Chapter 2 – Literature Review

2.1 – General

As with all engineered products, knowledge gained from research pertaining to laterally-loaded dowel-type connections forms the basis of their design. Thus, the results, significance, and influence these studies have had on connection design methods over the past eight decades are covered in this chapter. An overview of the current connection design method in the U.S. is also presented. Finally, the present status of reliability-based design for wood construction, especially as it pertains to laterally-loaded dowel-type connections, is discussed.

2.2 – Connection Research

Due to the fact that their stiffnesses and yield strengths are relatively low as compared to that of other structural components in the system, connections play a major role in the load-response characteristics of a light-frame building. Indeed, dowel-type connections manifest a positive influence on light-frame structural performance in two significant ways: 1) added ductility, and 2) increased ability to dissipate mechanical energy input (greater damping). Despite the importance of connections in wood structures, however, research on them was not begun in earnest until the mid-1920's. This is mainly due to the fact that an abundance of resources meant wood structures could historically be built much stronger than necessary. Thus, there was no pressing need for more efficient design techniques of wood structures; nor was there an impetus for generation of knowledge about their mechanical behavior. Over the last few decades, research on laterally-loaded connections has dramatically increased the knowledge of how these connections behave under both monotonic and cyclic loading. This knowledge has been effectively used to justify better design methods, more realistic reduction or adjustment factors, and more appropriate building code provisions. Because of the fact that specified design methods are so dependent upon understanding brought about by research, procedures based upon conclusions of said research are discussed simultaneously throughout this section.

2.2.1 – Historical Progression of Wood Connection Design

Prior to the 1920's, there was no unified specification giving details on how to safely design a laterally-loaded dowel-type connection in wood. Entities like the American Railway Engineering Association (1905) often provided specifications for *materials* to be used in such connections (in this case, for those found in timber trestles), but these usually did not give guidance on their actual design. A lack of knowledge on how these connections behaved under load was largely to blame for this.

In 1925, the first experimental results on laterally-loaded connections in wood were published by H.S. Grenoble. This study was funded by the U.S Navy, Bureau of Aeronautics in an effort to gain a better understanding of the mechanics of these connections and develop design techniques accordingly. It involved the testing of both single and double-shear connections with main and side members of either White Ash or Sitka Spruce (as these were the predominant species used in aircraft framing at the time) and focused on the bearing strength of bolts in wood.

Although the study was funded for the purpose of developing more efficient aircraft design techniques, its implications were more far reaching than the mere data and analysis it provided. It helped spark the notion amongst the structural engineering profession that it might be possible to more precisely design laterally-loaded connections in timber structures and also provided insight on possible methods of testing. Thus, since the mid 1920's, numerous studies have been conducted to build upon, verify, and in some cases, reevaluate data antecedently developed. Most of the research that has been conducted on wood-to-wood connections has been done with a broad range of structural uses in mind, not just aircrafts.

2.2.2 – Development of Empirically-Based Design

In 1932, G. W. Trayer, a research engineer at the Forest Products Laboratory in Madison, WI published the results of a study that applied to connections using only "common, commercial steel bolts…not [to] be construed as applying to [connections with] steel aircraft bolts." Trayer's 1932 publication was significant because until then, safe working values for design of bolted connections in timber construction had not yet been developed.

Trayer tested several hundred double-shear bolted lateral connections, including many different specimen and loading configurations. Species used for the main and side members were softwoods such as Douglas fir, sitka spruce and yellow pine, and hardwoods such as oak and maple. Various sizes of bolts were used. Two loading configurations were tested: loading parallel to, and loading perpendicular to the grain of the wood in the main member. Each main member was sized in its dimension parallel to the axis of the bolt such that the ratio of main member thickness to bolt diameter (L/D) was equal to either 2, 4, 6, 8, 10, or 12. Specimens were constructed to fit in these L/D groups because previous research done on aircraft connections suggested that all specimens with a fixed L/D ratio, regardless of main member thickness or bolt diameter, had similar proportional limit stresses (Trayer 1928).

Early research conducted at the Forest Products Laboratory focused primarily on the average stresses developed at the wood-bolt interface. Data collected from bolt bearing tests was expressed in the form of curves representing normalized proportional limit stresses with respect to the L/D ratio (Figure 2.1) (Trayer 1932). The curves were normalized with respect to the average ultimate crushing strength both parallel and perpendicular to the grain of clear (free of strength-reducing defects) specimens. It was

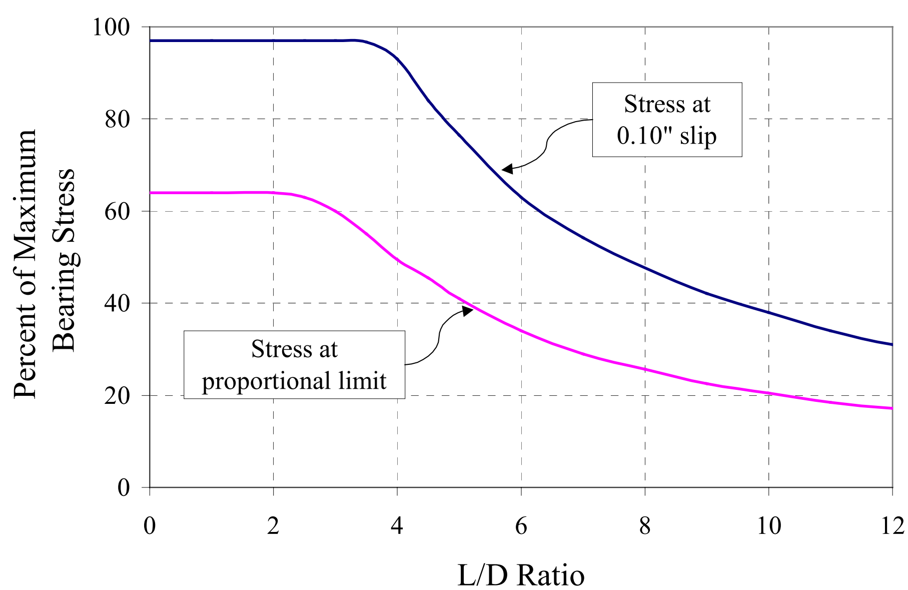


Figure 2.1: Relationship between average bolt bearing stress and L/D ratio for conifers (after Trayer 1932).

found that specimens with small L/D values exhibited little or no decrease in proportional limit stress. In other words, they could be taken as having a proportional limit stress of equal to, or slightly less than, 100% of the member's ultimate crushing strength. Conversely, specimens with larger L/D values were found to have increasingly lowered proportional limit stresses as L/D increases. Connections loaded parallel to the grain having an L/D value of 12, for example, would be expected to have proportional limit stresses of between 34% and 43% of the member's ultimate crushing strength. Further, it was found that the normalized proportional limit stress with respect to L/D in members made of species having high ultimate crushing strengths were predicted by curves that dropped off more steeply than those with lower ultimate crushing strengths. Using this information, percentages of ultimate crushing strengths were tabulated over varying L/D ratios and varying ultimate crushing strengths. Thus, through interpolation of variables like ultimate crushing strength (for which values were tabulated according to species) and L/D ratio, designers were able to calculate "safe" design values, or working loads, for bolted connections using the equation:

$$P = n(\sigma_b \rho \gamma)Dt \tag{2.1}$$

Where:

P = allowable or "safe" connection design load,

n = number of fasteners in the connection,

D = diameter of the bolt,

t = thickness of the main member, and

$\sigma_b\rho\gamma$ = allowable average stress (either parallel or perpendicular to the grain) under the bolt, such that σ_b is the tabulated basic crushing strength, ρ is the bolt diameter factor, and γ is a tabulated factor for reducing the basic crushing strength, based upon the L/D ratio.

Note, that only the thickness of the main member (and not that of the side member) is considered in this equation. This is because it was assumed that all connections would have side members with thicknesses, t_s equal to half that of the main member, t_m such that:

$$t_s = \frac{1}{2} t_m \tag{2.2}$$

One of the major drawbacks inherent to the aforementioned design method was the fact that it was based solely upon empirical data from a finite number of connection configurations. Subsequent reports from the Forest Products Laboratory helped broaden the design method's usefulness by offering suggestions on how to properly handle issues such as end distance, edge distance, and bolt spacing so that full design values would be safely attainable without the risk of premature cracking (Scholten 1939). For softwoods, prescribed minimum end distances were four times the bolt diameter for joints in compression, and seven times the bolt diameter for joints in tension. Minimum edge distance and bolt spacing were set at 1.5 and four times the bolt diameter, respectively. These geometric provisions have not changed and are still required in the current design method for full design value. A simplistic method of design for single-shear connections in which the allowable load would be taken as half that of a comparable symmetrical double-shear connection was prescribed as well. Clarification was provided on how to handle connections loaded at various angles to the grain using the Hankinson Formula and accompanying nomograph (Newlin 1939). While these reports aided in the design process, the empirical nature of the equations rendered them applicable only to the connection configurations that had been studied. Calculations of allowable loads for untested configurations, species, or load types using this design method were merely extrapolations. Nevertheless, connections were designed using this technique for decades and have generally demonstrated adequate performance within wood structures.

2.2.3 – Factors Affecting Bearing Resistance

Due to the importance of embedment resistance in determining the strength of a connection, many studies have focused on isolating and quantifying the effects of variables influencing it. In addition to the L/D ratio, factors including bearing surface roughness, angle between loading and grain direction, loading rate and/or duration, bolt angle with respect to load direction, bolt hole oversize, specific gravity, and moisture content have all been investigated. Many of these studies have resulted in changes in specified design methods. Additionally, intrinsic variability of each of these factors, in turn, affects the variability of resistance provided by connections, and thus, that of the structure as a whole.

Goodell and Phillips (1944) examined the effect of bolt hole or pilot hole roughness. It was found that bolt holes drilled at low rotational speeds and high feed rates had visibly rough inner wall surfaces. These rough surfaces, in turn, were found to afford as low as one-third the resistance at proportional limit afforded by smooth bearing surfaces. Based upon these observations, it was concluded that calculated design loads were applicable only in cases where bolt holes were drilled properly and suggestions were made for appropriate drilling methods.

Testing has shown that bearing resistance for loading perpendicular to grain is generally equal to or lower than that for loading parallel to grain in softwoods. For cases where a dowel-type fastener bears on wood in a direction other than parallel or perpendicular to the grain, the bearing resistance provided assumes some intermediate value. Two formulas in particular, the Hankinson Formula and the Osgood Formula (Osgood 1928) have been used to estimate this value for bearing resistance under bolts. Kojis (1953) examined and compared the accuracy of these equations for a number of grain angles in Southern Yellow Pine. It was found that while neither of them was perfectly suited, the Hankinson Formula, due to its more conservative output, was safer. Thus, it was considered preferable because of the high variability inherent to bearing resistance. Consequently, the Hankinson Formula has been prescribed for use in design of angled bolted connections in the U.S. ever since the publication of the first connection design method. While some studies have indicated a slight difference exists between perpendicular and parallel-to-grain bearing resistances under nails (Mack 1960; Foschi 1974), the general consensus is that this difference is not significant. Soltis et. al. (1987) showed this to be the case for bearing strengths under most nail diameters with the exception of those specimens having exceptionally low specific gravity. In this study, it was found that the dowel diameter at which parallel and perpendicular-to-grain ultimate bearing strengths diverge ranges from 0.15” to 0.40” for all practical specific gravities. For this reason, the angle between loading direction and grain is not accounted for in nailed connection design.

Since wood is a viscoelastic material, it exhibits time-dependent mechanical properties. Numerous studies have demonstrated this for the case of embedment resistance, stiffness, and creep under fasteners (Girhammar and Andersson 1988; Koponen 1992; Rosowsky and Reinhold 1999). Generally speaking, bearing resistances, and therefore connection resistances, increase with respect to monotonic loading rate or dynamic loading frequency. This time-dependent bearing strength relationship serves as the rationale behind the NDS® *load duration factor*, C_D (AF&PA 1997a). Nominal design loads for connections are multiplied by this factor, which currently ranges from 0.9 for dead loads to 1.6 for loads lasting ten minutes, to obtain an allowable design load. Such considerations have gained increased attention over the past few years, reflecting a growing interest in connection response to seismic or other dynamic loads. Gutshall and Dolan (1994), for example, investigated the load duration factor of 1.6 for fully-reversed cyclic loading (similar to that brought about by a seismic event) and found it to be appropriate.

Wilkinson (1993) conducted a study to evaluate the influence of both bolt hole oversize and angle of bolt axis with respect to loading direction on connection and embedment strength. While no significant difference was brought about by bolt oversize alone, interaction did bring about reduced resistances in connections where the bolt was at a small (2 degree) angle to the plane of the side member. This is most likely due to reduced initial bearing area at the wood-bolt interface and also explains why initial stiffness was greatly reduced. Despite the fact that bolt hole oversize alone was found not to affect connection resistance, the NDS® currently stipulates that bolt hole diameters shall be no more than $^{1}/_{16}$" and no less than $^{1}/_{32}$" greater than that of the bolt itself. The reason for these limits, however, is not based upon bearing strength. The upper limit is designed to prevent excessive slack in joints and also to minimize the possibility of bolt misalignment in multiple bolt connections. The lower limit was set in an effort to control the initiation of splitting in the wood members brought about by forcible driving of the bolt (AF&PA 1997b) and/or shrinkage of the wood around the bolt. This lower limit also helps make joints easy to assemble.

As with any wood product, light-frame wood and timber construction is subject to strength, stiffness, and dimensional changes with respect to moisture content. Despite its importance to overall structural performance, few studies have focused solely on the degree to which bearing resistance under dowel-type fasteners, and thus resistance of the connections themselves, are affected by moisture content. Some of the more recent research on this topic was conducted at the Forest Products Laboratory (Rammer and Winistorfer 2001). Among other findings, this study concluded that both bearing stiffness and strength under nails were negatively correlated to moisture content. The NDS® accounts for strength reduction and negative effects of dimensional change (discussed in Section 2.2.3) brought about by moisture content and fluctuations thereof with the simplified *wet service factor*, C_M. For bolted and nailed connections, this factor assumes a value of either 0.4, 0.7, or 1.0 depending upon moisture conditions at various stages in the life of the structure.

Relationships between bearing strength and specific gravity at various moisture contents were also investigated by Rammer and Winistorfer (2001). Linear regressions, fitted to data from this same study, closely matched those fitted to data from previous research conducted by Fahlbusch (1949) for bearing resistance under bolts. While a strong positive correlation between bearing resistance and specific gravity was evident, the slope of this relationship was shown to be dependent upon moisture content. It was concluded that the linear regression fit to data representing specimens at 12% moisture content closely followed the relationship determined by Wilkinson (1991b) (Rammer 2001). Because moisture contents of around 12% are typical in light-frame construction, this expression is used in determining embedment strengths for calculation of nominal connection design values per the NDS®. For the bearing strength under bolts, this expression is:

$$F_{e\parallel} = 11200G \tag{2.3}$$

$$F_{e\perp} = \frac{6100G^{1.45}}{\sqrt{D}} \tag{2.4}$$

and for the bearing strength under nails, it is:

$$F_e = 11600G^{1.84} \tag{2.5}$$

Where:

Fe = embedment strength (psi), and the subscripts $_{||}$ and $_{\perp}$ represent loading directions parallel and perpendicular to the grain,

G = specific gravity, and

D = nominal dowel diameter.

Note that the equation for embedment strength under nails does not differentiate between parallel and perpendicular bearing-to-grain angles. Due to conclusions made from research discussed previously, such a differentiation was deemed unnecessary. It has been shown that for extremely dense woods (esp. *Brosimum alscastrum* and *Vatairea lundellii*), these expressions are inaccurate (Rammer 1999). This, however, is not an issue with typical species groups used in light-frame and timber construction.

The preceding factors comprise most of the major variables influencing bearing stiffness and resistance afforded at proportional limit, yield, and/or capacity. Other variables include temperature and various forms of treatment. The manner in which each of these factors has been accounted for in historic and contemporary design was discussed. Additionally, the variability of each of these influences contributes to variability in the load-response characteristics of wood structures in general and, as is discussed in Section 2.4, must therefore be accounted for in the determination of resistance factors used in LRFD.

2.2.4 – Factors Affecting Fastener Loading Within a Structural System

The performance of a structure under a given loading input is largely dependent upon the relative distribution or concentration of stresses within its connections. Subsequently, the loading that a particular fastener within this system will be subjected to is influenced at least partially by interaction between itself and other components of the system. Phenomena such as composite action (load sharing) and the nonuniform load distributions developed among fasteners in a row are prime examples of this. Additionally, dimensional changes induced by fluctuations in moisture content can affect both the loading a connection will experience and its behavior under such loading.

The individual elements of certain subassemblies typical in light-frame construction often exhibit load sharing behavior (Bonnicksen and Suddarth 1965). This behavior, also known as *composite action*, has been investigated in a reliability sense for subassemblies such as floor diaphragms. The American Plywood Association (Tissell 1967) was among the first to study this phenomenon by testing diaphragms with plywood sheathing. Numerous studies conducted since then have confirmed that composite action significantly increases the efficiency by which loads are transferred through a light-frame structure into the foundation. This increase in load-carrying efficiency is, in turn, due to the more even (and thus, less variable) loading distribution experienced by fasteners between the sheathing and framing members (AF&PA 1997b). NDS® accounts for said increase in efficiency with the *diaphragm factor*, $C_{di} = 1.1$, which is applicable only to nailed connections in a diaphragm.

As was evident in Equation (2.1), the allowable load of a connection was originally based upon a linear relationship between the allowable load for one fastener, and the number of fasteners in that connection. Tests conducted in the 1960's, however, indicated that this assumption was incorrect (Doyle and Scholten 1963). For a connection having multiple fasteners in rows aligned along the direction of loading, the fasteners on the ends of the rows will experience greater loading than those in between. This phenomenon is recognized in current design and accounted for by a reduction in the allowable connection load. Said reduction is accomplished by the *group action factor*, C_g. More recently, appropriate values for this factor have been quantified for cyclic loading (Anderson 2001).

The effects of dimensional changes brought about by fluctuations in moisture content have also been investigated. While these dimensional changes do not affect the actual bearing strength or stiffness, they may induce internal loading or stresses due to the high degree of redundancy (and therefore static indeterminacy) typical of wood structures. Additionally, gaps which can develop between the members of a connection as a result of shrinkage have been shown to have a negative impact on the damping ability of a connection (Chou and Polensek 1987). This is especially pertinent in cases where lumber or timbers are green (high moisture content) at the time of fabrication and allowed to dry in service.

2.2.5 – Connection and Structural Response Models

Numerous models, both theoretical and empirical, have been developed to predict the behavior of laterally-loaded nailed or bolted connections. Some of these models only predict the resistance provided by a connection at a certain point along the load-slip function. Others predict the resistance provided at any displacement within a certain range along the load-slip function. The former of these makes for the derivation of simpler equations and thus often lend themselves more readily to design. The later may be useful in the analysis of structural stiffness or elastic response to loads.

Johansen (1949), in his "Theory of Timber Connections," derived mathematical models for various connection yielding modes. The work presented in this publication, which was also expanded upon by Moller (1950), represented a major advance in the understanding of laterally-loaded dowel-type connections because resistance values calculated using the models described therein are not subject to the same limitations as empirically derived values. The *European Yield Model*, or simply the y*ield model* , as it is called, is based upon the assumption that both the steel in the dowel and the wood in the connection members exhibit a rigid perfectly-plastic stress-strain relationship. In other words, it is assumed that no strain (or deformation) occurs in either the dowel or the wood at the bearing interface until the yield stress of each respective material is reached. Beyond this yielding point, it is assumed that deformations increase indefinitely with no additional increase in stress (i.e., perfectly plastic response). An idealized rigid-perfectly plastic stress-strain plot is shown as compared to a typical embedment or fastener bending plot in Figure 2.2. This assumption greatly simplifies the modeling of stress distributions along the length of the dowel, thereby allowing for the derivation of equilibrium equations. These equations may, in turn, be used to solve for the theoretical connection resistance at yield. Note, however, that since bearing resistance is assumed constant with respect to displacement, corresponding connection slip cannot be predicted using this model.

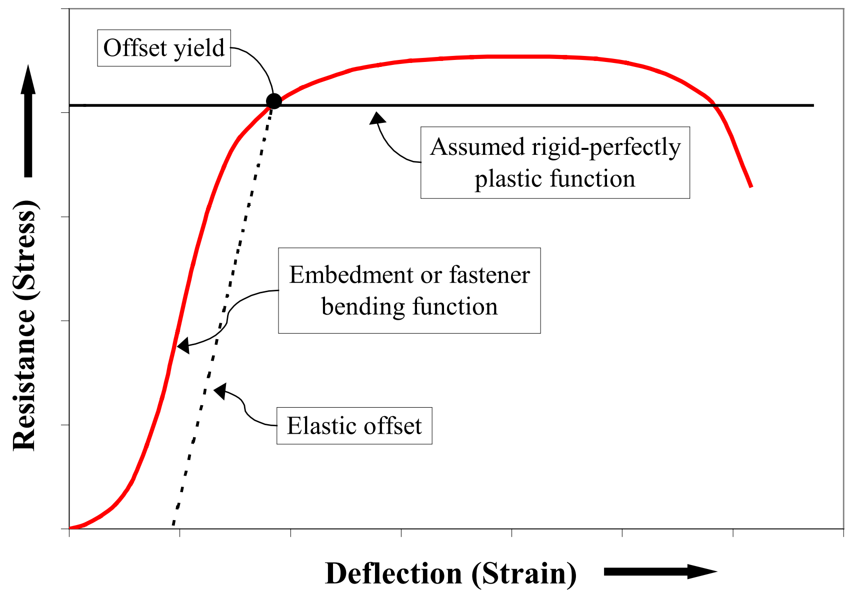


Figure 2.2: Typical embedment or fastener bending function versus idealized rigid-perfectly plastic function.

While the assumptions made in the yield model are by no means purported to be the actual stress-strain relationship exhibited by these materials, equations derived from the model have been experimentally proven to accurately predict the resistance provided by a connection at yield (Aune and Patton-Mallory 1986; McLain and Thangjitham 1983). Due to the advantages of the yield model over empirical design methods, it was adopted as the basis of U.S. connection design in 1991. Thus, a more detailed explanation of the *general dowel equations* derived from the yield model is warranted and is therefore provided in Section 2.3.

Kuenzi (1955) developed a model based upon the assumption that the dowel within a connection under lateral loading behaves as a beam on an elastic foundation. This report was significant in that it was the first attempt at modeling the load-slip characteristics of a nailed or bolted connection. Additionally, this model allowed for the rational analysis of both single-(two-member) and double-(three-member) shear connections, which was not possible using the empirical design method. Because the model assumes that the foundation on which it rests (the wood) has a *linear* stiffness function, k, the model predicts load-slip characteristics only up to the connection's proportional limit. Another limitation of this model is the fact that the stiffness of the bearing under the bolt was originally assumed to be equal to the compressive modulus of elasticity of the wood (k = E).

In one of the first non-linear load-slip models, Foschi (1974) assumed a nonelastic foundation exhibiting an embedment response function described by the following:

$$p = (p_o + p_1\omega)\left(1 - e^{\frac{-k\omega}{p_o}}\right) \tag{2.6}$$

Where p is the stress provided at the bearing at a given embedment, ω, k is the initial modulus, and the parameters p_o and p_1 are determined through least squares fitting of test data from embedment samples. This model was extended by Dolan (1989) to include hysteretic characteristics of connection behavior under cyclic and dynamic loading. It has proven to be quite useful in software designed to simulate the response of structures under such loading conditions (Foschi et al 2000).

Foliente (1993) derived a general hysteresis model for simple (single-degree of freedom) wood structures. This work has helped to advance studies in dynamic and cyclic loading response characteristics not just of connections, but of wood structures as a whole, as it may also be extrapolated, and applied to multiple-degree of freedom structures. Because of the fact that this model predicts hysteretic response of structures to *random vibration* inputs, it also may be applied to *dynamic reliability analysis* of wood structures.

Heine and Dolan (2001) recently constructed a load-slip model based upon a combination of assumptions made in the yield theory (i.e., formation of perfectly plastic hinges at finite points along the dowel) and the embedment function in Equation 2.6. Under these assumptions, an algorithm was developed to perform incremental calculations along the length of the dowel to attain a relationship between connection resistance and slip. This model has been shown to accurately predict said relationship in a single fastener monotonic connection configuration. Heine (2001) also expanded upon this by integrating other models (i.e., parameter estimation model, stochastic model, hysteresis model, and failure model) such that resistance, as well as intermediate parameters (i.e., material and hysteretic properties) were estimable for multiple bolt connections at given connection slips. This integrated model, implemented into a computer program termed "MULTBOLT," was validated with experimental data on numerous configurations of multiple-bolt connections.

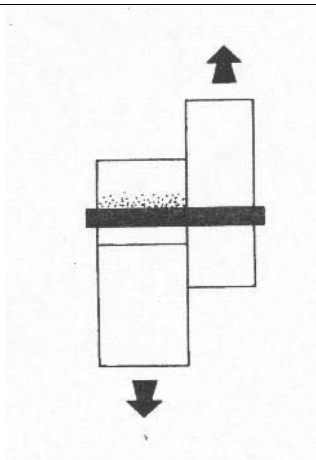
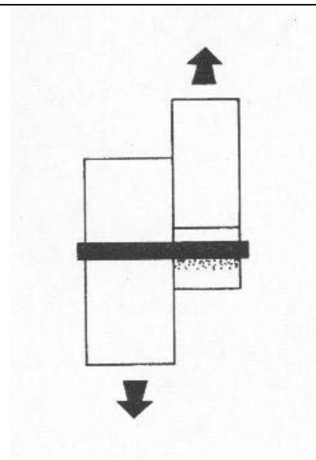
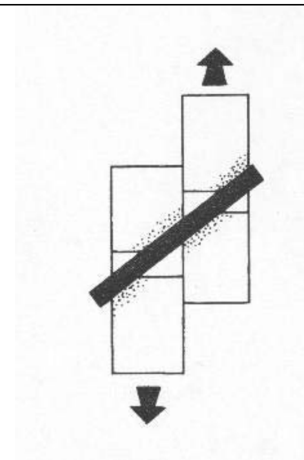
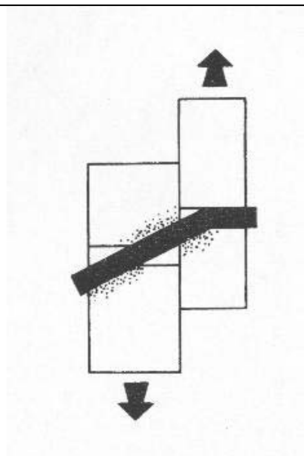
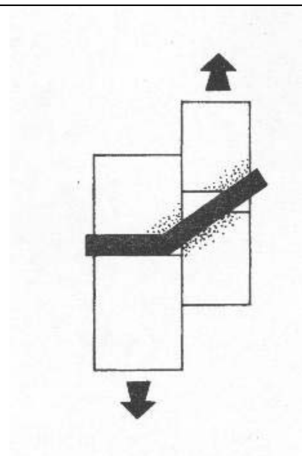
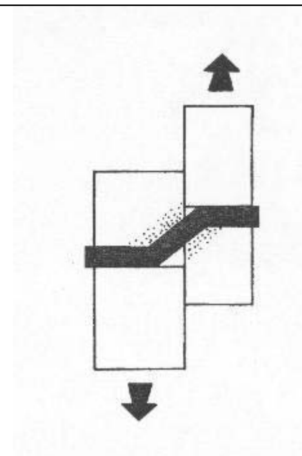
The aforementioned research comprises some of work that has been completed in the ongoing effort to model the mechanics involved in laterally-loaded connections. The complexity of this problem due to the unpredictable embedment response behavior of wood has made an exact load-slip model difficult to arrive at, although relatively accurate approximate solutions have been formulated. Through the comparison and fine-tuning of these models with experimentally-generated test data, their accuracy may be continually improved. In addition, new techniques such as the use of x-ray imagery to observe fastener deformation at various stages of loading (Humphrey and Ostman 1988) may be used to further this effort.

2.3 – Current Connection Design in the U.S.

Wood and timber structures may currently be designed using either *allowable stress design* (ASD) or *load and resistance factor design* (LRFD). The standard corresponding to ASD is the *National Design Specification for Wood Construction* (NDS®) (AF&PA 1997a). This manual has been mentioned several times thus far throughout the chapter and is the basis of discussion in Section 2.3.1, as it is currently the most widely used. The manual corresponding to LRFD is appropriately titled the *Load and Resistance Factor Design Manual for Engineered Wood Construction* (AF&PA 1996). In this chapter, it will simply be referred to as the LRFD Manual. Procedures for connection design using the LRFD Manual are discussed in Section 2.3.2.

Since 1991, the specified design method for laterally-loaded connections in the U.S. has been based upon the yield theory. This theory, as previously alluded to, was developed in Europe. Some modifications have been made since its original formulation in order to make the model more appropriate for incorporation into U.S. design specifications (McLain and Thangjitham 1983).

Table 2.1: Diagrams of yield modes assumed in yield theory for single-shear connections (from AF&PA 1997a).

Mode I_m	Mode I_s	Mode II
		
Mode III_m	Mode III_s	Mode IV
		

It is generally assumed in the Yield Theory that there are three modes of yield for a double-shear (three-member) connection and four modes of yield for a single-shear (two-member) connection. As is illustrated in Table 2.1, double-shear connections may exhibit one of *two* forms in Mode I. Similarly, single-shear connections may exhibit one of *two* forms in Mode I as well as in Mode III. Hence, there are effectively four yield modes in double-shear connections and six yield modes in single-shear connections. It must also be noted that restrictions are necessarily placed on the number of possible yield modes (a total of four) for *nailed* single-shear connections, as yield Modes I_m and II are not observed in practical applications of this configuration. This is because the combination of a minimum allowable main member penetration depth of 6D and the fact that main member material (solid wood) is of relatively high embedment strength usually precludes the possibility of rigid displacement (Mode I_m) or rigid rotation (Mode II) of the nail within the main member.

In single-shear, Mode I yield is exhibited when the wood in *one* of the members is crushed but the dowel remains free of plastic deformation. This is likely to occur if one of the members affords a relatively low bearing resistance at yield (as compared to that of the other member) and the dowel is of sufficiently high stiffness. Yield Mode II is the mode by which wood at the bearing in *both* members is crushed and the dowel, still remaining free of plastic deformation, simply rotates about an axis perpendicular to both the axis of the dowel and the direction of the load vector. This is likely to occur if both members provide about the same bearing resistance at yield and the dowel has a relatively high stiffness. Mode III yield is characterized as that in which crushing occurs predominantly in *one* of the members due to dowel rotation while the dowel undergoes plastic deformation within the other member. This mode is often exhibited in cases where one of the members has a relatively high bearing resistance, the other one has a relatively low bearing resistance, and the dowel has a relatively low stiffness. Finally, yield Mode IV is defined as that in which the dowel forms an "S" shape upon undergoing plastic deformation in both members, thereby causing limited amounts of crushing in each. Such a yield mode can occur if the ultimate bearing strength in both members is about the same and the stiffness of the dowel is relatively low. Note that bearing lengths within both the main and side members also have a significant effect on the magnitude of moment the dowel will be subjected to, and therefore play a role in determining the yield mode. The descriptions above assume approximately equal bearing lengths in both the main and side members.

For each of the aforementioned yield modes, equations have been derived as an expression of the resistance in a lateral connection at yield. These *general dowel equations*, as they are called, are based upon the assumption that all materials in the connection exhibit rigid-perfectly plastic stress-strain functions as described in Section 2.2.3. They may be expressed as shown in Table 2.2 (AF&PA 1999).

Table 2.2: General dowel equations of the yield model modified for single-shear connections with no gap between main member and side member.

Mode I_m	Mode I_s	Modes II - IV
$P = q_m l_m$	$P = q_s l_s$	$P = \dfrac{-B + \sqrt{B^2 - 4AC}}{2A}$

Inputs A, B, and C for Yield Modes II - IV			
Mode II	$A = \dfrac{1}{4q_s} + \dfrac{1}{4q_m}$	$B = \dfrac{l_s}{2} + \dfrac{l_m}{2}^{*}$	$C = -\dfrac{q_s l_s^2}{4} - \dfrac{q_m l_m^2}{4}$
Mode III_m	$A = \dfrac{1}{2q_s} + \dfrac{1}{4q_m}$	$B = \dfrac{l_m}{2}^{*}$	$C = -M_p - \dfrac{q_m l_m^2}{4}$
Mode III_s	$A = \dfrac{1}{4q_s} + \dfrac{1}{2q_m}$	$B = \dfrac{l_s}{2}^{*}$	$C = -\dfrac{q_s l_s^2}{4} - M_p$
Mode IV	$A = \dfrac{1}{2q_s} + \dfrac{1}{2q_m}$	$B = 0^{*}$	$C = -2M_p$

Where:

P = nominal lateral connection value, lbs

l_s = side member dowel bearing length, in

l_m = main member dowel bearing length, in

$q_s = F_{es}D$ = side member dowel bearing resistance, $^{lb}/_{in}$

$q_m = F_{em}D$ = main member dowel bearing resistance, $^{lb}/_{in}$

$M_p = F_b{*}D^3/6$ = dowel plastic moment, in-lbs

F_{es} = side member bearing strength, psi

F_{em} = main member bearing strength, psi

F_b = dowel bending strength, psi

D = dowel diameter, in

*Gap between members, g = 0

2.3.1 – Allowable Stress Design for Wood Connections

The general inequality used in the *allowable stress design* of any system component which is subjected to some combination of n stresses or *forcing functions*, $Q_1 \ldots Q_n$, brought about by a combination of *working loads* or *design loads* is (Boresi et al 1993):

$$\sum_{i=1}^{n} Q_i \le \frac{R}{\rho} \tag{2.7}$$

where R is the expected resistance afforded by the component at a given point in its input-response relationship (i.e., proportional limit, yield, or capacity) and ρ is the chosen factor of safety. Note that this equation does not involve the modification of any Q_i to account for variability which might be inherent in each.

The inequality in Equation (2.7) illustrates the basic methodology behind connection design as prescribed in the NDS®. Variations of the general dowel equations presented in Table 2.2 are given in the NDS® for calculation of nominal lateral design values, Z such that:

$$Z = \frac{P_{min}}{nK} \tag{2.8}$$

where P_{min}, which replaces the R term in Equation 2.7, is the value associated with the applicable yield mode providing the least lateral resistance, as calculated by the general dowel equations. The variable n is an adjustment factor which, in *bolted* connections, represents the approximate ratio of lateral resistance calculated using the yield model to values calculated by the empirically-based design method. Based upon research conducted by Wilkinson (1991a), adjustment factors were determined for each yield mode. These values are given in Table 2.3.

Table 2.3: NDS[90] values of n for single-shear bolted and nailed connections.

Yield Mode	Bolted Connections	Nailed Connections
I_m	4.0	1.0
I_s	4.0	1.0
II	3.6	1.0
III_m	3.2	1.0
III_s	3.2	1.0
IV	3.2	1.0

The term K in Equation 2.8 is a factor which, in bolted connections, K_θ, accounts for the angle, θ, between loading direction and grain direction. This is defined as (AF&PA 1997a):

$$K_\theta = 1 + \frac{1}{4}\left(\frac{\theta}{90^o}\right) \tag{2.9}$$

Thus, K_θ is a value between 1.00 and 1.25. In *nailed* connections, this K value is included as an approximate method by which to reduce lateral resistance values at yield to the lateral resistance values corresponding to the connection's proportional limit. For nailed connections, K_D is defined as (AF&PA 1997a):

$$K_D = \begin{vmatrix} 2.2 ....D \le 0.17" \\ 0.5 + 10D.......0.17" < D < 0.25" \\ 3.0.................D \ge 0.25" \end{vmatrix} \tag{2.10}$$

where D is the nail diameter. The values of 2.2, 3.0, and the intermediate linear interpolation were chosen such that lateral resistances calculated using the yield theory would be approximately equilibrated to nominal design values specified prior to 1991. They are based on research conducted by McLain et al (1990) in which comparisons were made between empirically-based and theoretically-based values. Note that the value n for all *nailed* connections is simply 1.0. This is because the ratio it represents in the design equation for bolted connections is (for that of nailed connections) accounted for in K_D.

According to the NDS®, the allowable lateral design load, Z', on a *bolted* connection is:

$$Z' = Z(C_D)(C_M)(C_t)(C_g)(C_\Delta) \tag{2.11}$$

and the allowable lateral design load on a *nailed* connection is:

$$Z' = Z(C_D)(C_M)(C_t)(C_d)(C_{eg})(C_{di})(C_{tn}) \tag{2.12}$$

where C_D, C_M, C_g, C_Δ, and C_{di} are adjustment factors that were discussed in Sections 2.2.2 and 2.2.3. The other adjustment factors, C_t , C_d, C_{eg}, and C_{tn} account for reductions in strength due to temperature, reduced nail penetration within the main member, end grain, and toe-nailing effects, respectively (AF&PA 1997a).

2.3.2 – Example Calculation of a Design Value Using ASD

As an example, consider a single shear nailed connection between a ¾" piece of plywood and a 2x4 Spruce-Pine-Fir wall stud. The connection involves a 10d common nail. In this example, the controlling design load is a wind load which has a duration of ten minutes or less, and thus C_D = 1.6. Assume no other adjustment factors apply (although the diaphragm factor, C_{di} = 1.1 would apply to the connection if it were in a shearwall or floor diaphragm). Recall from Table 2.2 that the three geometrical variables used in the calculation of theoretical connection resistance (using the general dowel equations of the yield theory) are: 1) side member bearing length, l_s, 2) main member bearing length, l_m, and 3) dowel (nail) diameter, D. These can be determined using the information given thus far, as each of these components has standard dimensions. The side member bearing length is simply taken as the thickness of the side member;

$$l_s = 0.75'' \tag{2.13}$$

The main member bearing length is calculated by subtracting the side member thickness from the nail length (an 8d common nail is 2.5" long);

$$l_m = 3.0'' - 0.75'' = 2.25'' \tag{2.14}$$

The nail diameter is a given since the nail type is specified;

$$D = 0.148'' \tag{2.15}$$

Recall also that the three basic material properties that are used as input for the general dowel equations are: 1) side member bearing strength, F_{es}, at 5% offset yield 2) main member bearing strength, F_{em}, at 5% offset yield and 3) dowel bending yield strength, F_{yb}, also at 5% offset yield. Main and side member bearing strengths can be determined either through approximation by using the relationship with specific gravity given in Equation 2.5, or through embedment tests which are conducted on specimens cut from the same pieces of material as the main and side members. While the latter is more desirable because it provides the actual bearing strengths of the materials, the former is almost always used in design because of its simplicity. One of the drawbacks of Equation 2.5, however, is that it is only intended to be used for solid wood members. Thus, the bearing strength of the plywood must be determined empirically. For this example, let's assume bearing strengths of 8400 psi for the side member (plywood) and 4100 psi for the main member (SPF 2x4), so that the bearing resistances, q_s and q_m, are:

$$q_s = F_{es}D = 8400\,psi(0.148'') = 1243\,{}^{lb}\!/_{in} \tag{2.16}$$

$$q_m = F_{em}D = 4100\,psi(0.148'') = 607\,{}^{lb}\!/_{in} \tag{2.17}$$

Also assume that the bending yield strength of the nail is 100,000 psi (Note: this reflects a typical experimental value observed in this research. The NDS published value for the bending yield strength of a 10d common nail is 90,000 psi.) so that the plastic moment, M_p, is:

$$M_p = \frac{F_b D^3}{6} = \frac{100{,}000\,psi(0.148'')^3}{6} = 54.0\,in \cdot lb \tag{2.18}$$

Due to the mechanics of a single shear nailed connection, only four of the six yield modes are possible. These are Modes I_s, III_m, III_s, and IV. The lateral 5% offset yield resistance, P_{Is}, associated with Yield Mode I_s is:

$$P_{I_s} = q_s l_s = 1243\,{}^{lb}\!/_{in}(0.75'') = 932\,lb \tag{2.19}$$

Results of the calculations for inputs A, B, and C of the equations for Yield Modes III_m, III_s, and IV, as defined in Table 2.2, are presented in Table 2.4.

Table 2.4: Example inputs A, B, and C of the equations for Yield Modes IIIm, IIIs, and IV.

	Input A	Input B	Input C
Mode III_m	0.000814	1.125	-822
Mode III_s	0.001025	0.375	-229
Mode IV	0.001226	0	-108

With these input parameters, values for lateral 5% offset yield resistance associated with Yield Modes III_m, III_s, and IV, P_{IIIm}, P_{IIIs}, and P_{IV}, respectively, are calculated using the quadratic equation given in Table 2.2 as follows:

$$P_{III_m} = \frac{-1.125+\sqrt{1.125^2-4(0.000814)(-822)}}{2(0.000814)} = 529lb \tag{2.20}$$

$$P_{III_s} = \frac{-0.375+\sqrt{0.375^2-4(0.001025)(-229)}}{2(0.001025)} = 324lb \tag{2.21}$$

$$P_{IV} = \frac{\sqrt{-4(0.001226)(-108)}}{2(0.001226)} = 297lb \tag{2.22}$$

The least of these values, P_{min}, is that which corresponds with Yield Mode IV, P_{IV}, with a calculated (or theoretical) 5% offset yield resistance of 297 lb. The *nominal lateral design value*, Z, using allowable stress design, is then calculated using Equation 2.8 as:

$$Z = \frac{P_{\min}}{nK_D} = \frac{297lb}{1.0(2.2)} = 135lb \tag{2.23}$$

Where n is 1.0 as shown in Table 2.3 and K_D is equal to 2.2, as defined in Equation 2.10. Finally, the allowable lateral design load is calculated as:

$$Z' = 135lb(1.6) = 216lb \tag{2.24}$$

Thus, 216 lb is the allowable design load for the described connection in the given application.

2.3.3 – Reliability-Based Design for Connections

The general inequality used in the *reliability-based design* of any system component which is subjected to some combination of n service loads, $Q_1 \ldots Q_n$, is (Boresi et al 1993):

$$\sum_{i=1}^{n} \gamma_i Q_i \leq \phi R \tag{2.25}$$

where, as in Equation (2.7), R is the expected resistance afforded by the component at a given point in its input-response relationship. The load factor γ_i is applied to the i^{th} service load in order to account for inherent variability in said function's mean or expected value with respect to time over the course of the component's service life. Load factors are typically greater than one and increase with respect to the variability of their respective forcing function. The resistance factor, ϕ, is applied to the component's assumed resistance. This factor accounts for variability in material properties and is typically less than one. It must be noted that in most applications, the term R is taken as the maximum resistance said component may provide (i.e., its capacity) since any increase in the forcing function function's amplitude beyond this point would initiate failure within the component. Hence, in such a case, the limit state is loss of functionality.

Procedures currently exist for reliability-based design of laterally-loaded connections in wood construction. These are prescribed in the LRFD Manual which was mentioned in Section 2.3. Like the NDS®, the LRFD Manual uses variations of the general dowel equations shown in Table 2.2 as a basis for calculation of connection resistance. The inequality used in this design procedure is as follows (AF&PA 1996):

$$\lambda \phi_z Z' \geq Z_u \tag{2.26}$$

Where:

λ = the time effect factor which is based upon the applicable load combination,

ϕ_z = 0.65, the resistance factor for connections,

Z' = adjusted lateral resistance provided by the connection, and

Z_u = factored lateral load applied to the connection.

The procedure represented by this inequality is based upon the methodology of Equation (2.25) with the addition of the time effect factor, λ. Said factor is necessary to account for the rheological changes that wood may undergo, as discussed in Section 2.2.2. The adjusted lateral resistance, Z', for bolted connections is calculated as:

$$Z' = Z(C_M)(C_t)(C_g)(C_\Delta) \tag{2.27}$$

and for nailed connections, this is calculated as:

$$Z' = Z(C_M)(C_t)(C_d)(C_{eg})(C_{di})(C_{tn}) \tag{2.28}$$

Note that Equations (2.27) and (2.28) are nearly identical to Equations (2.11) and (2.12), respectively, with the omission of the load duration factor, which is accounted for in the λ term of Equation (2.26). A difference *does* lie in the calculation of the *reference lateral resistance*, Z, which, based upon the equations in Table 7.5-2(a) of the AF&PA/ASCE Standard 16-95, may be calculated as:

$$Z = \frac{3.3P_{min}}{nK} \tag{2.29}$$

for both bolted and nailed connections. The variables P_{min}, n, and K in Equation (2.29) are the same as those in Equation (2.8). Thus, the only difference between these two equations is the factor of 3.3, which, in conjunction with the resistance factor, ϕ_z = 0.65, simply represents a "soft conversion" from ASD to LRFD.

2.4 – Status of Reliability-Based Design for Wood Construction

A basic overview of current wood design practice using both ASD and LRFD was provided in Sections 2.3, 2.3.1 and 2.3.3. It was stated that the LRFD Manual for Engineered Wood Construction, by definition, is a reliability-based design specification. In addition to the U.S., design specifications based upon reliability concepts currently exist in Canada, Europe, and Australia. None of these provisions, however, is fully reliability-based as of yet. Reasons for this are discussed in the present section.

Serious consideration of LRFD as a design procedure for wood construction has been made only during the past two decades. Three major factors which have contributed to the delayed development of reliability-based design for wood structures, as compared to that of steel or concrete structures are:

- rheological characteristics rendered occupancy load models which had been formulated for steel and concrete structures useless in the case of wood structures (Foschi et al 1989),
- complexities such as moisture content effects, high variability, and anisotropy have made the calibration of appropriate performance factors relatively difficult, and
- the ability to calibrate performance factors for connections has been (and still is) thwarted by a serious lack of data on their ultimate capacities.

Much of the research conducted in this field up until now has concentrated on the individual structural elements within light-frame or timber structures (Foschi et al 1989). As a result, reliability-based load and performance factors have been calibrated for the design of most members found within wood structures (i.e., joists, girders, columns, etc.). Research has not, however, been conducted to fully investigate the same parameters for design of the *connections* within wood structures. This imbalance in the progression of reliability-based design for wood structures as a whole is due to the fact that the mechanics of connections are significantly more difficult to characterize than those of simple members (Smith and Foliente 2002). Thus, so-called soft conversion factors, as mentioned in Section 2.3.3, are still used in connection design. In order to accurately quantify performance factors for connections, and thereby move away from dependency on these conversion factors, test data must be generated on their true capacities. In light of the important role connections play, therefore, it may be stated that reliability-based design for wood structures in general will not be a reality until its connections may be designed in a similar manner.

2.5 – Summary

Due to their prevalence and the extremely important role laterally-loaded connections play within light-frame wood and timber structures, their proper design is crucial. Since the 1920's, many studies have been conducted in an effort to develop the knowledge which would allow this on a safe and rational basis. These studies have shed light on many of the complexities inherent to the behavior of laterally-loaded dowel-type connections. Additionally, numerous models have been developed to predict the resistance this type of connection will provide at various points in its load-response relationship. The yield model, in particular, has been modified and adopted almost universally, as the equations derived from it have been shown to accurately predict connection resistance at yield. In the U.S., the yield model is used in both ASD and LRFD as a basis for calculating lateral connection resistance. Despite all the knowledge that has been gained on the behavior and mechanics of these connections, however, data on their capacities is lacking. This precludes their design on a reliability basis, and thus, reliability-based design for wood structures as a whole is also not possible at present.