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6. Connecting Elements+

6.1. Scope

This chapter applies to connecting elements or joints in wood structures that are individually designed and checked to insure safe performance. These include connections using split rings, shear plates, bolts, lag screws, pressed metal plates, nail plates and any joining mechanism for which appropriate test data are available.

This chapter does not apply to connections in wood systems such as floors, roofs, and walls where the connection specification is based on experimental tests of whole system performance.

The reduced net section of the member due to the connection should be examined according to bending, compression or shear safety checking equations covered in Chapters 3, 4, and 5.

Commentary C6.1. Scope. The scope of this draft chapter is limited to those connections where the design is based on individual element reliability rather than on system reliability. This is typical for connections used in heavy timber and nonresidential Construction.

A long term goal of reliability techniques should be to establish practical methods for system reliability of which connection performance would be one component. The objective of this chapter is to demonstrate methods of determining appropriate resistance factors for connecting elements used in wood construction. Where possible, discussion is included on evaluating adjustment factors which should be used on the basic joint capacity.

Three example methods are presented in this chapter for determining resistance factors, Ø, used in a connecting element strength limit states checking equation.

Example 1: Reliability analysis of the strength of bolted connections using a material property distribution (wood embedment strength) and the 'Yield Theory'. This method is appropriate where analytical models shown to adequately predict observed strength, and adequate material property data are available. A verified, general strength model describes values for modified connection geometries and materials with a minimum of additional testing. Reliability inherent in the current ASD design facilitates selecting a target reliability.

+ Chapter Authors: Thomas E. McLain, Associate Professor, Virginia Polytechnic Institute and State University; Marcia Patton-Mallory, Research Engineer, USDA FS Forest Products Laboratory

Example 2: Reliability analysis of bolted connections is based on the experimental proportional limit data from tests of bolted connections. This method is appropriate where adequate test data exists to describe distributions of connection performance. Extension to additional connection geometries and materials is dependent upon additional physical testing. Again, reliability inherent in the current design facilitates selecting a target reliability.

Example 3: Strict conversion of withdrawal strength of lag screws using allowable loads described in the National Design Specification for Wood Construction, NDS (National Forest Products Association, 1986) to determine mean or 5th percentile values. A resistance distribution is fit using these values. and finally a calibrated resistance factor is determined. See C4.2. for a detailed discussion of strict calibration to current practice.

These examples are presented in one format that could be used in a final specification. Other approaches which heavily employ tabulated information could be developed and effectively used for some connection types.

6.2. General

6.2.1 Definition of Terms

The following terms apply to all connections independent of type.

net section
end, edge distance
loaded and unloaded edge
grain angle, fastener angle
fastener axis angle
fastener row
eccentric joints
fastener penetration
main and side member
joint failure, stiffness

6.2.2. Limitations and Restrictions

The following factors are considered when detailing structural connections for wood.

shrinkage effects
shear depth
multiple fastener effects, nonuniform force distribution
manufacturing tolerances on holes and placement
multiple fastener types in same connection
fastener quality specification — size, material, etc.
no defects in connection area
rotation of member with respect to fastener fixity (simple beams)
eccentricity considerations
need to minimize tension perpendicular to grain stresses

Commentary C6.2. General. The terms and limitations shown above are typical (but not all inclusive) of those which would be considered in this section. In addition, a discussion of simple vs moment connections, and the need to consider the partial fixity of connections in structural analysis may be included. Consideration of joint eccentricity and combined loads could also be discussed here. Some of the information in this section is available in handbooks and current design codes (American Institute of Timber Construction 1985, National Forest Products Association 1986).

6.3. Connection Strength

Connecting elements shall be proportioned so that their strength equals or exceeds the required strength determined by a structural analysis for factored loads acting on the structure or, when deemed appropriate a specified proportion of the strength of the connected members, if this value is higher.

In the following sections, the factored connection capacity is determined as:

$$\phi_j P_n \qquad [6.1]$$

where

ϕ_j = resistance factor for type of connection

P_n = nominal connection capacity, adjusted for end use short – term loading

The nominal connection capacity, P_n, given in [6.1] is defined at end use conditions . The following are examples of adjustment factors such as loading condition, joint geometry, and service conditions which may influence joint capacity.

$$P_n = C_g C_1 C_s C_m P_j \qquad [6.2]$$

where

P_j = basic joint capacity for a given fastener type, connection geometry, and loading condition.

C_g = factor to adjust for joint geometry other than that specified for P_j

C_1 = factor to adjust loading conditions for other than that specified for P_j

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1. see footnote on page 44.

C_s = factor to adjust for service conditions other than those specified for P_j

C_m = member species factor

Commentary C6.3. Connection Strength. Design values for mechanically fastened connections in wood structures have been derived by diverse means, depending on the type of fastener. Some connection design strengths are based on a proportion of a test ultimate strength (lag screws) and others are based on a strength at a limiting amount of deformation (nails) or proportional limit (bolts).

Models of connection performance are available for bolted and nailed connections, and truss plate connections (McLain and Thangjitham 1983; Aune and Patton – Mallory1985). Some connections also have a significant recently developed research data base (such as Soltis et al 1986; Soltis and Wilkinson 1987). Reliability analysis should be used as the basis for connection capacities for these connections wherever possible. For other connection types which are not supported by significant new research, it may be expedient to propose a strict conversion of existing design values.

Key to the success of the LRFD code conversion process is establishing a consistent 'failure' criterion for the different connection types. Given the various modes of actions of different connection types under load, a consensus must be reached on what constitutes 'ultimate load'. This task can be accomplished by thoroughly compiling and examining all existing test data for mechanically fastened connections. Additionally. any newly developed modeling technology can be used for specific fastener types.

For each connection type within the scope of this chapter, a basic connection capacity, P_j, should be developed. This basic capacity would be defined on a consistent basis across the broad spectrum of fastener actions. This could be accomplished by defining a 'yield strength' as a design point. The physical representation of this yield point will vary from one joint type to another, but it might be consistently defined as the point beyond which no appreciable increase in load bearing capacity is experienced for a corresponding significant increase in deformation. While this may not always satisfy the pure definition of 'ultimate strength', it is a considerable improvement over the myriad of diverse definitions found in current working stress design specifications.

Basic connection capacity is defined for reference conditions using a single or multiple fastener or connector unit with a specific geometry and wood moisture content less than or equal to 15 percent. Adjustment factors apply to the basic capacity to account for variations from the reference condition. The nominal connection capacity, P_n, is the basic connection capacity at end use which is used in the safety checking equation. A duration of load factor, λ, consistent with that discussed in Chapter 2.

6.4. Connection Stiffness and Deformation

6.4.1. Connection Stiffness

Deformations in structural members and combinations of structural elements, including floors, roofs, partitions and exterior walls due to service loads shall not exceed the limiting value permitted. Estimates of joint stiffness are provided for those situations where joint deformations are necessary for member or component serviceability considerations, or are desirable input into structural analysis. Appendix C examines serviceability considerations and load combinations.

Estimates of nominal joint stiffness (lb/in) are provided for those situations where joint stiffness is needed for design. Nominal joint stiffness is given as:

$$S\ C_{gs}\ C_{ms}\ C_{ss} \qquad [6.3]$$

where

S = nominal joint stiffness for reference conditions (basic connection)

C_{gs} = stiffness factor for connection geometry other than reference

= stiffness factor for member properties other than reference

= Stiffness factor for service conditions other than reference

6.4.2. Connection Deformation Serviceability Criteria

For some applications joint deformation should be limited to insure adequate serviceability. For the serviceability limit state, the joint resistance at a prescribed deformation limit mast be greater than or equal to the specified load. The factored resistance is defined as:

$$\phi_{jd}\ P_{n,s} = \phi_{jd}\ P_{jd}\ C_{gd}\ C_{md}\ C_{sd}\ C_{cr} \qquad [6.4]$$

where

$P_{n,s}$ = connection load at prescribed deformation limit

ϕ_{ja} = deformation resistance factor for connection type

P_{jd} = mean connection load at prescribed deformation limit as determined by standard short term test procedure for reference conditions.

C_{gd} = modification factor for connection geometry other than reference conditions

C_{md} = modification factor for member properties other than reference conditions

C_{sd} = modification factor for service conditions other than reference

C_{cr} = modification factor for creep

Commentary C6.4. Connection Stiffness and Deformation. Stiffness estimates assume a linear joint response to loads below the level of the factored basic joint capacity. Nominal connection stiffness is a mean value rather than a minimal estimate. Estimates of joint stiffness will enable designers to take advantage of structural analysis techniques such as finite analysis that allow greater sophistication in design. As a result, more accurate estimates of deformation under service loads and component stresses will result. With some exceptions, little stiffness information is currently available and will have to be placed on a research agenda. Modification factors to the joint stiffness, for other than reference conditions, are largely unknown except for select situations.

A serviceability limit state may be established for some connection types in applications where joint deformation may be critical. Such applications may include built-up beams, trusses or other subassemblies. It is likely that not all connector types will have a stated serviceability limit state. Most data needed for this section are currently unavailable. Re-examining available data map yield enough information to make reasonable estimates of some connection deformation limit states. Calibrating to current design includes an implied deformation limit state for some connection types.

If analytical models for predicting strength are used for basic connection capacities, then this section should also include guidance for handling situations where deformations are critical. To date, theoretical models have not been developed sufficiently to predict deformations in most types of wood connections found in the U.S.

6.5. Bolted Joints

The basic joint capacity P. for bolted joints shall be determined from the the minimum value obtained from the equations presented in Table 6.1. All basic joint capacities are based on bolts which conform to ASTM A307 (ASTM 1973). minimum steel yield strength of 45000 psi. Provisions are provided in the yield theory for including the effect of higher bolt yield strength.

Basic joint capacities are a function of species and are not dependent upon grade of the wood product. The connection area shall be free from all defects. The basic joint capacity applies for connections where holes are drilled 1/32 inch over the bolt diameter for bolts of 1/2 inch diameter or less, and 1/16 inch over the bolt diameter for bolts larger than 1/2 inch diameter.

Basic joint capacities apply where adequately sized rashers are used, and where the threaded portion of the bolt is not in the bearing plane. Adjustments for threads in the bearing plane are given in equation 6.5.

The following adjustment factors on P_j apply to bolted connections:

$$P_n = P_j \, C_{br} \, C_{sp} \, C_{mb} \qquad [6.5]$$

where

P_n = nominal connection capacity (lbs)

P_j = basic connection yield capacity for a given geometry and loading condition (lbs)

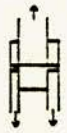
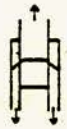
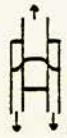
Cbr = factor for threads in bearing

C_{sp} = factor for spacing less than the basic bolt spacing

C_{mb} = factor for multiple bolts in a connection

Table 6.1a Yield Theory Equations Describing Failure Modes for 3-Member Bolted Connections (from Soltis and Wilkinson, 1987)

For any geometry, the minimum failure load obtained from the equations below will govern.

Mode of failure number	Failure geometry	Yield strength F_y
1a		$F_y = DL_2S_2$
1b		$F_y = 2DL_1S_1$
2		$F_y = \frac{2DL_2S_2}{\alpha(2+\beta)}\left[\sqrt{\frac{2(1+\beta)}{\beta} + \frac{2S_y(2+\beta)\alpha^2}{3S_2\left(\frac{L_2}{D}\right)^2}} - 1\right]$
3		$F_y = 2D^2S_2\sqrt{\frac{2S_y}{3S_2(1+\beta)}}$

D = bolt diameter.

S_y = bolt yield stress.

S_1 = embedment yield stress of side members.

S_2 = embedment yield stress of main member.

L_1 = side member thickness.

L_2 = main member thickness.

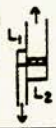
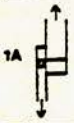
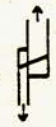
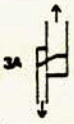
f_c = main member compressive strength.

$\beta = S_2/S_1$.

$\alpha = L_2/L_1$.

Table 6.1b Yield Theory Equations Describing Failure Modes for 2-Member Bolted Connections (from Soltis and Wilkinson, 1987)

For any geometry, the minimum failure load obtained from the equations below will govern.

Mode of failure number	Failure geometry	Yield strength F_y
1		$F_y = DL_2S_2$
1A		$F_y = \frac{DL_2S_2}{\alpha\beta}$
2		$F_y = \frac{DS_2L_2}{\alpha\beta}\,\frac{\sqrt{\beta + 2\beta^2(1+\alpha+\alpha^2) - \alpha^2\beta^3} - \beta(1+\alpha)}{(1+\beta)}$
3		$F_y = \frac{DS_2L_2}{\alpha(2+\beta)}\left[\sqrt{\frac{2(1+\beta)}{3} + \frac{2S_y(2+\beta)\alpha^2}{3S_2\left(\frac{L_2}{D}\right)^2}} - 1\right]$
3A		$F_y = \frac{DS_2L_2}{(1+2\beta)}\left[\sqrt{2(1+\beta) + \frac{2S_y(1+2\beta)}{3S_2\left(\frac{L_2}{D}\right)^2}} - 1\right]$
4		$F_y = D^2\sqrt{\frac{2S_2S_y}{3(1+\beta)}}$

D = bolt diameter.
S_y = bolt yield stress.
S_1 = embedment yield stress of member 1.
S_2 = embedment yield stress of member 2.
L_1 = thickness of member 1.
L_2 = thickness of member 2.

Commentary C6.5. Bolted Joints. This example specification for bolted joints highlights two major problems that have to be addressed in establishing an LRFD format code for wood connections. The first is to agree on a predictable failure criterion and the second is to formulate a rational code presentation.

For dowel – typeconnections in general and bolted joints specifically, there is a well accepted model for connection strength. the European yield theory. The Canadian Code (Canadian Standards Association 1986) and Eurocode 5 (Crubile et al 1985) both use theis yield model for basic connection capacity. Advantages of this method include accurate allowances for steel side members, combinations of species, and direct consideration of the steel yield strength of the bolt. Wood embedment strength is the basic wood property used in the analysis, and is available as a function of compression strength of the wood. Distributions of wood embedding strength are much easier to obtain through tests than are connection proportional limit loads which are the basis for current code provisions. Wood embedding strength values also apply to all connection geometries and failure modes.

The examples below describe the calculation of resistance factors, $Ø_j$, using both the yield theory and proportional limit test data. Both methods use actual distributions of test data to describe the variability in connection resistance. The example connections are three different geometries with Douglas – fir side and main members and a 1 inch diameter bolt loaded parallel to the grain. All examples use 5 minute values for nominal connection or property strength. This analysis results in Ø values represented the top curve in Figure 2.3. Load – durationfactors are discussed in more detail in Chapter 2. For all examples, $L_o = L_n$ was assumed; that is. no reduction was applied to live load for tributary or influence area.

<u>Example 1</u>. This analysis uses bolt embedding data from 78 tests. These values represent the proportional limit load observed during a compression test which uniformly embeds the bolt into a half bolt hole. (See Soltis et al 1986, for details on the test method). These data only apply to Douglas – firloaded parallel to the grain.

A 3-parameter Weibull distribution was fit to the embedding data shown in Table 6.2. Assuming a CV for the bolt diameter and steel yield each to be .05, the resulting connection yield strength distributions have approximately the same CV as wood embedding strength. Hence, most of the connection strength variability is due to variability of wood properties. Connection yield loads shown in Table 6.3 were calculated using yield theory equations given in Table 6.1.

Table 6.2 78 Bolt Embedment Data for Douglas-fir Loaded Parallel to the Grain (psi)

5580	4430	4180	4050	3450	4300
4460	3710	4550	2760	4170	4170
4750	4450	4540	3840	4230	3990
4260	3960	3490	5060	4790	4050
5030	5040	5760	5500	4790	4580
4320	4140	5660	4780	4460	3670
4500	4527	3110	3970	4210	3730
4250	3910	3080	5870	4710	4200
4530	4310	3900	4500	4830	3830
4330	4330	3320	4170	4260	3470
5330	4700	4680	5270	5600	3280
4040	4040	4390	4220	5330	4020
5140	4320	3100	3990	3720	4530

2 parameter Weibull distribution with shape = 7.10, scale = 4632
3 parameter Weibull distribution with shape = 4.137, scale = 4589
location = 1880

Table 6.3 Yield Theory Parameters, Connection Geometry and Predicted Yield Loads for Three Sample Joint Geometries

L_1 (in)	L_2 (in)	D (in)	L_2/D	$S_1=S_2$ (psi)	S_y (psi)	$F_y=\bar{R}$ (lbs)	Failure Mode (Table 6.1)
1.25	2.5	1.0	2.5	4340	45000 A/	10850	1a,1b
2.56	5.13	1.0	5.13	4340	45000	12420	2
4.38	8.75	1.0	8.75	4340	45000	15890	2
1.25	2.5	1.0	2.5	4340	81000 B/	10850	1a,1b
2.56	5.13	1.0	5.13	4340	81000	15660	2
4.38	8.75	1.0	8.75	4340	81000	18230	2

A/ current design assumes minimum steel yield = 45000 psi

B/ test data on actual yield strength of 1 inch bolts

Example 2. Three sets of 15 Douglas-fir bolted connection tests form the basis for this analysis (from Soltis et al 1986). Three parameter Weibull distributions were fit to proportional limit loads for each set (Table 6.4). Coefficients of variation are those corresponding to the fitted Weibull distribution.

Table 6.4 Data for Proportional Limit Load of 3 Sets of Douglas-fir 3-member Joints and Weibul Distribution Parameters

		Set 1	Set 2	Set 3
	L_1	1.25	2.56	4.38
	L_2	2.5	5.13	8.75
	D^2	1.0	1.0	1.0
		6080	8600	14600
		6320	8000	7600
		6640	8800	8800
		5600	7000	6000
		5840	6600	9200
		4800	6600	14200
		6560	8400	8700
		6400	8400	6100
		6320	9000	12000
		6160	7000	9700
		6480	8400	7600
		6160	8000	9600
		6640	7600	8600
		6000	6000	8800
		6000	7200	8000
Parameters of	shape	9.728	4.985	2.312
fitted Weibull	scale	6304	8026	10014
distribution	location	3000	4000	4000
	CV	.06	.11	.26

Table 6.5 describes details of the two reliability analysis calibrated Lo nominal connection strength, R_n. R_n is defined as 1.9 times the published (National Design Specification for Wood, 1986) load for the connection geometries. Significant differences exist for reliability indices calculated for different connection geometries. [Note: these examples are for fixed levels of $ß_o$ determined for only three geometries. [n practice $ß_o$ would be selected after examination of a wider variety of geometries.] The resistance factors using yield theory and $ß_o$ = 3.8 vary from .78 to 1.24, and using proportional limit and $ß_o$ = 3 vary from .78 to 1.06. As expected, yield theory values are more variable when calibrated to current design since the basis for the connection strengths are not the same. However, inadequate test data exists to formulate proportional limit distributions for all species and connection geometries.

Table 6.5 Example Reliability Analyses of Bolted Connections with combination of live and dead loads.

Yield Theory (78 Embedding Tests, S_y = 45000 psi)

Joint (see Table 6.3) L_2/D	$\bar{R}$ (lbs)	CV[A/]	R_n[B/] (lbs)	$\bar{R}/R_n$	R_o/R_n	β	$\phi(\beta_o=3.8)$
2.5	10850	.17	5990	1.82	.5	3.7	.78
5.13	12420	.17	5070	2.45	.5	4.3	1.00
8.75	15890	.17	5080	3.13	.5	>4.5	1.24

Proportional Limit

Joint (see Table 6.3) L_2/D	$\bar{R}$ (lbs)	CV	R_n[B/] (lbs)	$\bar{R}/R_n$	R_o/R_n	β	ϕ $(\beta_o=3)$
2.5	6140	.06	5990	1.03	.5	2.8	.78
5.13	7700	.11	5070	1.52	.5	3.8	1.06
8.75	9300	.26	5080	1.83	.5	3.1	.84

A/ Assumes CV of connection yield load is due only to variation in wood embedment values

B/ R_n = published allowable load times 1.9

The curves in Figures 6.1 and 6.2 show present reliability (no-load-duration) compared to the target reliability, dotted horizontal line.

For these examples, the resulting safety checking equation is:

$$\lambda \phi_j P_n = \lambda (.78) P_n \geq 1.2 D_n + 1.6 L_n \qquad [6.6]$$

where

P_n = nominal connection capacity adjusted for end use

O_j = resistance factor

λ = load-duration factor(dependent on load case)

For this example λ is assumed equal to unity.

Notice that in Table 6.3 that when bolt yield strength is increased from 45000 psi to 81000 psi only the connections which fail due to bolt bending (modes 2,3,4) show an increased strength due to increased bolt capacity. Furthermore, steel side members do not significantly increase the strength of connections which fail predominantly in bolt bending modes (modes 3 and 4) (see Figure 6.3 and Soltis and Wilkinson 1987). Figure

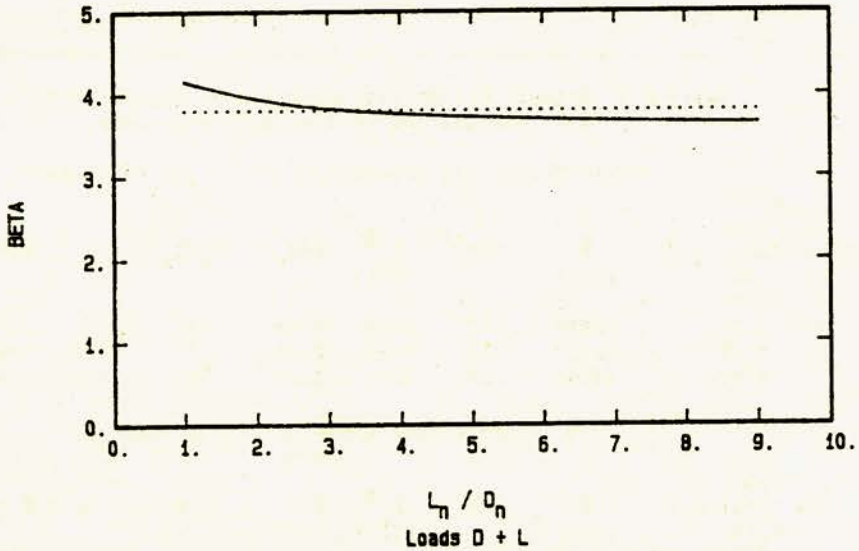


Figure 6.1 Bolted Connection Reliability - Yield Theory

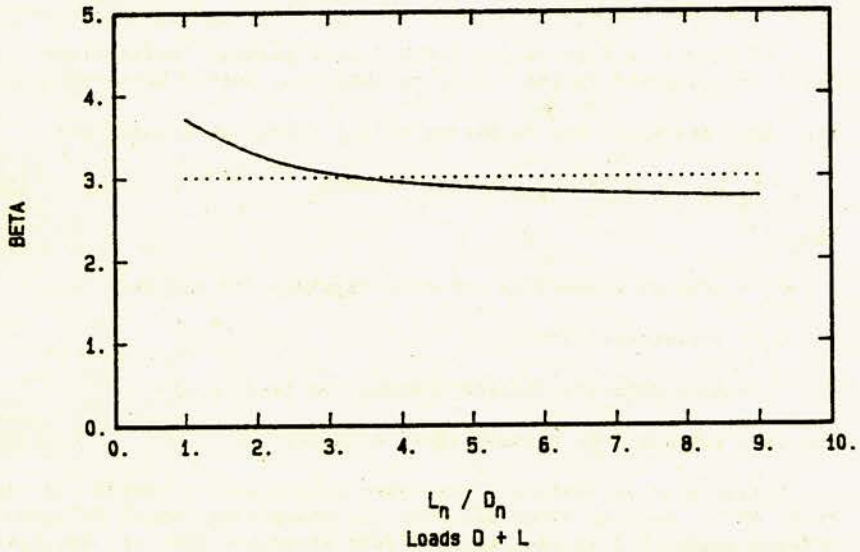


Figure 6.2 Bolted Connection Reliability - Proportional Limit

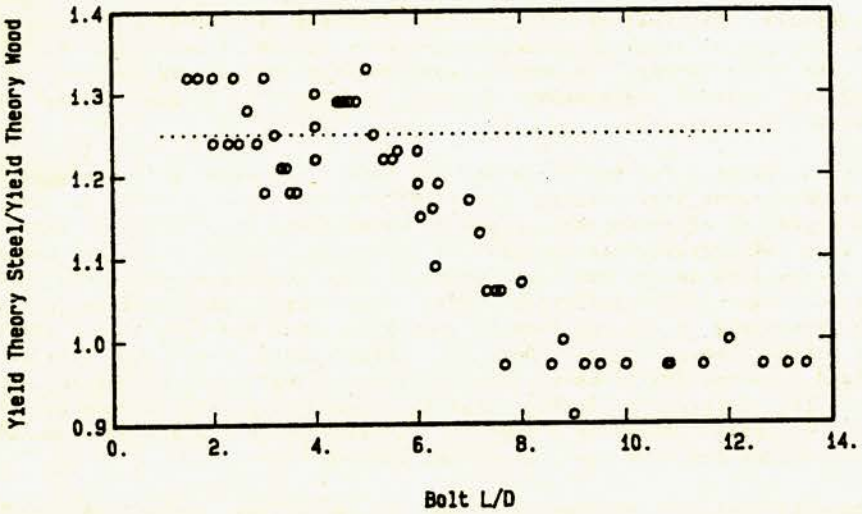


Figure 6.3 Adjustment Factor for Steel Side Members Currently in the NDS does not necessarily apply to Connection Yield Strengths

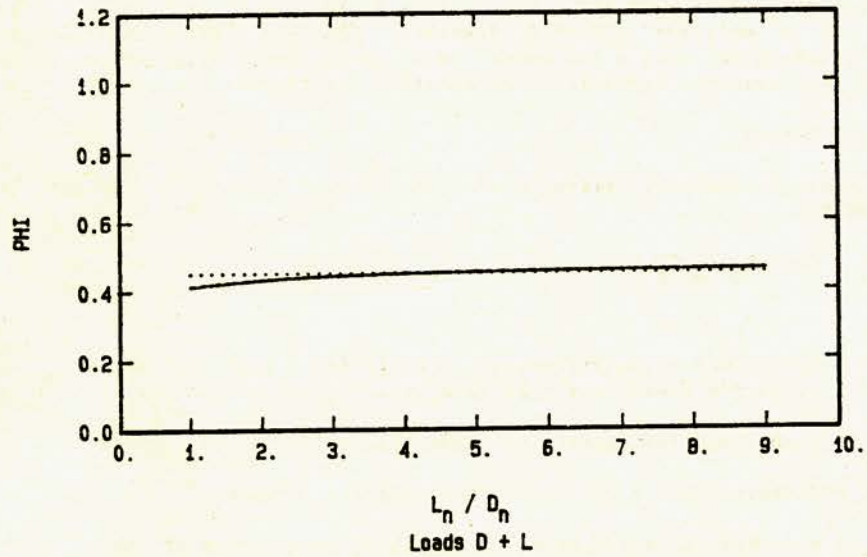


Figure 6.4 Lag Screw Reliability - Strict Conversion

6.3 compares the current 25 percent increase for steel side members (dotted horizontal line) to predicted increase due to steel side plates using the yield theory. Therefore, considerable care should be exercised in adopting current adjustment factors blindly to a new theory of connection strength.

Once a suitable failure criterion is chosen, the problem becomes one of rational code presentation. Eurocode 5 (Crubile et al 1985) presents design values in equation form which is convenient for computer users. This type of presentation explicitely recognizes different failure modes but can also lead to calculation errors. The Canadian code CSA 086 (Canadian Standards Association 1986) discretizes presentation of the design provisions in tabular form in such a way that the user is not aware of its yield theory origin. Both of these codes have the distinct advantage over the NDS of being inherently much simplier to understand and apply. The particular format that is chosen for presenting connection design criteria requires careful consideration and substantial deliberation by industry, government and code authorities.

The equations or methodology used to establish basic connection strength P_j should be provided in the LRFD format for the designer who must interpolate between or extrapolate beyond tabular values. It would be very desirable to minimize the number of species groups used in the table. One way to approach this is to use a member properties factor C_m. Methodology for deriving basic connection strength for wood composites and other wood based materials not currently in the code should be included in the commentary. A basic joint geometry for two member and three member joints can be defined as wood side members 1/2 the thickness of the main member with minimum spacing as defined in a table. The basic joint geometry for joints with steel side plates would have minimum steel plate equal to a multiple of bolt diameter. CSA 086 (Canadian Standards Association, 1986) uses a two member joint as the basic joint which lends itself to convenient application of modification factors.

6.6. Lag Screws

The basic withdrawal strength of lag screws from wood members is defined by:

$$P_j = 6160\ C_m\ C_d\ C_p \qquad [6.7]$$

where

P_j = basic connection Withdrawal capacity for a specified set of reference conditions (lbs/in of penetration of thread)

C_m = member species factor for other than reference conditions

C_d = diameter factor for other than reference diameter

C_p = penetration factor equal to length of penetration of the threaded portion of the lag screw

Commentary C6.6. Lag Screws. The origin of equation [6.7] is the Wood Handbook (USDA Forest Products Laboratory. 1972):

$$P_w = 8100\ (SG)^{3/2}\ D^{3/4} \qquad [C6.1]$$

where

SG = specific gravity

D = lag screw diameter. in.

P_w = average withdrawal strength per inch of penetration of the threaded part, lb/in.

Equation [C4.4] describes compution of the 5th percentile of a lognormal distribution given the mean and CV. P_j is defined as the 5th percentile of the withdrawal strength, and is the nominal connection strength used in Example 3 subsequently. The factor to adjust the average to a 5th percentile is included in [6.7].

Example 3. A strict conversion (direct calibration) is performed for withdrawal strength of a 1 inch diameter lag screw from Douglas – fir (Table 6.6). The nominal strength, R_n, of the fastener is defined as the lower 5th percentile of the 5 minute strength . The predicting equation [C6.1] from the Wood Handbook predicts mean withdrawal strength, R. Therefore, the 5th percentile was calculate assuming a lognormal distribution, equation [C4.4]. The ratio of the 5th percentile to the published value is 3.4. A CV of .20 is used based on inspection of test data (Newlin and Gahagan 1938).

Table 6.6 Example of a Strict Conversion to 4 for Withdrawal Strength of Lag Screws and 1.2 D_n + 1.6 L_n

Published Allowable Load = 599 lb/in (DNS Table 8.64)
Specific Gravity = SG = .48 (Douglas – fir South)
Bolt Diameter = D = 1.0 in.

$$\bar{R} = 8100\ (SG)^{3/2}\ (D)^{3/4} = 2700\ \text{lb/in}$$

$R_n = R_{05}$ of lognormal distribution with CV = .20

= .761 $\bar{R}$ (see Equation [C4.4])
= 2048 lb/in

R_{05} / published allowable = 3.4

4 (strict conversion, $L_n/D_n = 4$) = .45

The calibrated Ø of .45 is reasonable when compared to the factor of .22 used to calculate current design allowables from the Wood Handbook average withdrawal strength. However, the current factor includes a load duration adjustment. Figure 6.4 shows a calibrated Ø versus load ratio with R_{05}/(published value) = 3.4.

6.7. Other Adjustment Factors

The nominal connection capacity, P_n, is defined to be at end use conditions. Sections 6.3 through 6.6 give examples of adjustment factors, 'C_{xx}', which should be used to adjust basic connection capacities for situations other that what is specified for the basic connection. Additional factors are outlined below.

6.7.1. Geometry Factors

Additional factors could exist for member dimensions. edge distance, end distance, multiple shear planes, multiple fasteners, spacing of fastener rows, grain orientation, and fastener orientation, depth of penetration, bearing area, oversized holes, and combined lateral/ withdrawal load.

6.7.2. Member Property Factors

Basic connection capacities should have adjustment factors for species or specific gravity, chemical treatments or exposure, exposure to repeated wetting and high moisture content, fastener strength properties, and steel member effects.

Commentary C6.7. Other Adjustment Factors. For some fasteners, the conditions for the factors for grain orientation, bearing area, multiple shear planes or depth of penetration can be included in the basic connection capacity. An example is the yield theory for bolted connections. This decision must be made when selecting a failure criterion and code format.

Consideration should be given for the way in which the adjustment factors are applied. The product of all factors assumes independence of each factor. A reasonable probability of all factors affecting connection strength at the same time should be considered.

The adjustment factors should result in design effects similar to those when current adjustment factors are used. However, if a strength limit state is not influenced in the same manner as a serviceability limit state, new factors may need to be developed for the strength limit state. Considerable analysis of the basis for current adjustment factors is needed before they are adopted for the LRFD code. Significant test data on most of these adjustment factors do not exist.

6.8. Bearing

The bearing capacity of a connected member shall exceed the factored load effects. Basic bearing capacity is defined as:

$$P_n = P_j A_b C_m C_{gr} \qquad [6.8]$$

where

P_n = nominal connection capacity in bearing adjusted for end use (lbs)

P_j = basic joint capacity for a reference species and bearing configuration

A_b = bearing area (in^2)

C_m = member species adjustment factor

C_{gr} = grain orientation adjustment factor

Commentary C6.8. Bearing. Many engineered wood components rely on bearing connections either parallel to or perpendicular to grain. This section will define the procedure for determining basic capacities.

6.9. Joist Hanger Connections

The capacity of a joist hanger connection shall exceed the factored load effects.

This section to be developed by joint engineer-manufacturer task group and is left uncompleted by intent.

Commentary C6.9. Joist Hanger Connections. One objective of the LRFD code is to provide a relatively uniform reliability for similar member types. Joist hangers play I significant role in engineered wood construction. Therefore, a section on analysis and safety checking and detailing of joist hangers should be included here in the connection chapter. Basic connection capacities would be provided by the manufacturer. This section would define resistance factors, adjustments to the basic connection capacity and means of obtaining nominal resistances.

6.10. Truss Plate Connections

The capacity of a truss plate connection shall exceed the factored load effects.

This section to be developed by joint engineer-manufacturer task group and is left uncompleted by intent.

Commentary C6.10. Truss Plate Connections. A section on analysis and safety checking of truss plate connections should be included. Nominal resistances will be provided by the manufacturer and this section will define resistance factors and adjustments to the nominal resistance. This analysis applies only to the consideration of a truss plate connection as a single element. Systems reliability analysis which considers the truss plate connection in the context of the whole truss is outside the scope of this document.

Analytical models exist to describe the deformation and failure of truss plate connections using data from standard plate and member test configurations. Appropriate formatting of the analysis for ease of use in safety checking is critical.

6.11. Shear Plate and Split Ring Connections

The capacity of a shear plate or split ring connection shall exceed the factored load effects.

This section to be developed by joint engineer-manufacturer task group and is left uncompleted by intent.

Commentary C6.11. Shear Plate and Split Ring Connections. Considerable detail exists in the current code for design of these type of connections. Results of recent test data should be incorporated into the adjustment factors where appropriate. Basic connection strengths calibrated from the current allowables should be included here, as well as resistance factors and adjustments to the basic connection strength. A model exists for the load capacity of a row of shear plates (Erki and Huggins 1983).

6.12. References

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