</th>
<th>ASD</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_p$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$= 0.43$</td>
<td>$= 0.48$</td>
<td></td>
</tr>
<tr>
<td>$P' = AF_c C_p$</td>
<td>$P' = AF_c^* C_p$</td>
<td></td>
</tr>
<tr>
<td>$(61)(2,678)(0.43)$</td>
<td>$(61)(1,550)(0.48)$</td>
<td></td>
</tr>
<tr>
<td>$(61)(1,146)$</td>
<td>$(61)(744)$</td>
<td></td>
</tr>
<tr>
<td>$69,914$ lbs</td>
<td>$45,384$ lbs</td>
<td></td>
</tr>
</tbody>
</table>

The values of $C_p$ we come up with are reasonable for a slender column a little on the “fat” side. Super-skinny columns would produce very low values of $C_p$.

And the expression of the $C_p$ equation is exactly the same for both ASD and LRFD. Setting in the $\alpha_c$ values gives the corresponding numerical results. Finally we compute the capacity of our column using the expressions in $P'$ at the bottom. Note again, the LRFD capacity value is higher.
Chapter 3 – Column Equations

Column Example – Axial Load only

Safety Limit State

<table>
<thead>
<tr>
<th>COMPRESSION</th>
<th>LRFD</th>
<th>ASD</th>
</tr>
</thead>
<tbody>
<tr>
<td>P'</td>
<td>P ≤</td>
<td>P ≤</td>
</tr>
<tr>
<td>57,000 lbs</td>
<td>69,914 lbs</td>
<td>37,000 lbs</td>
</tr>
</tbody>
</table>

Demand / Capacity Ratio

Now we compare the load demands to the column capacities for each method. Our trial column works for this design for both methods. No doubt, LRFD has higher numbers, but we can see the approximate equivalence in the two methods through the demand/capacity ratio. The two ratios are exactly the same for both ASD and LRFD processes. Overall, the design process for LRFD has not changed from ASD, but in fact is remarkably similar.
Chapter 3 – Behavioral Equations

• Tension members (tension parallel to grain)

ASD 
\[ F_t' = F_t C_D \times \text{adjustment factors} \]

LRFD 
\[ F_t' = F_t K_F \phi_t \lambda \times \text{adjustment factors} \]

For tension parallel-to-grain members, behavioral equations don’t change and the format is the same as that for bending or compression members.
Chapter 3 – Behavioral Equations

- wood and tension perpendicular to grain
  - the evil of wood connections

**initiators:**
- notches
- large diameter fasteners
- hanging loads

Tension perpendicular-to-grain is a different story. This is wood’s weakest link: *tension perpendicular to the grain*. Tension-perp often leads to sudden catastrophic failures and should be avoided at all costs. Awareness of how the wood is being loaded is all that is needed to avoid this issue. Large diameter connectors can also initiate this weak strength mode.
Chapter 3 – Behavioral Equations

• Combined bending and axial-compression

\[ \frac{f_c}{F_c'} \left[ \frac{f_c}{F_c'} \right]^2 + \frac{f_{b1}}{F_{b1}'} \left[ 1 - (f_c/F_{ce1}) \right] + \frac{f_{b2}}{F_{b2}'} \left[ 1 - (f_c/F_{ce2}) - (f_{b1}/F_{ce})^2 \right] \leq 1.0 \quad (3.9-3) \]

The use of \( E_{min} \) is the new feature in the interaction equation for combined bending and axial-compression strength for wood members.
Chapter 3 – Behavioral Equations

- Combined bending and axial - compression

\[ E_{\text{min}} \]

where:

\[ f_{e} < F_{e11} = \frac{0.822 E_{\text{min}}}{(t_{e} / d_{w})^{2}} \] for either uniaxial edge-wise bending or biaxial bending

and

\[ f_{e} < F_{e22} = \frac{0.822 E_{\text{min}}}{(t_{e} / d_{w})^{2}} \] for uniaxial flatwise bending or biaxial bending

and

\[ f_{e} < F_{e44} = \frac{1.20 E_{\text{min}}}{(d_{n})^{2}} \] for biaxial bending

\[ f_{e} = \text{actual edgewise bending stress (bending load applied to narrow face of member)} \]

\[ f_{e} = \text{actual flatwise bending stress (bending load applied to wide face of member)} \]

\[ d_{w} = \text{wide face dimension (see Figure 3H)} \]

\[ d_{n} = \text{narrow face dimension (see Figure 3H)} \]

\[ E_{\text{min}} \] will appear as a variable in any of the Euler terms.
Chapter 3 – Behavioral Equations

- **Bearing** $F_{c\perp}$
  - $C_b$, bearing area factor

  $$C_b = \frac{\ell_b + 0.375}{\ell_b} \quad (3.10-2)$$

  where:

  $\ell_b$ = bearing length measured parallel to grain, in.

  *same as NDS 2001*

The provisions for bearing perpendicular to the grain are the same as those in the 2001 NDS.
Chapter 4 – Lumber

- Design values
  - More foreign species added
  - MSR / MEL values expanded
  - 2001 NDS shear values *increased*
    - stayed the same for 2005

Chapter 4 begins the wood material chapters. In these, the 2005 NDS introduces new changes to accommodate LRFD in addition to ASD and adds more varieties of wood species and grades.
Chapter 4 – Lumber

• Adjustment factors for lumber
  – $C_f$  form removed
  – $C_{fu}$  flat use
  – $C_i$  incising
  – $C_r$  repetitive use
  – $C_t$  temperature

Adjustment factors in these chapters are unique to the material described in the chapter. All the adjustment factors appear in basically the same format that has changed slightly for 2005 to include factors unique to ASD and LRFD in addition to common factors.

For lumber in the 2005 NDS, the form factor $C_f$ has been removed.
Chapter 4 – Lumber

• Adjustment factors
  – $C_f$ form factor removed

Why?
  – derived from plastic deformation in small clear specimens that may not be applicable to full-size members
  – applicability to standard wood products was limited (not allowed in poles & piles – it’s built into the reference design value)

The removal of the form factor stems from the reason that this value was originally derived from plastic deformation in small clear specimens that may not be applicable to full-size members. In addition, its applicability to standard wood products (which are almost always rectangular in cross-section) was limited. The form factor is not allowed in poles & piles since it is already built into the reference design value. Thus it was removed.
Chapter 5 – Glued Laminated Timber

- Design values added to supplement
- Reformatted glulam radial tension values to apply both to ASD and LRFD
- Shear values increased 10%

Chapter 5 for glulam for 2005 adds new design process capability, increased shear strength, and new materials.
Chapter 5 – Glulam

• Design values
  – $F_{rt}$ radial tension

5.2.2 Radial Tension, $F_{rt}$

For curved bending members, the following reference radial tension design values perpendicular to grain shall apply:

<table>
<thead>
<tr>
<th>Southern Pine</th>
<th>all loading conditions</th>
<th>$F_{rt} = 0.75 F_{so}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Douglas Fir-Larch, Douglas Fir South, Hem-Fir, Western Wood, and Canadian softwood species</td>
<td>other types of loading</td>
<td>$F_{rt} = 15$ psi</td>
</tr>
</tbody>
</table>

The radial tension adjustment factor $F_{rt}$ is now included in the adjustment factor table. The table has also been reformatted to include adjustment factors unique to ASD and LRFD in addition to common ones. Radial tension is often a design consideration in curved or arched glulam members.
Chapter 5 – Glulam

• Adjustment factors
  – \( C_v \) volume

\[
C_v = \left( \frac{21}{L} \right)^{1/3} \left( \frac{12}{d} \right)^{1/3} \left( \frac{5.29}{b} \right)^{1/3} \leq 1.0
\]

where:
- \( L \) = length of bending member between points of zero moment, ft
- \( d \) = depth of bending member, in.
- \( b \) = width (breadth) of bending member.
  For multiple piece width layouts, \( b \) = width of widest piece used in the layup.
  Thus, \( b < 10.75^* \)
- \( x = 20 \) for Southern Pine
- \( x = 10 \) for all other species

The volume effect factor for glulam retains its 2001 NDS form.
Chapter 6 – Poles & Piles

• Design values
  – No changes

No changes to the poles and piles chapter from the 2001 NDS.
Chapter 7 – I-joists

• Design values
  – M, V, EI, K – no changes

• Evaluation Reports
  – Contain proprietary design

No changes to design values from the 2001 NDS provisions from the I-joists chapter. Again, this chapter deals with generics; designers are encouraged to consult proprietary design information for the product under consideration.
Chapter 7 – I-Joists

• Adjustment factors
  – \( C_r = 1.0 \)
  
  • revised to agree with ASTM D5055-02.
  
  • factor of 1.0 maintained for clarity transitioning from 2001 NDS

A significant change in this chapter, however, is the repetitive use factor returning to 1.0. This was revised to agree with a change in ASTM D5055-02, and is maintained for clarity coming from the 2001 NDS.
Chapter 8 – Structural Composite Lumber (SCL)

• Design values in evaluation reports
  – Note less variation (low COV)
  – No changes

• Evaluation Reports
  – Contain proprietary design

Chapter 8 materials (SCL) have extremely low variability. Designers should consult proprietary information for the product under consideration.
Chapter 8 – Structural Composite Lumber (SCL)

• Adjustment factors
  – \( C_r = 1.04 \)
    • \( C_r \) is different than lumber \( (C_{r\text{ lumber}} = 1.15) \)
    • Applied to \( F_b \)

The repetitive use factor \( C_r \) remains at 1.04 for 2005. It is different in value than lumber and is applied only to the bending stress if three or more members are sharing load in close proximity.
Design values are expressed algebraically in typical terms in this chapter for Wood Structural Panels, but numerical design values need to be obtained from an approved source.
The adjustment factor table includes adjustment factors for product fabrication as well as size for both ASD and LRFD processes.

<table>
<thead>
<tr>
<th>Table 9.3.1 Applicability of Adjustment Factors for Wood Structural Panels</th>
<th>ASD only</th>
<th>ASD and LRFD</th>
<th>LRFD only</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Load/Service Force</td>
<td>Web, Nails, etc.</td>
<td>General Force</td>
</tr>
<tr>
<td>$F_{AW} = F_{AS}$</td>
<td>$C_{G}$</td>
<td>$C_{M}$</td>
<td>$C_{C}$</td>
</tr>
<tr>
<td>$F_{AI} = F_{AI}$</td>
<td>$C_{G}$</td>
<td>$C_{M}$</td>
<td>$C_{C}$</td>
</tr>
<tr>
<td>$F_{BL} = F_{BL}$</td>
<td>$C_{G}$</td>
<td>$C_{M}$</td>
<td>$C_{C}$</td>
</tr>
<tr>
<td>$F_{A}^* = F_{A}$</td>
<td>$C_{G}$</td>
<td>$C_{M}$</td>
<td>$C_{C}$</td>
</tr>
<tr>
<td>$E_{I} = EI$</td>
<td>$C_{G}$</td>
<td>$C_{M}$</td>
<td>$C_{C}$</td>
</tr>
<tr>
<td>$E_{B} = EB$</td>
<td>$C_{G}$</td>
<td>$C_{M}$</td>
<td>$C_{C}$</td>
</tr>
<tr>
<td>$G_{L} = G_{L}$</td>
<td>$C_{G}$</td>
<td>$C_{M}$</td>
<td>$C_{C}$</td>
</tr>
</tbody>
</table>
Chapter 10 – Mechanical Connections

• “full design value” terminology revised
  – Examples:
    • Minimum spacing for full design value
    • Minimum end distance for full design value
  – alternate placement descriptions ensure full design value is developed (local stresses)
  – apply section 10.1.2 (Appendix E) to check wood strength at connections

Chapter 10 begins the connections chapters. For mechanical connections, the term “full design value” is revised with the intent that mechanical fasteners should be appropriately placed so that they can develop their full design value capability as tabulated in the 2005 NDS. In order to do this, an assure proper placement, the provisions of Appendix E to check local stresses should be used.
• “full design value” terminology revised
  – Examples:

10.2.2 Multiple Fastener Connections
  When a connection contains two or more fasteners of the same type and similar size, each of which exhibits the same yield mode (see Appendix I), the total adjusted design value for the connection shall be the sum of the adjusted design values for each individual fastener. Local stresses in connections using multiple fasteners shall be checked in accordance with principles of engineering mechanics (see 10.1.2).

11.1.2.4 Edge distance, end distance, and fastener spacing required to develop full design values shall not be less than the requirements in be in accordance with Table 11.5.1A-D.

These provisions show the changes made in the 2005 NDS from the 2001 NDS to make the terminology more clear in intent.
As in the 2001 NDS, provisions for stresses in members at connections have been written as follows:

10.1.2 Structural members shall be checked for load carrying capacity at connections in accordance with all applicable provisions of this standard including 3.1.2, 3.1.3, and 3.4.3.3. Local stresses in connections using multiple fasteners shall be checked in accordance with principles of engineering mechanics. One method for determining these stresses is provided in Appendix E.
2005 NDS Appendix E

- **Appendix E** Local Stresses in Fastener Groups

- new topic introduced in 2001 NDS
  - retroactive to all previous editions of the NDS

---

**Appendix E (Non-mandatory) Local Stresses in Fastener Groups**

**E.1 General**

When a fastener group is composed of closely-spaced fasteners loaded parallel to grain, the capacity of the fastener group may be limited by wood failure at the net section or tear-out around the fasteners caused by local stresses. One method to evaluate member strength for local stresses around fastener groups is outlined in the following:

E.3.1 Assuming one shear line on each side of bolts in the row (observed in tests of bolted connections), Equation E.3.1 becomes:

\[ \tau = \frac{F}{2} \left( \frac{t}{d} \right) \left( \frac{b}{w} \right) \]

where:

- \( F \) = net tension force
- \( t \) = thickness
- \( d \) = diameter
- \( b \) = width
- \( w \) = width

---

Appendix E is significant.

Local stresses in fastener groups was new to the 2001 NDS and provides one method to evaluate capacity of a fastener group limited by wood related failure mechanisms. Example problems are added to demonstrate application of Appendix E provisions for checking net section tension capacity, row tear-out capacity, and group tear-out capacity.
Local Stresses in Fastener Groups

- Closely spaced fasteners
  - brittle failure
  - lower capacity

*wood failure mechanisms need to be considered in design*

Where a fastener group is composed of closely-spaced fasteners loaded parallel to grain, the capacity of the fastener group may be limited by wood failure at the net section or tear-out around the fasteners caused by local stresses.
Local Stresses in Fastener Groups

- Properly spaced fasteners
  - increased ductility
  - higher capacity

spread out the fasteners!

By increasing the spacing between the fasteners, much higher capacity and ductility is achieved, even with fewer fasteners!

The 2001 Edition of the *National Design Specification® (NDS®) for Wood Construction* contained editorially clarified provisions for checking stresses in members at connections. The following requirements, included in the 2005 NDS, are also applicable to all prior editions of the NDS:

**Stresses in Members at Connections** - Structural members shall be checked for load carrying capacity at connections in accordance with all applicable provisions of the NDS. Local stresses in connections using multiple fasteners shall be checked in accordance with principles of engineering mechanics.

One method for determining these stresses is provided in Appendix E from the 2005 NDS, which is also available free from www.awc.org. All referenced sections and design values used in sample solutions of this Addendum are based on information in the 2005 NDS.
Local Stresses in Fastener Groups

- **Appendix E NDS Expressions**

  - **Net tension:**
    \[ Z'_{NT} = F'_t A_{net} \]
  
  - **Row tear-out:**
    \[ Z'_{RT} = n_i F'_t t_{min} \]
    \[ Z_{RT} = \sum_{i=1}^{n} Z'_{RT} \]

Tabulated nominal design values for timber rivet connections in Chapter 13 account for local stress effects and do not require further modification by procedures outlined in Appendix E. The capacity of connections with closely-spaced, large diameter bolts has been shown to be limited by the capacity of the wood surrounding the connection. Connections with groups of smaller diameter fasteners, such as typical nailed connections in wood-frame construction, may not be limited by wood capacity.

Appendix E leads the designer through the stress checks for three failure modes: net tension capacity of the wood through the cross-section, row tear-out, and...
Modification of fastener placement within a fastener group can be used to increase row tear-out and group tear-out capacity limited by local stresses around the fastener group. Increased spacing between fasteners in a row is one way to increase row tear-out capacity. Increased spacing between rows of fasteners is one way to increase group tear-out capacity.

Footnote 2 to Table 11.5.1D (2005 NDS) limits the spacing between outer rows of fasteners paralleling the member on a single splice plate to 5 inches. This requirement is imposed to limit local stresses resulting from shrinkage of wood members. When special detailing is used to address shrinkage, such as the use of slotted holes, the 5 inch limit can be adjusted.

These provisions apply to the 2005 NDS and ALL PRIOR EDITIONS. The example calculations provided in Appendix E use design values from the 2005 NDS. Appendix E in its entirety is available as a free PDF download from www.awc.org.
Local Stresses in Fastener Groups

The Three Checks

- Net Section Tension Capacity
- Row Tear-Out Capacity
- Group Tear-out Capacity

Here is a summary of the three stress checks that may be applied to various common connector situations.
Local Stresses in Fastener Groups

Example

Truss Bottom Chord and Splice

• Design the bottom chord of a sawn lumber commercial/industrial truss to support a tensile force (T) of 20,000 lbs.
• Assume a dry moisture service condition, un-incised material and a load duration factor of 1.0.

Now, let’s work a complete example for truss bottom chord splice.

Example 3-1: Truss Bottom Chord
Local Stresses in Fastener Groups

Example

Truss Bottom Chord and Splice

- The chord includes connections with two rows of 7/8 inch bolts (in a 1/16 inch oversized hole) spaced per NDS Section 11.5 for full design values. Check the local stresses to verify your member size selection.

Example 3-1: Truss Bottom Chord

Practical Considerations

Efficient choice of a trial section requires practical, as well as engineering, considerations. For example, choice of lumber species, grade and even commonly available sizes may differ among geographic regions of the country. Consult your local supplier for assistance. In addition, other considerations include dimensional compatibility with the other members of the truss or minimum sizes required to adequately connect the truss members (while meeting fastener edge distance requirements).
Local Stresses in Fastener Groups

Example Solution

Solution:

• Select a member(s) from the Tension Member Selection Tables in the Structural Lumber Supplement that is adequate to resist 20,000 lbs tensile force (T) due to combined dead load and occupancy live load (D+L).

Engineering Calculations

Using Selection Tables: Select a member(s) from the tension member selection tables in the Structural Lumber Supplement that is adequate to resist 20,000 lbs tensile force (T) due to combined dead load and occupancy live load (D+L).

A double chord of nominal 2x12 ‘s meets practical considerations. Try No.1 Douglas Fir-Larch:
Local Stresses in Fastener Groups

Example Solution

Solution:

double chord of nominal 2x12's meets practical considerations. Try No.1 Douglas Fir-Larch:

T' = (11,900 lbs.)(2 plies) = 23,800 lbs.

Engineering Calculations

T' = (11,900 lbs.)(2 plies) = 23,800 lbs.

But...
Local Stresses in Fastener Groups

Example Solution

**Solution:**

- **Using Design Value Tables:** Calculate allowable tension capacity using tabulated design values and adjustment factors.

  Try a nominal 4x12 No.1 Hem-Fir.

  *(Hem-Fir’s on sale this week!)*

...the local lumber yard has Hem-Fir on sale this week, and you like the price.
Local Stresses in Fastener Groups

Example Solution

Solution:

- Obtain $F_t$ and applicable adjustment factors from the NDS-2001 Supplement and the Structural Lumber Supplement. Calculate the tensile capacity:

$$ T' = F_t' \times A = F_t \times C_D \times C_F \times A $$

$$(625 \text{ psi})(1.0)(1.0)(39.38 \text{ in}^2)$$

$$ = 24,600 \text{ lbs} > 20,000 \text{ lbs} \quad \text{OK} $$

This member satisfies the strength criteria for a tension member.

Using Design Value Tables: calculate allowable tension capacity using tabulated design values and adjustment factors. Try a nominal 4x12 No.1 Hem-Fir. Obtain $F_t$ and applicable adjustment factors from the NDS-2001 Supplement and the Structural Lumber Supplement.

$$ T' = F_t' \times A = F_t \times C_D \times C_F \times A $$

From the Supplements, $F_t$ is 625 psi, $C_D$ equals 1.0, and $C_F$ equals 1.0. The area of a 4x12 is 39.38 square inches. Thus the allowable tension capacity is:

$$ T' = (625)(39.38) = 24,600 \text{ lbs} $$

This member satisfies the strength criteria for a tension member.
Local Stresses in Fastener Groups

**Example Solution**

**Solution:**

*same as the table value*

\[ T' = 24,600 \text{ lbs} \]

...or use the Tension Member Selection Table from the *Structural Lumber Supplement* and get the same answer:

\[ T' = 24,600 \text{ lbs} \]

This member satisfies the strength criteria for a tension member.
Local Stresses in Fastener Groups

Example Solution

Solution:

The chord includes connections with two rows of 7/8 inch bolts (in a 1/16 inch oversized hole) spaced per NDS Section 11.5 for full design values.

- Check the local stresses at the chord connection to verify your member size selection.

Now for the splice connection. To make this easy, we’ll consider a single shear connection using one splice plate and neglect the eccentricity in the fasteners. We’ll set the rows at the 1/3 depths, ‘cause it looks nice (it’s well within NDS spacing limitations). Let’s see if this works.
Local Stresses in Fastener Groups

Example Solution

Solution:
Using Appendix E provisions:

Net Section Tension

Net cross section area = (3.5)(11.25 - 2(0.875+0.0625))
= 32.8 in.²

\[ Z_{NT}' = 625(32.8) = 20,500 \text{ lbs} \]

> 20,000 lbs demand OK

Engineering Calculations

Using NDS Appendix E provisions, calculate local stresses in the fastener group:

Net Section Tension

Calculations follow those previously, but the net area of \((3.5)(11.25-(2)(0.9375)) = 32.8 \text{ square inches}\) replaces the gross area (39.38 in.²) in the calculation:

\[ Z_{NT}' = 625(32.8) = 20,500 \text{ lbs} \]

OK so far...
Local Stresses in Fastener Groups

Example Solution

Solution:

Using Appendix E provisions:

- Row Tear-out Capacity

From the NDS Supplement, $F_v' = 150\text{psi}$

Critical spacing $s_{critical}$ is lessor of end distance (7D), or fastener spacing (4D) = $4(0.875") = 3.5\text{ inches}$.

$$Z_{RT'} = n_{row} n_i F_v' t s_{critical} = (2)(8)(150)(3.5)(3.5) = 29,400\text{ lbs}$$

Still OK...

Engineering Calculations

Row Tear-out Capacity

From the NDS Supplement, $F_v' = 150\text{psi}$. Critical spacing is the lessor of the end distance (7D here for full design value), or the spacing between fasteners in a row (4D); in this case, 3.5 inches. Therefore, row tear-out capacity is calculated as:

$$Z_{RT'} = n_{row} n_i F_v' t s_{critical} = (2)(8)(150)(3.5)(3.5) = 29,400\text{ lbs}$$

Still OK...
Local Stresses in Fastener Groups

Example Solution

Solution:
Using Appendix E provisions:
- Group Tear-out Capacity

Assume:
- uniform row spacing
- edge distance = 3.75 inches

\[
Z_{GT'} = \frac{Z_{RT'}}{2} + F_t A_{\text{group-net}}
\]

\[
= \frac{29,400}{2} + 625(3.5) [11.25 - 2(3.75) - (0.875 + 0.0625)]
\]

\[
= 20,850 \text{ lbs.} > 20,000 \text{ lbs. Demand OK}
\]

The design is still acceptable. Design is net section critical.

Engineering Calculations

Group Tear-out Capacity

Assuming a uniform row spacing, and edge distance of 3.75 inches, calculate group tear-out capacity as:

\[
Z_{GT} = \frac{Z_{RT}}{2} + F_t A_{\text{group-net}} = \frac{29,400}{2} + 625(3.5)[11.25 - 2(3.75) - (0.9375)] = 20,850 \text{ lbs.}
\]

Note that Group_{net} is the net area between the outer rows in the group, which is why the bolt holes are subtracted out.

The design is still acceptable. We have met all three checks.
**AMERICAN FOREST & PAPER ASSOCIATION**  
American Wood Council  
*Engineered and Traditional Wood Products*

---

**Local Stresses in Fastener Groups**  
*Example Solution*

---

*Exploration:* maximum spacing between rows

Using *Appendix E* provisions:

- **Group Tear-out Capacity**
  
  Assume:
  - uniform row spacing
  - edge distance = 1.31 inches

  \[
  Z_{GT} = Z_{RT}'/2 + F_t A_{\text{group-net}}
  \]

  \[
  = (29,400)/2 + 625(3.5)[11.25 - 2(1.31) - 0.875 - 0.0625]
  \]

  \[
  = 31,527 \text{ lbs.} > 20,000 \text{ lbs. Demand OK}
  \]

  The design is still acceptable. Capacity dramatically increases with increased row spacing.

---

What happens if spread out the rows to the minimum permissible edge distance of 1.5 D?

**Engineering Calculations**

**Group Tear-out Capacity**

Assuming a uniform row spacing and edge distance of 1.31 inches, calculate group tear-out capacity as:

\[
Z_{GT} = Z_{RT}'/2 + F_t A_{\text{group-net}} = (29,400)/2 + 625(3.5)[11.25 - 2(1.31) - (0.9375)] = 31,527 \text{ lbs}
\]

...a dramatic capacity increase at that!
Local Stresses in Fastener Groups

Example Solution

**Exploration:** minimum spacing between rows

Using Appendix E provisions:

- **Group Tear-out Capacity**

  Assume:
  - uniform row spacing
  - inter-row distance = 1.31 inches

  \[
  Z_{GT}' = \frac{Z_{RT}'}{2} + F_t A_{group-net} \\
  = \frac{29,400}{2} + 625(3.5)(1.31) \\
  = 17,566 \text{ lbs.} < 20,000 \text{ lbs. Demand NG}
  \]

  *The design is unacceptable. Spacing between rows is too tight!*

Well then, what happens if we space the rows really close together on the NDS minimum spacing?

**Engineering Calculations**

**Group Tear-out Capacity**

Assuming a uniform row spacing and inter-row distance of 1.31 inches, calculate group tear-out capacity as:

\[
Z_{GT} = \frac{Z_{RT}}{2} + F_t A_{group-net} = \frac{29,400}{2} + 625(3.5)(1.31) = 17,566 \text{ lbs}
\]

Not good - in fact dangerous! Message: **spread out the fasteners!**
Chapter 10 – Mechanical Connections

• Adjustment factors

- $C_g$
- $C_{\Delta}$
- $C_d$
- $C_{eg}$
- $C_{st}$
- $C_{di}$
- $C_{tn}$

As in the other chapters, adjustment factors unique to mechanical connections are described here for both ASD and LRFD processes.
Group Action Factor, $C_g$

Larger fasteners

- group action factor $C_g$
  - equation calculation
  - NDS tables

accounts for load distribution within the connection
- tabulated values still exist in the NDS
- can calculate your own group factor if outside the tabulated table range

The Group Action Factor $C_g$ provided in the NDS for connections involving large diameter fasteners often causes a lot of confusion.

Nominal lateral design values for split ring connectors, shear plate connectors, bolts with D less than or equal to 1”, or lag screws in a row are multiplied by $C_g$.

There are two ways to determine $C_g$: tables and calculation.
Group Action Factor, $C_g$

$C_g$ definitions:

- **row of fasteners:**
  - 2 or more split ring or shear plate connector units aligned in the direction of load
  - 2 or more bolts of same diameter loaded in direction of load
  - 2 or more lag screws of same type and size loaded in direction of load

Let’s first review $C_g$ terms.
Group Action Factor, $C_g$

What is a row?

Determining numbers of rows can be tricky…here are some diagrams to assist. Using the ratios in the diagrams helps determine the number of rows.
Group Action Factor, $C_g$

- **Equation method**

$$C_g = \left[ \frac{m(1 - m^{2n})}{n(1 + R_{EA}m^n)(1 + m) - 1 + m^{2n}} \right] \left[ \frac{1 + R_{EA}}{1 - m} \right]$$

where:

- $R_{EA} = \text{the lesser of } \frac{E_sA_s}{E_mA_m} \text{ or } \frac{E_mA_m}{E_sA_s}$

- $m = u - \sqrt{u^2 - 1}$

- $u = 1 + \gamma s \left[ \frac{1}{E_mA_m} + \frac{1}{E_sA_s} \right]$  

This is the calculation equation for $C_g$. 
## Group Action Factor, $C_g$

- Load / slip modulus, $\gamma$ (lb/in)

\[
D = \text{diameter of bolt of lag screw (in)}
\]

\[
\gamma \text{ (lb/in)}
\]

<table>
<thead>
<tr>
<th>Type</th>
<th>$\gamma$ (lb/in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bolts, lag screws: wood-to-metal connections</td>
<td>(270,000)$D^{1.2}$</td>
</tr>
<tr>
<td>Bolts, lag screws: wood-to-wood connections</td>
<td>(180,000)$D^{1.5}$</td>
</tr>
<tr>
<td>2 ½” split ring</td>
<td>400,000</td>
</tr>
<tr>
<td>2 5/8” shear plate</td>
<td>400,000</td>
</tr>
<tr>
<td>4” split ring</td>
<td>500,000</td>
</tr>
<tr>
<td>4” shear plate</td>
<td>500,000</td>
</tr>
</tbody>
</table>

The calculation depends to a degree on the load-slip relationship between the fastener and the holding material(s). The NDS tabulates the load-slip modulus for various installations as shown here. For fasteners into concrete, wood-to-wood values are used as a reasonably conservative approach.
Group Action Factor, $C_g$

- **Equation method Example**

  Find $C_g$ for two rows of 1” diameter bolts spaced 4” apart in a wood-to-wood double shear splice connection using 2x12’s for main and side members.

  
  \[
  \gamma = 180000 \frac{\text{lbf}}{\text{in}^{2.5}} \cdot D^{1.5} \quad \gamma = 1.8 \times 10^5 \frac{\text{lbf}}{\text{in}}
  \]

  \[
  \begin{align*}
  A_m &= 16.875 \text{ in}^2 \\
  \frac{A_m}{A_s} &= 0.5 \\
  E_m &= 14000000 \text{ psi} \\
  E_s &= 1400000 \text{ psi} \\
  s &= 4 \text{ in} \\
  n &= 10 \\
  D &= 1 \text{ in} \\
  A_m &= 1.5 \text{ in} \cdot 11.25 \text{ in} \\
  A_s &= 2 \cdot 1.5 \text{ in} \cdot 11.25 \text{ in}
  \end{align*}
  \]

  Here is an example of a calculation run for $C_g$. The problem set-up and material data are featured here.
Group Action Factor, $C_g$

- **Equation method Example**

$$C_g \text{ equation} \quad u := 1 + \gamma \cdot \frac{s}{2} \left( \frac{1}{E_g A_s} \right) \quad u = 1.023$$

$$m := u - \sqrt{u^2 - 1} \quad m = 0.808$$

$$R_{EA} := \min \left( \frac{A_m}{E_m A_g}, \frac{A_s}{E_s A_m} \right) \quad R_{EA} = 0.5$$

$$C_g := \frac{m \left( 1 - m^2 \cdot n \right)}{n \cdot \left( 1 + R_{EA} \cdot m \right) \cdot \left( 1 + m^2 \cdot n \right) \cdot \left( 1 - m^2 \cdot n \right)} \left( \frac{1 + R_{EA}}{1 - m} \right) \quad C_g = 0.669$$

...then the equation is run for a $C_g$ of 0.669.
Group Action Factor, $C_g$

- Table method

$A_m =$ gross x-sectional area of main member, $\text{in}^2$

$A_s =$ sum of gross x-sectional areas of all side members, $\text{in}^2$

We can use the table method for the same problem since criteria fits the bounds of the tables in the NDS. If the bounds are exceeded, then calculation is the only approach.
Group Action Factor, $C_g$

- **Table method**
  - **Example**
    - $A_s/A_m > 1.0$, so use $A_m/A_s = 0.5$ to enter column 1 of the table
    - also, use $A_m$ for column 2 according to Note 1 ($A_m = 16.875 \text{ in}^2$)
    - read across to column for 10 fasteners in a row
    - interpolate $C_g = 0.665$

The steps here are explained in the slide. The table provides a $C_g$ result of 0.665, consistent with what we calculated.
The Group Action Factor does not apply to sill plates because such loads are not necessarily axial with the plate.
Chapter 10 – Mechanical Connections

• Adjustment Factors – Table 10.3.1
  – for all fasteners

<table>
<thead>
<tr>
<th>Lateral Loads</th>
<th>ASD Only</th>
<th>ASD and LRFD</th>
<th>LRFD Only</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dowel-type Fasteners</td>
<td>Z = Z x</td>
<td>C_0 C_1 C_2 C_3</td>
<td>C_4 C_5</td>
</tr>
<tr>
<td>SFN Ring and Strip Fasteners</td>
<td>P = P x</td>
<td>C_0 C_1 C_2 C_3</td>
<td>C_4 C_5</td>
</tr>
<tr>
<td>Timbered Connectors</td>
<td>Q = Q x</td>
<td>C_0 C_1 C_2 C_3</td>
<td>C_4 C_5</td>
</tr>
<tr>
<td>Metal Plate Connectors</td>
<td>Z = Z x</td>
<td>C_0 C_1 C_2 C_3</td>
<td>C_4 C_5</td>
</tr>
<tr>
<td>Spike Grisels</td>
<td>Z = Z x</td>
<td>C_0 C_1 C_2 C_3</td>
<td>C_4 C_5</td>
</tr>
</tbody>
</table>

Withdrawal Loads

Nails, spikes, lag screws, wood screws, and drift pins

W = W x C_0 C_1 C_2 C_3 C_4 C_5 K_1 f_x L_1

1. The load distribution factor C_0 applies to other connections in Table 10.3.1.
2. The load distribution factor C_0 applies to other load combinations in Table 10.3.1.
3. The load distribution factors C_0, C_1, and C_2 apply to other load combinations in Table 10.3.1.
4. The load distribution factor C_3 applies to other load combinations in Table 10.3.1.
5. The load distribution factor C_4 applies to other load combinations in Table 10.3.1.

Table 10.3.1 applies to all mechanical fasteners.
Chapter 10 – Mechanical Connections

• Wet Service Factor, $C_M$

Another adjustment factor that is important to connections is the wet service factor, $C_M$. 
Connection strength varies with wood EMC, and the NDS has provisions to address this effect - the Wet Service Factor $C_M$ that affects connection Z values. Two conditions of EMC at fabrication and in-service are important: \(<19\%\) and \(>19\%\). The latter condition includes both continuous or occasional exposure at moisture levels greater than 19%. The designer must assess the environmental situation to see which occurs when.

At MC levels above 19%, wood is more elastic, and wood strength properties reduce somewhat. When wood connections are fabricated using wood with high MC’s over 19%, and MC levels are expected to drop to final values below 19% in service, considerable shrinkage takes place around the fasteners, and grouped fasteners are especially vulnerable in initiating tension perp failures; hence the low value of $C_M = 0.4$. A design penalty? Perhaps. But there is a workaround...
NDS Provisions since 1997

Wet Service Factor, $C_M$ for connection Z values

Saturated

19% MC

fabrication MC

in-service MC

Dry

$C_M = 0.4$  Lateral Load

$C_M = 1.0$  if:

1 fastener

2+ fasteners

split splice plates

The NDS has a detailing provision for the 0.4 value on bolt and lag screw connections that can provide full fastener capacity ($C_M = 1.0$).

Use:
- one fastener only, or
- two or more fasteners placed in a single row parallel to grain, or
- use fasteners placed in two or more rows parallel to grain with separate splice plates for each row.

Minimum distances between fasteners, and fasteners and edges still need to be maintained. This detailing allows the wood to change shape across the grain on drying without being hung up on the fasteners - the fasteners can move with the wood.
Good detailing to allow for shrinkage pays off...

<table>
<thead>
<tr>
<th>Direction of Loading</th>
<th>Minimum Spacing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parallel to Grain</td>
<td>1.5D</td>
</tr>
</tbody>
</table>
| Perpendicular to Grain:
  when $f/D \leq 2$   | $2.5D$           |
  when $2 < f/D < 6$   | $(5 \cdot f + 1D)/8$ |
  when $f/D \geq 6$    | $5D$            |

1. The $D$ value used to determine the minimum spacing between rows shall be the lesser of:
   1. The nominal thickness of boards in contact
   2. $2(1 + f/3)D$

2. The spacing between outer rows of fasteners shall be the smaller of a single splice shear slot size equal to 3/8" (see Figure 11I).

Keep spacing between rows of bolts on a common splice plate to less than 5 inches to avoid splitting the wood due to changes in equilibrium moisture content. Good detailing on connections often pays off to avoid shrinkage-related problems.
Chapter 11 – Connections

- Yield theory basis
  - ASD and LRFD accommodated through Table 10.3.1

Chapter 11 deals with dowel-type fasteners. Table 10.3.1 contains ASD and LRFD adjustment factors for various fastener types and loading directions as described.
In terms of dowel-type fasteners, the simplest is the nail. Unfortunately, there are many variations of a nail as shown here, with a variety of names, even variations in the way they are installed. Nail capacities are tabulated for only some of them, such as box and common nails since these are standardized to some degree based on shank diameter - the driver of the capacity tables. Other nail types are standardized in ASTM F1667. However, there has not been a demand for standardized design values, so they have not been tabulated. The NDS equations can also be used to develop design values if proper inputs are developed by the designer per the NDS.
Appendix L of the new 2005 NDS describes and details various nail types.
Nail Types and Designations

Nail capacity tables in 2005 NDS

In terms of shank diameter, same-designation box, common, and sinker nails are NOT necessarily the same: a 6D common is similar to an 8D box, for example. Shank diameters differ among same-designation nail types. This table is an excerpt from the new 2001 NDS nail capacity tables that shows side by side designation comparisons of common, box and sinker nails based on shank diameter. What is important in nail capacity determination is nail shank diameter as seen in the capacity formulae on which the table is based. APA has similar tables 8.11A and 8.11B in the APA Engineered Wood Handbook. These are really handy tables for a lot of good reasons.
Fastener Values

### Included in U.S. design literature

<table>
<thead>
<tr>
<th>Fastener Type</th>
<th>Reference</th>
<th>NER’s are now called:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bolts</td>
<td>NDS or NER</td>
<td>• ESR’s or</td>
</tr>
<tr>
<td>Lag Screws</td>
<td>NDS or NER</td>
<td>• ES’s or</td>
</tr>
<tr>
<td>Wood Screws</td>
<td>NDS or NER</td>
<td>• ICC Evaluation</td>
</tr>
<tr>
<td>Nails &amp; Spikes</td>
<td>NDS or NER</td>
<td>Service Reports</td>
</tr>
<tr>
<td>Split Ring Connectors</td>
<td>NDS</td>
<td></td>
</tr>
<tr>
<td>Shear Plate Connectors</td>
<td>NDS</td>
<td></td>
</tr>
<tr>
<td>Drift Bolts &amp; Drift Pins</td>
<td>NDS</td>
<td></td>
</tr>
<tr>
<td>Metal Plate Connectors</td>
<td>NER</td>
<td></td>
</tr>
<tr>
<td>Hangers &amp; Framing Anchors</td>
<td>NER</td>
<td></td>
</tr>
<tr>
<td>Staples</td>
<td>NER</td>
<td></td>
</tr>
</tbody>
</table>

Design values for connections loaded in single and double shear tabulated in the NDS Chapter 11 are based on the fastener bending yield strengths, $F_{yb}$, given in the footnotes of the respective tables. Other fastener bending yield strengths may be used with the yield mode equations in these Chapters to calculate design values for the connections involved. However, bolts, lag screws and wood screws must conform to the applicable ANSI/ASME Standard referenced for these fasteners in 8.1.1, 9.1.1 and 11.1.1; and nails and spikes must meet the requirements specified in 12.1.2. Bending yield strength of nails and spikes may be determined in accordance with ASTM F1575-95 (see Appendix I of the NDS).
Fasteners need to be resistant to static and repetitive bending to be effective in transferring load. Static fastener capacities are determined from a center-point bending yield test....
Yield Limit Equations

Fastener Bending Yield Values, $F_{yb}$

<table>
<thead>
<tr>
<th>Fastener Type</th>
<th>Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bolts</td>
<td>$0.5 (F_y + F_u)$</td>
</tr>
<tr>
<td>Common Wire Nails</td>
<td>$130,400 - 213,900 D$</td>
</tr>
</tbody>
</table>

...that results in the following relationships for bolts and common wire nails. The values for common nails are standardized in ASTM F1667. The values calculated using the empirical equation here may be higher or lower than the standardized value for a given fastener diameter.
Yield Limit Equations

Table 11.3.1A  Yield Limit Equations

<table>
<thead>
<tr>
<th>Yield Mode</th>
<th>Single Shear</th>
<th>Double Shear</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>( Z = \frac{D_1 + F_m}{R_1} ) (11.3-1)</td>
<td>( Z = \frac{D_1 + F_m}{R_2} ) (11.3-7)</td>
</tr>
<tr>
<td>II</td>
<td>( Z = \frac{D_1 D_2}{R_1} ) (11.3-2)</td>
<td>( Z = \frac{2D_1 D_2}{R_2} ) (11.3-8)</td>
</tr>
<tr>
<td>III</td>
<td>( Z = \frac{D_1 D_2}{R_1 (L - 2R_1 R_2)} ) (11.3-3)</td>
<td>( )</td>
</tr>
<tr>
<td>IV</td>
<td>( Z = \frac{D_1}{R_1 \sqrt{3(L + R_1)}} ) (11.3-5)</td>
<td>( Z = \frac{2D_1 D_2}{(D^2 + R_1 R_2)^{3/2}} ) (11.3-10)</td>
</tr>
</tbody>
</table>

Equations have been developed as part of the 2005 NDS for four possible yield modes that dowel fasteners can take on. Yield equations for connections in double shear, in addition to the single shear set, are tabulated.

Notes:
- \( k_1 = \frac{M_1 R_1^2 + 2R_2^2 (L + R_1 R_2) + R_2^2 R_1^2 - R_1^2 L}{2R_1^2 R_2^2} \)
- \( k_2 = \frac{L - 1}{2} \left( 1 - \frac{R_2^2}{M_1 R_1^2} \right) \)
- \( k_3 = \frac{1}{2} \left( \frac{R_1^2}{M_1 R_1^2} \right) \)
- \( D = \) diameter, in. (see 11.3-6)
- \( F_{m,sk} = \) dowel bending yield strength, psi
- \( R_1 = \) reduction term (see Table 11.3.1B)
- \( R_2 = \) \( F_{m,sk} \)
- \( R_m = \) main member dowel bearing length, in.
- \( L_n = \) side member dowel bearing length, in.
- \( F_{m,sk} = \) main member dowel bearing strength, psi (see Table 11.3.2)
- \( F_{m,sk} = \) side member dowel bearing strength, psi (see Table 11.3.2)
The NDS considers six yield limit equations for dowel connectors. Reduction terms, appearing in the denominator of the NDS yield equations, vary by dowel type. To facilitate a general format for the six yield limit equations, reduction terms have been separated from the yield equations and are shown here for bolts and lag screws loaded parallel and perpendicular to the grain....
Fastener Penetration

<table>
<thead>
<tr>
<th>Fastener Type</th>
<th>Minimum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lag Screws</td>
<td>4D</td>
</tr>
<tr>
<td>Wood Screws</td>
<td>6D (<em>increased from 4D</em>)</td>
</tr>
<tr>
<td>Nails &amp; Spikes</td>
<td>6D</td>
</tr>
</tbody>
</table>

To be effective in holding and to develop its full capacity, fasteners must achieve a minimum penetration depth into the holding member as indicated in the table. In the 2001 NDS, the minimum penetration for wood screws was increased to 6D from 4D in the 1997 NDS.
On toe-nailing, the NDS provides guidance here:

11.1.5.3 Reference design values herein apply to nailed and spike connections either with or without bored holes. When a bored hole is desired to prevent splitting of wood, the diameter of the bored hole shall not exceed 90% of the nail or spike diameter for wood with $G > 0.6$, nor 75% of the nail or spike diameter for wood with $G \leq 0.6$ (See Table 11.3.2A).

It is important to understand that toe nails only resist loads in certain directions and thus are not recommended when the load application can be from several directions as shown.
Chapter 11 – Connections

- Capacity definitions
  - $Z_{||}$
  - $Z_{\perp}$
  - $Z_{m,\perp}$
  - $Z_{s,\perp}$
  - $Z_{\perp}$

Chapter 11 provides guidance to help define capacity definitions and the variables leading to determination of capacity. One of the most important is direction of applied load with respect to the grain of the wood. To determine this, knowledge of the load path through all the components of the connection is key. An exploded view of the assembly often is useful to the designer to determine the load path. From the load path, load direction with respect to wood grain can be determined for each wood component in the connection. This leads to correct extraction of the respective capacity value from the NDS fastener capacity tables.
Determining numbers of rows can be tricky…here are some diagrams to assist. Key here are the “a” ratios to help define the rows.
This diagram applies for perpendicular to grain loading. The NDS does contain minimum fastener spacing, edge, and end distance rules for fastener placement. Again, load direction can play a role in their determination. Correct fastener placement to develop full design capacity of the fastener may also be governed by the provisions of Appendix E on checking local stresses.
Nail Capacity Calculations

• *Example:* Shear Wall Chord Ties with Nails
  – LRFD the hard way
  – ASD & LRFD the easy way

As a connection example with the new 2005 NDS, let’s work this nailed shear wall chord tie design first by using LRFD yield equations, and then using the 2005 NDS tables for both LRFD and ASD. Then we’ll compare results.
Nailed Tension Tie

How many nails for this connection?

Design connection ties between first and second floor shear wall chords. Floor framing consists of 9.5" deep pre-fabricated wood I-joists. Walls are 2x6, dry Douglas Fir-Larch studs spaced at 16" OC. The specified wind overturning force is 2.4 kips.

The first practical consideration faced by a designer in this case is to choose a fastener type. Many proprietary pre-fabricated metal connectors are available to make this connection, (see AF&PA Guideline for Pre-Engineered Metal Connectors). However, a connection can be designed that will use commonly available, non-proprietary, components.
Nailed Tension Tie - LRFD

**Try:**
- ASTM A653 Grade metal strap 16 gage x 2.5" wide
- 2 rows staggered 10d common nails

**Adjustment Factor:** *Penetration*

\[
l_p = 3.0\ \text{"} > 12D
\]

\[
C_d = 1.0
\]

\[
\begin{align*}
I_s &= 0.06" \\
D &= 0.148" \\
I_p &= 3.0" \\
F_{yb} &= 90 \text{ ksi} \\
F_{em} &= 4.65 \text{ ksi} \\
F_{es} &= 61.85 \text{ ksi} \\
R_e &= \frac{F_{em}}{F_{es}} \\
&= \frac{4.65}{61.85} \\
&= 0.0752 \\
R_d &= 2.2
\end{align*}
\]

Material design parameters are listed here. Since the strap is so thin, the penetration adjustment factor produces a value of 1.0.
Nailed Tension Tie - LRFD

*Mode III_s controls; factored lateral strength $\lambda \phi z' Z'*:

$$k_3 = -1 + \frac{2(1+R_s) + 2F_{yo}(2+R_s)D^2}{3F_{em}I_s}$$

$$= -1 + \frac{2(1.0752) + 2(90)(2.0752)(0.148)^2}{0.0752 \times 3(4.65)(0.06)^2}$$

$$k_3 = 12.84$$

Unfactored unit capacity:

$$Z = \frac{k_3Dl_sF_{em}}{(2+R_e)R_d}$$

$$= \frac{(12.84)(0.148)(0.06)(4.65)}{(2.0752)2.2}$$

$$Z = 0.116\text{ kips}$$

First, calculate the Unfactored unit capacity $Z$ of the nail from Mode III_s (2005 NDS Table 11.3.1A Single Shear Mode III_s Eqn 11.3-5). The numerical result will be an ASD value.

Normally, design equations of Chapter 11 require calculation of all four yield modes (6 equations) to determine the controlling mode. Here, it turns out that Mode III_s controls which is shown here in detail for convenience.
Nailed Tension Tie - LRFD

*Mode III_* controls; factored lateral strength \( \lambda \phi_z K_F Z' \):

Factored unit capacity:

\[
K_F = \frac{2.16}{\phi_z} = \frac{2.16}{0.65} = 3.32
\]

\[
\lambda \phi_z K_F Z' = 1.0(0.65)(3.32)(0.116) = 0.250 \text{ kips}
\]

Factored demand:

\[
\alpha_{ol} W_{ol} = 1.5(2.4) = 3.6 \text{ kips}
\]

Number of nails:

\[
n = \frac{3.6}{0.250} = 14.4 \rightarrow 15 \text{ nails}
\]

Use 15 - 10d nails per side, or 2 rows of 8 each.

…then convert it to LRFD using the provisions of Appendix N to get the factored resistance of one nail.

Determine the factored demand on the tie from wind overturning using the appropriate load factor. Divide the demand into the resistance (both factored) to arrive at the number of nails required: 15 per side of the joint, in this case. Increase to 16 (2 rows of 8 each) for ease of installation.
Nailed Tension Tie – LRFD & ASD

*Try:*  
- ASTM A653 Grade metal strap  
  16 gage x 2.5” wide  
- 2 rows staggered 10d common nails

Now let’s try it again, this time, the easy way using the 2005 NDS tables for LRFD.

Material design parameters are listed here. Since the strap is so thin, the penetration adjustment factor produces a value of 1.0.
Nailed Tension Tie – LRFD & ASD

2005 NDS Table 11P

Z = 116 lbs

First, calculate the unfactored unit capacity Z of the nail (ASD value) from Mode III_s (2005 NDS Table 11P)...
Nailed Tension Tie – LRFD & ASD

2005 NDS Table 11P notes & Table 10.3.1

<table>
<thead>
<tr>
<th>ADJUSTMENT FACTORS</th>
<th>LRFD</th>
<th>ASD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Time dependent</td>
<td>λ = 1.0 Table N3</td>
<td>C_D = 1.6</td>
</tr>
<tr>
<td>Wet service</td>
<td>C_M</td>
<td>1.0</td>
</tr>
<tr>
<td>Temperature</td>
<td>C_t</td>
<td>1.0</td>
</tr>
<tr>
<td>Group Action</td>
<td>C_g</td>
<td>1.0</td>
</tr>
<tr>
<td>Geometry</td>
<td>C_l</td>
<td>1.0</td>
</tr>
<tr>
<td>End grain</td>
<td>C_\text{eg}</td>
<td>1.0</td>
</tr>
<tr>
<td>Diaphragm</td>
<td>C_{\text{di}}</td>
<td>1.0</td>
</tr>
<tr>
<td>Toe nail</td>
<td>C_{\text{tn}}</td>
<td>1.0</td>
</tr>
<tr>
<td>Penetration</td>
<td>3” &gt; 10D</td>
<td>1.0</td>
</tr>
</tbody>
</table>

Format conversion

<table>
<thead>
<tr>
<th>Resistance</th>
<th>LRFD</th>
<th>ASD</th>
</tr>
</thead>
<tbody>
<tr>
<td>K_F</td>
<td>2.16 / \phi_z = 3.32 Table N1</td>
<td></td>
</tr>
<tr>
<td>\phi_z</td>
<td>0.65</td>
<td>Table N2</td>
</tr>
</tbody>
</table>
Nailed Tension Tie – LRFD & ASD

**DEMAND – Wind Overturning**

<table>
<thead>
<tr>
<th>LRFD</th>
<th>ASD</th>
</tr>
</thead>
<tbody>
<tr>
<td>( w = \alpha_{ot} W_{ot} )</td>
<td>( w = W_{ot} )</td>
</tr>
<tr>
<td>( = (1.6)(2,400) )</td>
<td>( = 2,400 \text{ lbs} )</td>
</tr>
</tbody>
</table>

\( = 3,840 \text{ lbs} \)

Factor the loads for LRFD, noting that this is for wind uplift only (no dead load here)…
Nailed Tension Tie – LRFD & ASD

**CAPACITY**

*Safety Limit State*

<table>
<thead>
<tr>
<th>LRFD</th>
<th>ASD</th>
</tr>
</thead>
<tbody>
<tr>
<td>Z' = Z φ₂ λ Kₚ (all C factors)</td>
<td>Z' = Z C₀ (all C factors)</td>
</tr>
<tr>
<td>= (116)(0.65)(1.0)(3.32)(1.0)</td>
<td>= (116)(1.6)(1.0)</td>
</tr>
<tr>
<td>= 250 lbs</td>
<td>= 186 lbs</td>
</tr>
</tbody>
</table>

**Nails Needed**

<table>
<thead>
<tr>
<th>n = w / Z'</th>
<th>n = w / Z'</th>
</tr>
</thead>
<tbody>
<tr>
<td>= (3,840) / (250)</td>
<td>= (2,400) / (186)</td>
</tr>
<tr>
<td>= 15.4 → 16 nails</td>
<td>= 12.9 → 13 nails</td>
</tr>
</tbody>
</table>

Determine the LRFD and ASD capacities accordingly for one nail, then dividing into the demand, determine the number of nails required. You’ll notice a difference in the result between the ASD and LRFD methods. You’ll also notice that the numbers in LRFD are bigger.
### Nailed Tension Tie – LRFD & ASD

#### CAPACITY

*Safety Limit State*

<table>
<thead>
<tr>
<th>LRFD</th>
<th>ASD</th>
</tr>
</thead>
<tbody>
<tr>
<td>$w \leq Z'$</td>
<td>$W \leq Z'$</td>
</tr>
<tr>
<td>3,840 lbs $\leq 4,000$ lbs</td>
<td>2,400 lbs $\leq 2,976$ lbs</td>
</tr>
</tbody>
</table>

### Demand / Capacity Ratio

<table>
<thead>
<tr>
<th>16 Nails</th>
<th>13 Nails</th>
</tr>
</thead>
<tbody>
<tr>
<td>Demand / Capacity Ratio</td>
<td>0.96</td>
</tr>
</tbody>
</table>

If we ratio the results for ASD and LRFD on the basis of demand over capacity, we see that LRFD is more conservative than ASD. So, why the discrepancy?
Nailed Tension Tie – LRFD & ASD

- Why the ASD / LRFD discrepancy?
  - 2005 NDS format conversion does not benefit LRFD in the Wind Only case
  - Real benefits are realized with combined multiple transient loads (ie. wind + snow + live) – examine load combination cases and LRFD load factors in addition to relative magnitudes of the loads themselves

First, the format conversion from ASD to LRFD in the 2005 NDS does not benefit LRFD in the Wind Only case. However, real benefits are realized with combined multiple transient loads (ie. wind + snow + live). If the designer examines load combination cases and LRFD load factors in addition to relative magnitudes of the loads themselves, it will be easy to see how these will combine effectively to provide a more realistic assessment of load demand over ASD, since ASD demand is usually a straight summation of load.
Chapter 12 – Split Rings and Shear Plates

• Capacity tables - unchanged

Chapter 12 features information on split ring and shear plate connectors. ASD capacity tables have not changed for many editions of the NDS, and this is still true for the 2005 edition. These devices are high capacity fasteners meant for use in very large members and member cross-sections.
Chapter 13 is for timber rivets, a very useful and effective device for connecting members of small or large cross-sections. The capacity tables remain unchanged from the 2001 NDS.
Chapter 13 – Timber Rivets

• Capacity tables to 10 lb resolution

<table>
<thead>
<tr>
<th>Rivet Length</th>
<th>P_y (lbs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>11/2&quot; 2&quot; 1&quot;</td>
<td>14100 17050 19700</td>
</tr>
<tr>
<td>1&quot; 1&quot; 1&quot;</td>
<td>19800 24770 27950</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Rivet Length</th>
<th>P_y (lbs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>11/2&quot; 2&quot; 1&quot;</td>
<td>20870 24770 27950</td>
</tr>
<tr>
<td>1&quot; 1&quot; 1&quot;</td>
<td>22620 26520 30000</td>
</tr>
</tbody>
</table>

Table 13.2.1A Reference Wood Capacity Design Values Parallel for Timber Rivets

<table>
<thead>
<tr>
<th>Member Thickness in.</th>
<th>Rivets per row</th>
<th>No. of rows per side</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>2000 4000 7600</td>
<td>13770 14100 17050 19700 22600</td>
</tr>
<tr>
<td>4</td>
<td>3010 6040 12100 18150 24200 29250 34300 39350 44400</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>4040 8080 13770 18300 23830 29360 34890 39420 44950 50480</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>5110 10220 15970 20540 25110 30680 35250 39820 44390 49960</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>6180 12390 17970 22560 27150 31740 36330 40920 45510 50090</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>7250 14560 19970 24550 29140 33730 38320 42910 47500 52090</td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>8320 16730 21970 26550 31140 35730 40320 44910 49500 54090</td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>9390 18900 26970 31550 36140 40730 45320 49910 54500 59090</td>
<td></td>
</tr>
<tr>
<td>18</td>
<td>10460 21070 27970 32550 37140 41730 46320 50910 55500 60090</td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>11530 23240 28970 33550 38140 42730 47320 51910 56500 61090</td>
<td></td>
</tr>
</tbody>
</table>

The capacity tables still maintain their 10 lb resolution to permit use in smaller or lower load demand connections.
Chapter 13 – Timber Rivets

• Many applications

Timber rivet connections have been used in Canada for several decades. The NDS design criteria introduced in Chapter 13 of the NDS apply to joints with steel side plates for either Southern Pine or Western Species glued laminated timber. The term "timber rivet" was chosen to accommodate future application to sawn lumber as well.

Provisions of the Specification are applicable only to timber rivets that are hot-dipped galvanized. Rivets are made with fixed shank cross-section and head dimensions (Appendix M) and vary only as to length.

Because of the species test results and property values used to develop the rivet bending and wood capacity equations, use of design values based on the provisions of 13.2.2 should be limited to Douglas fir-Larch and southern pine glued laminated timber. The NDS presently limits use of timber rivets to attachment of steel side plates to glued laminated timber.
Chapter 13 – Timber Rivets

grow on trees?

TIMBER RIVETS - as grown in Canada for strength.
Provisions of the Specification are applicable only to timber rivets that are hot-dipped galvanized. Rivets are made with fixed shank cross-section and head dimensions (Appendix M) and vary only as to length.
Timber Rivet System

Perforated steel plates

Plates also have a fixed hole pattern geometry. Hole sizes are chosen deliberately to firmly hold and lock the head of the rivet in position, preventing the rivet from rotating next to the plate, to fully develop a cantilever action for the rivet shank embedded in the wood.
Timber Rivet System

One or two-sided connection

Rivet connections can be made from one or both sides of a member.
Similar rules apply as before in properly and safely loading the wood.
Angle to grain capacity values are also provided in the NDS.
Chapter 14 begins the sections of the NDS dealing with special provisions. Chapter 14 on shear walls and diaphragms covers general requirements for framing members, fasteners, and sheathing. The reference document for the design process of shear walls and diaphragms is AF&PA’s Special Design Provisions for Wind and Seismic Supplement.
The Special Design Provisions for Wind and Seismic Supplement is the scope of another course. In addition to design process for shear wall and diaphragm elements, the Supplement includes reference design values for a wide variety of panel products. The table of contents of the supplement is shown here.
Chapter 15 – Special Loading

• Built-up columns
  – Revised to correct limitation on short built-up columns

15.3.2.2.... Each ratio shall be used to calculate a column stability factor, $C_p$, per section 15.3.2.4 and the smaller $C_p$ shall be used in determining the allowable compression design value parallel to grain, $F_c'$, for the column. $F_c'$ for built-up columns need not be less than $F_c'$ for the individual laminations designed as individual solid columns per section 3.7.

Chapter 15 on Special Loading describes various topics related to loads such as: lateral distribution of a concentrated load, spaced columns, built-up columns, and wood columns with side loads and eccentricity.

The 2005 NDS revises a limitation on short built-up columns whereby the designer can use the lesser of the column capacity reduced on the basis of slenderness of the entire cross-section, and the column capacity an individual lamination multiplied by the number of laminations.
Chapter 16 on the design of exposed wood members to meet building code prescribed fire endurance times first introduced in the 2001 NDS is only applicable to ASD design.
Chapter 16 – Fire (ASD)

- Fire resistance up to **two hours**
  - Columns
  - Beams
  - Tension Members
  - Combined Loading

- Additional special provisions for glulam

16.2.4 Special Provisions for Glued Laminated Timber Beams

For glued laminated timber bending members given in Table 5A and rated for 1-hour fire endurance, an outer tension lamination shall be substituted for a core lamination on the tension side for unbalanced beams and on both sides for balanced beams. For glued laminated timber bending members given in Table 5A and rated for 1½ or 2-hour fire endurance, two outer tension laminations shall be substituted for two core laminations on the tension side for unbalanced beams and on both sides for balanced beams.

ASD provisions address tension, compression and bending members and members subjected to combined loading. Special provisions for glued laminated timber beams are also included.
Chapter 16 – Fire (ASD)

TR10
Calculating the Fire Resistance of Exposed Wood Members

Expands the uses for large, exposed wood members (tension, bending/compression, bending/tension members, decking)

Expands applicability of current methods to other EWP’s (SCL)

Expands use of large, exposed wood members to 2 hour fire endurance applications.

The basis for Chapter 16 is found in AF&PA’s document TR 10: Design of Fire Resistive Exposed Wood Members

This document also forms the technical basis for AF&PA’s DCA 2. It is complete with detailed explanation, test results, and comprehensive calculation examples.
Chapter 16 – Fire

• **Superior fire performance of heavy timbers**
  – attributed to the charring effect of wood

• **Benefits of charring**
  – an insulating char layer is formed
  – protects the core of the section

The physical basis for Chapter 16 is the charring characteristic of wood when subjected to fire. Charring of wood occurs at a measurable rate, and because of wood’s insulation properties, the cross-section interior remains capable of sustaining and carrying load.
Chapter 16 – Fire

• Experimental charring rates measured in various parts of the world appear to be consistent
  – North America - Standard fire endurance test ASTM E-119
  – many other countries - comparable fire exposure in ISO 834

• Effects of fire on adhesives
  – synthetic glues used in the manufacture of glulam do not adversely affect performance

Charring rates of wood under standard fire exposure conditions were measured in studies world-wide. Glued products did not perform any differently than their solid counterparts.
Mechanics-Based Design Method

- expands the use of large exposed wood members:
  - loading conditions
  - fire exposures
  - mechanical properties
  - stress interactions
  - expanded range of wood products

This design method is a rational approach that allows for exposed structural wood members to be used in structures that could be exposed to fire.
Design Considerations

• predicts reduced cross-sectional dimensions

• adjusts for charring at the corners

• accounts for the loss of strength and stiffness in the heated zone

The equations used in this method account for all the charring characteristics of a wood cross-section exposed to fire.
A standard terminology was established for describing the charred and uncharred section dimensions for two common fire exposures.
### Estimating Cross-sectional Dimensions due to Charring

<table>
<thead>
<tr>
<th>Exposure Type</th>
<th>Formula 1</th>
<th>Formula 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>4-Sided Exposure (i.e. columns)</td>
<td>$b = B - 2\beta t$</td>
<td>$d = D - 2\beta t$</td>
</tr>
<tr>
<td>3-Sided Exposure (i.e. beams)</td>
<td>$b = B - 2\beta t$</td>
<td>$d = D - \beta t$</td>
</tr>
<tr>
<td>2-Sided Exposure (i.e. decking)</td>
<td>$b = B - \beta t$</td>
<td>$d = D - \beta t$</td>
</tr>
</tbody>
</table>

where:
- $\beta$ is the char rate of the material
- $t$ is the fire exposure time

…which resulted in these relations for charred width and depth.
Model for Charring of Wood

- Nonlinear char model used - nominal linear char rate input.
- To account for rounding at corners and reduction of strength and stiffness of the heated zone, the nominal char rate values, $\beta_n$, are increased 20%.

$$\beta_{\text{eff}} = 1.2 \frac{\beta_n}{t^{0.187}}$$

where:
- $\beta_{\text{eff}}$ is the effective char rate (in/hr), adjusted for exposure time, $t$
- $\beta_n$ is the nominal linear char rate (in/hr), based on 1-hr exposure
- $t$ is the exposure time (hrs)

In terms of the charring characteristics of wood, this is the char model used.
Effective Char Rates and Char Layer Thicknesses (for $\beta_n = 1.5$ inches/hour)

<table>
<thead>
<tr>
<th>Required Fire Endurance (hr)</th>
<th>Effective Char Rate, $\beta_{\text{eff}}$ (in/hr)</th>
<th>Effective Char Layer Thickness, $\alpha_{\text{char}}$ (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-Hour</td>
<td>1.80</td>
<td>1.8</td>
</tr>
<tr>
<td>1½-Hour</td>
<td>1.67</td>
<td>2.5</td>
</tr>
<tr>
<td>2-Hour</td>
<td>1.58</td>
<td>3.2</td>
</tr>
</tbody>
</table>

...and these are the charring results based on a typical char rate of 1.5 inches per hour.
Design for Member Capacity

\[
\text{Dead Load + Live Load} \leq K \times \text{Allowable Design Capacity}
\]

where:

\[K\] is a factor to adjust from allowable design capacity to average ultimate capacity.

The factor, K, adjusts from allowable design capacity of the member to average ultimate capacity - the maximum capacity the member can physically sustain (no safety factors).
**Allowable Design Stress to Average Ultimate Strength Adjustment Factor**

<table>
<thead>
<tr>
<th>Member Capacity</th>
<th>K</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending Moment Capacity, in-lb.</td>
<td>2.85</td>
</tr>
<tr>
<td>Tensile Capacity, lb.</td>
<td>2.85</td>
</tr>
<tr>
<td>Compression Capacity, lb.</td>
<td>2.58</td>
</tr>
<tr>
<td>Beam Buckling Capacity, lb.</td>
<td>2.03</td>
</tr>
<tr>
<td>Column Buckling Capacity, lb.</td>
<td>2.03</td>
</tr>
</tbody>
</table>

This table lists the values of $K$ for various mode capacities to adjust to an ultimate strength basis.
General Comparison

- Given the theoretical derivation of the new mechanics-based design method, existing test results from fire tests of exposed, large wood members were compared against the model predictions.

- International and North American test data were reviewed.

The theoretical model for charring was checked against full scale tests from all over the world...
...and was found to be excellent agreement. Here is one such example where the model and test agreement were good for wood beams exposed on 3 sides.
And now, a detailed ASD design example, worked from start to finish.

Consider Douglas fir beams spanning 18 feet and spaced 6 feet apart. The beams support 100 psf live load and 15 psf dead load. Timber decking laterally braces the compression flange of the beams.

Size the beam for a 1 hour rating.
For the structural design of the beam, calculate the induced moment:

- **Beam load:**
  \[ w_{total} = s (q_{dead} + q_{live}) = (6')(15+100) = 690 \text{ plf} \]

- **Induced demand moment:**
  \[ M_{max} = w_{total} L^2 / 8 = (690)(18)^2 / 8 = 27,945 \text{ ft-lb} \]

**Solution:**

First, calculate the induced demand moment based on the tributary width of 6 feet (beam spacing).
Fire Design Example (ASD)

Select a 6-3/4" x 12" 24F-V4 Douglas-fir glulam beam
Tabulated bending stress, $F_b$, equal to 2400 psi

Calculate the beam section modulus:
$$S_b = BD^2/6 = (6.75)(12)^2 / 6 = 162.0 \text{ in}^3$$

Calculate the adjusted allowable bending stress:
Assuming: $C_D = 1.0$, $C_M = 1.0$, $C_t = 1.0$, $C_L = 1.0$, $C_V = 0.99$

$$F'_b = F_b C_D C_M C_t \text{ (lesser of } C_L \text{ or } C_V)$$
$$= 2400(1.0)(1.0)(1.0)(0.99)$$
$$= 2371 \text{ psi}$$

Pick a beam, calculate its section modulus from actual dimensions, and the adjusted allowable bearing stress of the material.
Fire Design Example (ASD)

Calculate the design resisting moment:
\[ M' = F'_b S_s = \frac{(2371)(162)}{12} = 32,009 \text{ ft-lb} \]

Structural Capacity Check: \[ M' > M_{\text{max}} \]

\[ 32,009 \text{ ft-lb} > 27,945 \text{ ft-lb} \]

Multiply the adjusted allowable bending stress by the section modulus to get the maximum resisting moment offered by your chosen beam. Check for adequacy, and in this case, OK.
For the fire design of the wood beam:
- the loading is unchanged,
- therefore, the maximum moment is unchanged,
- the fire resistance must be calculated

From NDS Table 16.2.1, find charring depth $\alpha_{\text{char}}$ for 1 hour duration:

<table>
<thead>
<tr>
<th>Required Fire Endurance (hr)</th>
<th>Effective Charring Rate, $\beta_{\text{eff}}$ (in/hr)</th>
<th>Effective Char Layer Thickness, $\alpha_{\text{char}}$ (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-Hour</td>
<td>1.80</td>
<td>1.8</td>
</tr>
<tr>
<td>1½-Hour</td>
<td>1.67</td>
<td>2.5</td>
</tr>
<tr>
<td>2-Hour</td>
<td>1.58</td>
<td>3.2</td>
</tr>
</tbody>
</table>

Now, design the cross-section for fire endurance. A certain amount of the cross-section will char during the duration of the rating time, reducing the cross-section size required to sustain load.

From the table in Chapter 16, find the char depth for the duration you are seeking, in this case, 1 hour.
Fire Design Example (ASD)

Substitute in residual cross-section dimensions for 3-sided beam into the section modulus relation, i.e.:

- 3-Sided Exposure (i.e. beams)  
  \[ b = B - 2\beta t \quad d = D - \beta t \]
  
  \[ = B - 2\alpha_{\text{char}} \quad = D - \alpha_{\text{char}} \]

Calculate charred beam section modulus exposed on 3-sides:

\[ S_f = \frac{(B-2\alpha_{\text{char}})(D-\alpha_{\text{char}})^2}{6} = \frac{(6.75 - 2(1.8))(12-1.8)^2}{6} \]
\[ = 54.6 \text{ in}^3 \]

Determine the charred section dimensions and calculate a new charred section modulus for the residual section.
Fire Design Example (ASD)

Calculate the adjusted allowable bending stress (some adjustment factors don't apply and may have been other than 1.0 before):
\[ F'_{b} = F_{b} \times \text{lesser of } C_{L} \text{ or } C_{V} = 2400 \times 0.99 = 2371 \text{ psi} \]

Calculate strength resisting moment using charred cross-section:
\[ M' = K F'_{b} S_{I} = (2.85)(2371)(54.6) / 12 = 30,758 \text{ ft-lb} \]

Fire Capacity Check:
\[ M' > M_{\text{max}} \]
\[ 30,758 \text{ ft-lb} > 27,945 \text{ ft-lb} \]

Recalculate the adjusted allowable bending stress, since not all of the adjustment factors apply here and may have been a value other than 1.0 before.

Determine the strength resisting moment based on the charred cross-section, and in this case is good for a 1 hour fire duration.
2005 NDS Fire Design

- Full-scale test results indicate that the mechanics-based method conservatively estimates the fire endurance time of all exposed wood members.

- Given the theoretical derivation of the new mechanics-based design method, it is easily incorporated in current wood structural design provisions.

- Incorporation of new mechanics-based method in the NDS assists in the proper design of all exposed wood members for standard fire exposures.

The modeled behavior is conservatively accurate, can be easily implemented as a design process, and permits designers to use exposed large section wood members in structural applications that could be subject to fire exposure.
2005 NDS Appendix E has remained substantially the same. Appendices N is the only new one, and is a mandatory part of the standard necessary to provide the LRFD element to the NDS.
Appendix N *new!*

- Load and Resistance Factor Design
  - source for new variables
  - tabulates $K_F$ conversion factors to convert from ASD reference values (see NDS Supplement) to LRFD reference values
  - tabulates resistance factors $\phi$
  - tabulates time effect factors $\lambda$, for load combinations listed in:
    - ASCE 7-02 – Minimum Design Loads for Buildings and Other Structures
      - NDS clarified for cases involving hydrostatic loads (H) and for cases where H is not in combination with L, use $\lambda = 0.6$

Appendix N, only two pages long, provides the necessary tables for use of the NDS with LRFD. The ASCE 7 -02 and -05 are the reference load documents to be used with the 2005 NDS LRFD process.
2005 NDS Supplement

• Updated to include latest reference values for:
  – visually graded lumber and timber
  – mechanically graded lumber
  – glued laminated timber

The 2005 NDS Supplement contains all of the reference design values for various lumber and engineered wood products, and is part of the standard.
2005 NDS Supplement - $E_{\text{min}}$

- $E_{\text{min}}$ addition for reference MOE for beam and column stability:
  - visually graded lumber and timber
  - mechanically graded lumber
  - glued laminated timber
- Represents 5% lower exclusion shear-free E value so that design value adjustments are not part of the basic design equation for column and beam stability

A new feature of the NDS Supplement that corresponds to a change in NDS provisions is the tabulation of the 5th-percentile E values used in beam stability and column design equations. $E_{\text{min}}$ translates well between the ASD and LRFD processes through the tabulation. Thus, reference design value tables for all lumber and engineered wood products now include the $E_{\text{min}}$ values in their tables – a time saver for the busy designer.
Visually graded dimension lumber (Table 4A)

- Four new species added:
  - Alaska cedar (Alaska & Western states)
  - Alaska Hemlock (Alaska & Western states)
  - Alaska Yellow Cedar (Alaska-grown only)
  - Baldcypress

Reference design value data has now been added for four new wood species of lumber ....
Visually graded timber (Table 4D)

- Two new species added:
  - Alaska cedar (Alaska & Western states)
  - Baldcypress

... as well as two new species of timber.
Non-north American Species (Table 4F)

• Several new species added:
  – Montane pine (South Africa)
  – Norway Spruce (Romania and the Ukraine)
  – Silver fir (Germany, NE France, and Switzerland)
  – Southern pine (Misiones Argentina)
  – Southern pine (Misiones Argentina free of heart center and medium grain density)

The list of non-North American Species continues to grow, adding several new species to the list of tabulated reference design data.
Mechanically graded dimension lumber (Table 4C)

- **New design values added:**
  - Table 4C Footnote 2 – new $G$, $F_v$, $F_{c\perp}$ values for MSR and MEL
  - Table 4C new $E_{\text{min}}$ values for MSR and MEL

New design values have been added for mechanically graded dimension lumber. Specifically, footnote 2 of Table 4C in the *NDS Supplement* provides specific gravity, shear parallel to grain, and compression perpendicular to grain design values for machine stress rated (MSR) and mechanically evaluated lumber (MEL). Table 2 provides an overview of the new design values for MSR and MEL lumber. As with visually graded lumber and timbers, modulus of elasticity for beam and column stability, $E_{\text{min}}$, design values have been added to Table 4C for MSR and MEL lumber.
Several changes have been made to structural glued laminated timber design values in the 2005 NDS Supplement. As with dimension lumber and timber tables, modulus of elasticity for beam and column stability, $E_{\text{mod}}$, design values have been added for glued laminated timber. Species groups for split ring and shear plate connectors were removed from Tables 5A–5D. In some cases, these groups did not correspond to species groups assigned according to NDS Table 12A. A review of the data used to establish connector species groups indicated that values in Table 12A are appropriate. Specific gravity, $G$, of the wood located on the face receiving the connector should be used with NDS Table 12A for assignment of species group. This change is consistent with current recommendations of the American Institute of Timber Construction (AITC) and APA–The Engineered Wood Association.

There were specific changes to Tables 5A, 5A-Expanded, and 5B. Design values for tension parallel to grain, $F_t$, compression parallel to grain, $F_c$, and specific gravity, $G$, are revised for the 16F stress class. The 2001 NDS Supplement showed different values for this stress class in Table 5A vs. 5A-Expanded. Analysis indicated that the values in Table 5A-Expanded were correct, so Table 5A was updated accordingly.

Shear parallel to grain (horizontal shear) design values have increased for prismatic members, and adjustment factors in accordance with Footnote d have been revised. Horizontal shear values in the 2001 NDS Supplement were based on full-scale tests of laminated beams, which were reduced by 10 percent based on judgments made at that time. Shear values for non-prismatic members were those derived according to ASTM D3737 from tests of small shear-block specimens. Since that time, the structural glued laminated timber industry has revised its recommendations and has elected to publish test-based shear values for prismatic members, removing the 10 percent reduction. This change is reflected in the 2005 NDS Supplement consistent with recommendations of AITC and APA. Footnote d adjustment factors were revised to keep shear values for non-prismatic members essentially unchanged.

Historically, radial tension design values for structural glued laminated timber were established as one-third of shear parallel to grain design values. In the 1997 NDS, radial tension values were 67 psi for Southern Pine and 55 psi for Douglas Fir-Larch, respectively. For Douglas Fir-Larch, radial reinforcement designed to carry all induced stresses was required to utilize this value, otherwise the radial tension value was limited to 15 psi—this point was clarified in the 2005 NDS. Comparing 2005 to 1997 NDS Supplements, increased shear values for non-prismatic members of Douglas Fir-Larch and Southern Pine have resulted in small increases for radial tension design values in these species. The slightly increased radial stresses are recommended by AITC and APA and are considered appropriate and preferable to multiple adjustment factors as were used in the 2001 NDS.
Table 5B of the *NDS Supplement* incorporates the following changes:

- Re-formatting of bending design values for bending about the X-X axis, $F_{bx}$. If special tension laminations are included, tabulated values may be adjusted according to applicable footnotes.
- New combinations for Southern Pine were added with extra information regarding slope of grain differences.
- Shear value columns were consolidated for bending about the Y-Y axis, $F_{vy}$, and shear values were updated consistent with Table 5A discussion above.

The most notable change to all design value tables in the *NDS Supplement* is the addition of minimum modulus of elasticity values for beam and column stability, $E_{min}$, design. The change to shear design values for prismatic glued laminated timber members is another significant modification.
Changes from previous editions

- A new format for the future
- New *green* covers

For 2005, the NDS provides a new format for the future that allows two design processes to be used: ASD, and LRFD. And the covers are green!
The 2005 NDS comes complete with two supplements: Design Values, and Special Design Provisions for Wind and Seismic loadings.
The complete tool package adds two more documents: the ASD/LRFD Manual filled with helpful non-mandatory information in the application of the NDS to wood building design, and a Workbook of ASD and LRFD practical design examples to shorten the learning curve.
NDS 2005 Summary

• format changes to accommodate addition of LRFD:
  – Revised terminology
  – Expanded applicability of adjustment factor tables
  – Re-format of radial tension design values
  – Revised format of beam and column stability provisions (addition of $E_{min}$ property)
  – Addition of NDS Appendix N – Load and Resistance Factor Design

• other changes introduced in the 2005 Edition:
  – Removal of form factor
  – Revision of repetitive member factor for I-joists
  – Revision of full-design value terminology
  – Clarification of built-up column provisions

Here is a summary point list of all the changes to the 2005 NDS document from the 2001 version….
NDS 2005 Supplement Summary

• changes in design value tables:
  – $E_{\text{min}}$ values added for all materials
  – $F_v$ values for prismatic glulam increased
  – minor re-formatting

…and another for the 2005 NDS Supplement.
The SDPWS is the design tool for shear wall and diaphragm assemblies and comes complete with a commentary.
The ASD / LRFD Manual completes the package in one handy volume.
Structural Wood Design Using ASD and LRFD (Workbook)

• features:
  – ASD solutions in addition to the 40 examples and solutions in the current LRFD Workbook – updated to the 2005 NDS.

To aid in applying the 2005 NDS to everyday design challenges, the ASD / LRFD Workbook is a helpful tool and resource.
NDS Design Tool Package

2005 NDS and Supplement available now!
– call AWC Publications at 1 800 890 7732

The 2005 NDS – available now!