Chapter 2 – Literature Review

2.1 Background

Due to recent natural disasters such as Hurricane Andrew in 1992, the Loma Prieta earthquake in 1989, the Northridge earthquake in 1994, and the rash of hurricanes in 2004, there has been an increased interest in the behavior of structures under wind and seismic loading conditions. Generally, more catastrophic failures occur during dynamic and reverse cyclic loading associated with natural disasters such as earthquakes and hurricanes than during gravity or monotonic loading. Monotonic loading has a constant loading of either tension or compression but does not alternate between the two. A reverse cyclic loading pattern travels through zero alternating between tension and compression. To dissipate the energy associated with cyclic loading, structures are designed to be ductile and must be able to undergo large displacements without significant loss of strength.

Wood members, under monotonic loading parallel to grain, typically have a linear-elastic behavior until reaching the proportional limit then behavior is non-linear until failure. Failure can be brittle and sudden. Up to 90% of a wood structure's ability to dampen the effects of vibration or oscillatory movement is due to connections (Chui and Smith, 1989) (Yeh, et al., 1971). Connections play an important role in providing the necessary ductility and energy dissipation in wood structures. Therefore, the design of wood structures should recognize and account for the dynamic behavior of connections.

2.2 Yield Limit Model

The National Design Specification (NDS) for Wood Construction dictates the structural design for wood construction in the United States and is published by the American Forest and Paper Association (AF&PA, 2001). In 1991, AF&PA adopted the Yield Limit Model to predict the behavior of connections (AF&PA, 1991). A significant shortcoming of this model is that it only addresses yield behavior and does not consider failure or group action of multiple bolts in a connection. A failure mode is the behavior of the connection when it has reached its capacity and can no longer sustain the applied load. An example would be splitting of the wood member.

The force versus displacement behavior of wood is often idealized by a curve as seen in Figure 2.1. The initial portion of the curve is some times nearly linear and when loading causes such behavior it is called linear-elastic. Linear-elastic behavior does not cause permanent deformation in the wood so when the load is removed, a return to zero deformation occurs. The proportional limit corresponds to the point where the load begins to cause non-linear behavior in the wood and the wood material begins to yield. Loading that produces non-linear behavior causes permanent deformation in the connection so that when the load is removed, zero deformation can not be obtained.

The behavior of the wood during yielding is used to develop the yield limit model which predicts the behavior and strength of a connection. The model does not consider how the wood ultimately fails, but rather uses the yield strength of the wood material. To determine the yield strength of wood, load is applied to a fastener which is bearing against the wood, thus, creating a force versus deformation curve. A line parallel to the initial portion of the force-deformation curve but offset at a distance equal to 5% of the fastener diameter is formed. The load where the generated line crosses the force-deformation curve is assigned as the yield strength of the wood material as illustrated in Figure 2.1. Further information regarding the determination of the yield strength of wood material can be found in ASTM D 5764-97a(2002), *Standard Test Method for Evaluating Dowel Bearing Strength of Wood and Wood-Based Products* (ASTM, 2003d) and Section 3.6.1 of Chapter 3.

Not all wood subjected to loading exhibits the initial linear-elastic behavior, but instead behaves non-linearly throughout the entire loading process. Yielding of the wood materials is then occurring throughout the loading process and a return to zero deformation can not occur. The yield strength of the wood in these cases is determined similarly as just described.

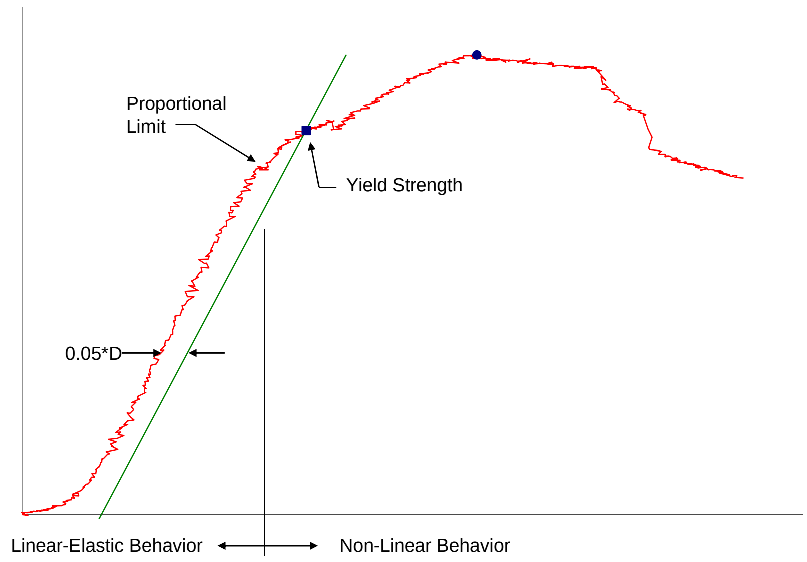

Figure 2.1: Idealized Force (lbs.) - Deformation (in.) Curve

The Yield Limit Model relates a connection's design load to the maximum stresses in the wood members and fasteners through equations for each yield mode. The predicted connection yield strength for a given yield mode is based on the bearing yield strength of the wood and the bending yield strength of the dowel. Bearing yield strength of wood is based on the performance of the wood due to the interaction with the dowel fastener. Bending yield strength of the dowel is a property of the dowel material. The design load for the connection is the lowest calculated yield strength for the various modes. The four yield modes for single-shear connections are illustrated in Figure 2.2 and equations 2.1 through 2.6 are the equations used to calculate which yield mode will control design.

The Yield Limit Model was originally based on the European Yield Model (EYM). The EYM was developed from the work of Johansen (1949) and verified with available experimental results from Trayer (1932), Soltis et al. (1986) and Wilkinson (1978). Several modifications have been made to the EYM to form the Yield Limit Model found today in NDS. In their 1991 publication, Soltis and Wilkinson discuss some of the adaptations from the EYM which are present in the US Yield Limit Model. They note that the US definition of the yield point differs and that prior to the adoption of the EYM, the dowel bearing strength property was not used in the US. A relationship between dowel bearing strength and specific gravity was determined from test data and presented in their publication. Also, the authors note that dowel bearing strength can be determined from test procedures found in a standard published by American Society of Testing and Materials (ASTM) (Wilkinson, 1991). EYM predictions were compared to more than 1,000 bolted connection tests and were found to be adequate (Soltis & Wilkinson, 1991).

The modifications to the EYM proposed by McLain and Thangjitham (1983) aided the model's ability to account for the influence of nuts and washer at bolt ends on joint bearing capacity. To account for sliding friction between timbers in a connection, Larsen (1973) introduced an additional modification. In 1993, Wilkinson published results from his studies on the effect of bolt hole size and angle of drilling on the EYM-predicted loads. He found that increased bolt size had little effect on the yield load or maximum load, but generally increased the deformation. Oversized holes decreased the load up to 21 percent when the hole was oversized 1/16-in. The EYM predicted the yield load adequately (Wilkinson, 1993). The Yield Limit Model and the US modifications assume that the end and edge distances of the bolts are sufficient to prevent failure due to shear or splitting (Gattesco, 1998).

Four primary yield modes are possible for single-shear bolted connections. A single-shear connection is defined as a connection where only two members are present with a fastener between them thus resulting in only one shear plane where the fastener passes between the two members. The bearing of a dowel fastener on wood fibers is represented by Yield Mode I. The yielding can occur in either the main member or the side member. The main member is defined as the member that has the largest thickness dimension as compared to the second member. The side member has a smaller material thickness dimension than the main member. If the connection has members of the same thickness, than assignment of main and side is arbitrary. The rotation or pivoting of a dowel fastener at the single shear plane without bending and with limited localized crushing of the wood fibers near the faces of the wood members is represented by Yield Mode II. This mode is likely to occur in bolted connections with oversized bolt holes and when a dowel has a large diameter. Yielding occurs in both members. The yielding of a dowel fastener at one plastic hinge location is represented by Yield Mode III. The wood fibers in contact with the bolt yield mostly due to the bending of the dowel. Lastly, the yielding of a dowel fastener at two plastic hinge locations is represented by Yield Mode IV and includes limited localized crushing of the wood fibers in contact with the fastener near the shear plane. The four yield modes are depicted in Figure 2.2.

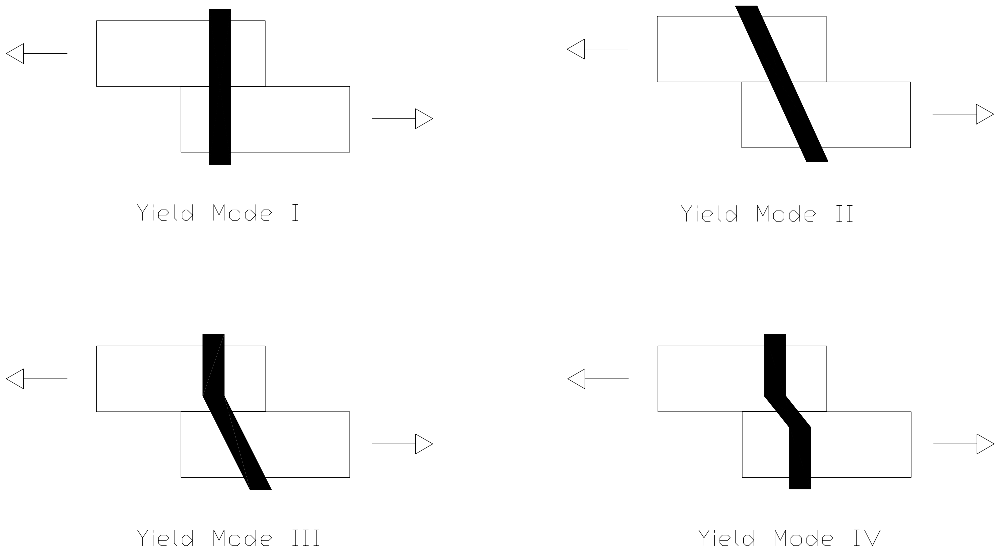

Figure 2.2: Yield Modes for Single-Shear Connections.

2.2.1 Yield Limit Equations

The 2001 edition of NDS gives the following equations to calculate the capacity of various modes through the calculation of the nominal design value, Z. The value of Z used in design shall be taken as the minimum value as calculated by each equation. A subscript of "m" denotes yielding initially occurs in the main member while a subscript of "s" denotes an initial yielding in the side member.

$$Z(I_m) := \frac{D \cdot l_m \cdot F_{em}}{R_d} \quad (2.1)$$

$$Z(I_s) := \frac{D \cdot l_s \cdot F_{es}}{R_d} \quad (2.2)$$

$$Z(II) := \frac{k_1 \cdot D \cdot l_s \cdot F_{es}}{R_d} \quad (2.3)$$

$$Z(III_m) := \frac{k_2 \cdot D \cdot l_m \cdot F_{em}}{(1 + 2 \cdot R_e) \cdot R_d} \quad (2.4)$$

$$Z(III_s) := \frac{k_3 \cdot D \cdot l_s \cdot F_{em}}{(2 + R_e) \cdot R_d} \quad (2.5)$$

$$Z(IV) := \frac{D^2}{R_d} \cdot \sqrt{\frac{2 \cdot F_{em} \cdot F_{yb}}{3 \cdot (1 + R_e)}} \quad (2.6)$$

where:

$$k_1 := \frac{\sqrt{R_e + 2 \cdot R_e^2 \cdot \left(1 + R_t + R_t^2\right) + R_t^2 \cdot R_e^3} - R_e \cdot \left(1 + R_t\right)}{\left(1 + R_e\right)} \tag{2.7}$$

$$k_2 := -1 + \sqrt{2 \cdot \left(1 + R_e\right) + \frac{2 \cdot F_{yb} \cdot \left(1 + 2 \cdot R_e\right) \cdot D^2}{3 \cdot F_{em} \cdot l_m^2}} \tag{2.8}$$

$$k_3 := -1 + \sqrt{\frac{2 \cdot \left(1 + R_e\right)}{R_e} + \frac{2 \cdot F_{yb} \cdot \left(2 + R_e\right) \cdot D^2}{3 \cdot F_{em} \cdot l_s^2}} \tag{2.9}$$

D = diameter of bolt, in.

F_{yb} = dowel bending yield strength, psi

R_d = reduction term

For 0.25” ≤ D ≤ 1”

Yield I_m, I_s: $R_d = 4{*}K_\theta$

Yield II: $R_d = 3.6{*}K_\theta$

Yield III_m, III_s, IV: $R_d = 3.2{*}K_\theta$

For 0.17” < D < 0.25”: $R_d = 2.2$

For D ≤ 0.17”: $R_d = 10{*}D + 0.5$

$K_\theta = 1 + 0.25{*}(\theta/90)$

θ = maximum angle of load to grain for any member in a connection

$R_e = F_{em}/F_{es}$

$R_t = l_m/l_s$

l_m = main member dowel bearing length, in.

l_s = side member dowel bearing length, in.

F_{em} = main member dowel bearing strength, psi

F_{es} = side member dowel bearing strength, psi

According to NDS, the above equations may be used if the following conditions are met:

a) faces of the connected members are in contact
b) the load acts perpendicular to the axis of the dowel
c) edge distances, end distances, and spacing are sufficient to develop full design values
d) the depth of fastener penetration in the main member for single shear connections or the side member holding the point for double shear connections is greater than or equal to the minimum penetration required

The determination of the limiting wood stresses used in the yield model (F_{em}, F_{es}) is described in a standard published by ASTM. Testing and calculation methods are given in the standard D5764-97a(2002) (ASTM, 2003d). To determine the member dowel bearing strength of wood, a fastener rests on a sample piece of wood. The fastener is loaded thus creating a force versus displacement curve. A line is created which is parallel to the initial linear portion of the force-deformation curve. The line is offset from the force-deformation curve at a distance equal to 5 percent of the fastener diameter that is used in the test. The force at which the force-deformation curve intersects the linear line is the dowel bearing strength. This yield point is usually located between the proportional limit and the ultimate limit of the force for the material and for the connection as illustrated in Figure 2.3.

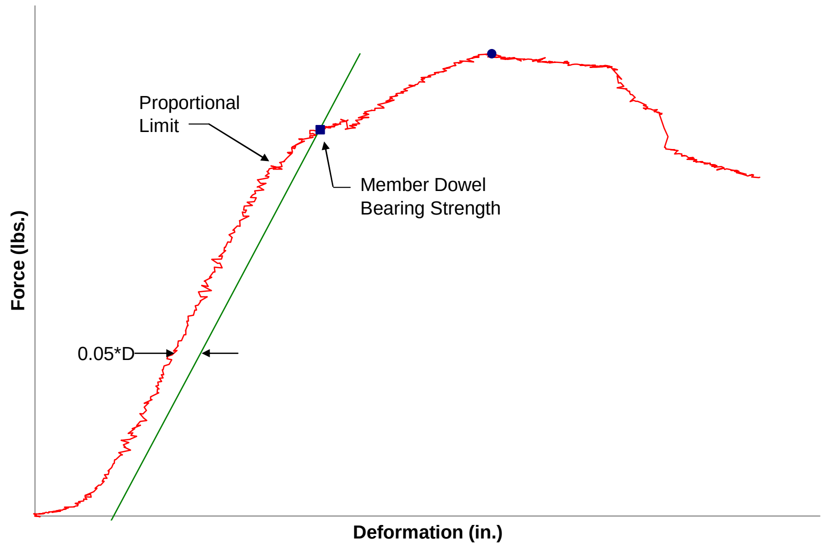

Figure 2.3: Determination of Member Dowel Bearing Strength (ASTM, 2003d).

The determination of the dowel bending yield strength (F_{yb}) is also described in an ASTM standard. Testing and calculation methods are given in the standard F1575-95 (ASTM, 2002). The standard recommends that the dowel be loaded in a three-point bending configuration where the load is applied to the center of the dowel while it is supported by two points. From this test, a force-deformation curve is produced. A line is created which is parallel to the initial linear portion of the force-deformation curve. The line is offset from the force-deformation curve at a distance equal to 5 percent of the dowel diameter. The force at which the force-deformation curve intersects the offset line is the dowel bending yield strength. This yield point is often located between the proportional limit and the ultimate limit of the force for the material and for the connection as illustrated in Figure 2.4.

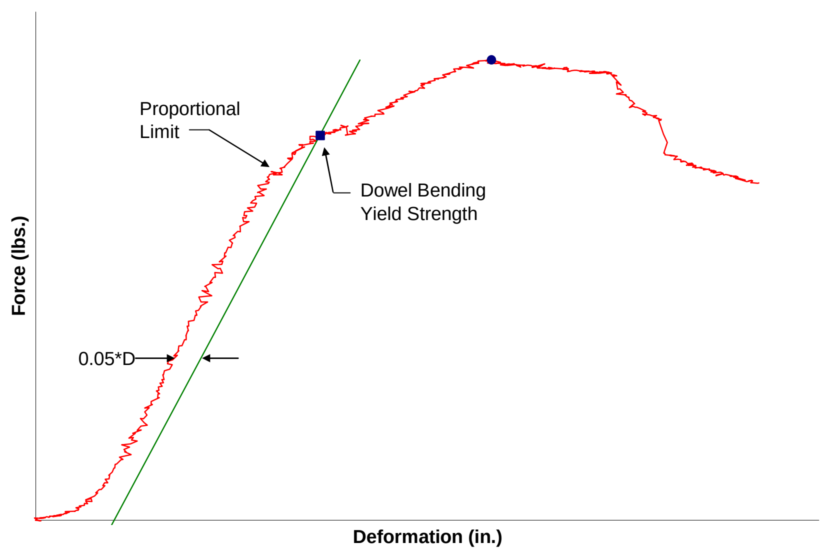

Figure 2.4: Determination of Dowel Bending Yield Strength (ASTM, 2002).

The bolts used in Trayer's 1932 research had an approximate yield strength of 45,000 psi (1932). In 1928, Trayer published research findings on the bearing strength of wood under steel aircraft bolts (Trayer, 1928). He performed the research for the formerly named National Advisory Committee, which is currently known as the National Advisory Committee for Aeronautics (NACA). This research showed that for extremely small length to depth ratios the proportional limits for joints using low-strength bolts were nearly the same as for aircraft bolts which had yield strengths of approximately 125,000 psi. However, for larger length to depth ratios, the proportional limit is considerably higher for the higher-strength bolt. Therefore, the bending strength of the bolt has considerable effects on the proportional limit strength of the joint. While the strength of bolt that Trayer used in his 1932 research was typical of the day, current bolt strengths are higher and the effects of using high-strength bolts in connections should be noted.

2.3 Force Distribution of Multiple-Bolt Connections

According to a model developed by Lantos (1969), the force distribution in main and side members remains constant between dowels and steps at each dowel location in multiple-bolt connections where dowels are the main force transferring mechanism. See Figure 2.5. Force decreases in the side member while increasing in the main member and vice versa. The rate of change depends on modulus of elasticity of the members, number of dowels, tightness of dowel fit, and spacing between dowels. When looking at a connection in section, the total force changes from location to location because the stresses and strains in the main and side members are not equal. The dowel must accommodate the unequal forces in the main and side members. Maximum loads initially develop in the first and last dowels in a row because the unequal force is the greatest at those locations. Because an unequal force distribution exists among dowels in a multiple-bolt connection, the behavior of multiple-bolt connections is different than single-bolt connections. In forming his model, Lantos (1969) assumed that the stresses in the connection members are uniformly distributed across the cross section and that the force-deformation curve for the fastener is linear.

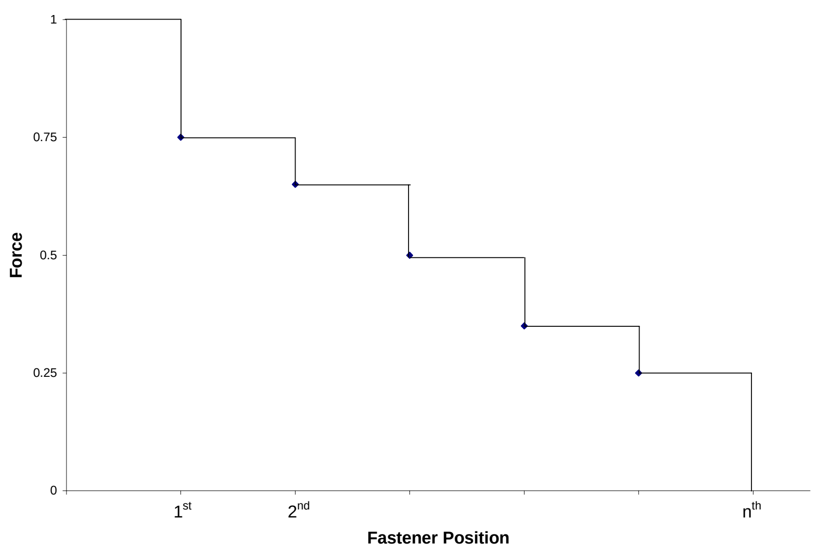

Figure 2.5: Typical Force Distribution in Side Member (Lantos, 1969).

Lantos's (1969) research concluded that stiffer joints have greater deviation from uniform load sharing among bolts in a multiple-bolt connection. Greater deviation also occurs as the number of bolts in the connection increases. An increase in the number of rows increases the strength of the joint, thus, fewer bolts in more rows gives a more efficient connection design. However, this negates the ability of more fasteners in a row to cope with the effects of load distribution.

Lantos (1969) reported that upon initial loading, the first and last bolt in a row will be stressed to allowable limits while intermediate bolts carry only a portion of their allowable loads. As time-dependant deformation occurs, the load is redistributed and the intermediate bolts begin to carry more load while the load in the first and last bolts is reduced. Therefore, design should not be based on final values of deformation because that may cause initial overloading of the first and last bolts (Lantos, 1969).

Research performed by Moss (1996) found when using piece-wise linear load-slip curves for each bolt and including fabrication effects, any bolt may carry the maximum load initially, but at the ultimate load all bolts are near the maximum load on their individual load-slip curves.

In research performed by Wilkinson, he states that the load distribution among bolts for any given connection is unique and is difficult to accurately predict because of the large effects of variability in a single fastener's force-deformation behavior and fabrication tolerances on the load distribution. Therefore, present design methods may be non-conservative since any one bolt may be the major load carrier and any bolt could be misdrilled causing that bolt to not transmit any load for most of the joint loading process (Wilkinson, 1986).

2.4 Group Action Factor

Many previous studies focused on single-bolt connections. Until 1973, the strength of multiple-bolt connections was based on the strength of a single bolt multiplied by the number of bolts in the multiple-bolt connection with out any consideration for the effects of the bolts acting together. This method assumes that the strength and stiffness of single-bolt connections is directly proportional to the strength and stiffness of multiple-bolt connections. Trayer (1932) stated, "Tests of joints having a number of bolts of the same diameter showed that the applied load was equally distributed among the several bolts, provided the bolt holes were carefully centered." However, research by Lantos in 1969 disputed this statement. His findings indicated that individual bolts do not share the applied load equally, thus resulting in localized stresses. The localized stresses cause the wood under some bolts to yield before other bolts as described in the linear elastic model section.

Modifications to account for the premature yielding found in multiple-bolt connections were first introduced in the 1973 edition of the U.S. design code, National Design Specifications for Stress-Grade Lumber and its Fasteners (NFPA, 1973). The code gave the following equation for the design of multiple-bolt connections:

$$P = n * P_{single} * C_g \tag{2.10}$$

where:
- P = multiple-bolt connection strength
- n = number of bolts per row
- P_{single} = single-bolt connection strength
- C_g = group action factor

Current recommendations by NDS for the design of multiple-bolt connections utilize the yield limit equations to calculate the nominal design value, Z (AF&PA, 2001). The nominal design value is then multiplied by the group action factor in addition to other adjustment factors. The formula for the group action factor was formulated by Zahn (Zahn, 1991) and Section 10.3.6 in the 2001 edition of NDS gives the following calculation for the group action factor, C_g:

$$C_g := \frac{m\left(1 - m^{2n}\right)}{n \cdot \left[\left(1 + R_{ea} \cdot m^{n}\right) \cdot (1 + m) - 1 + m^{2n}\right]} \cdot \left(\frac{1 + R_{ea}}{1 - m}\right) \tag{2.11}$$

where:
- C_g = 1.0 for dowel type fasteners with D < ¼".
- n = number of fasteners in a row
- R_{ea} = the lesser of

$$\frac{E_s * A_s}{E_m * A_m} \quad \text{or} \quad \frac{E_m * A_m}{E_s * A_s}$$

E_m = modulus of elasticity of main member, psi

E_s = modulus of elasticity of side members, psi

A_m = gross cross-sectional area of main member, in^2

A_s = sum of gross cross-sectional areas of side members, in^2

$m = u - (u^2 - 1)^{1/2}$

$u = 1 + \gamma^* (s/2)^*\{(1/(E_m{}^*A_m))+(1/(E_s{}^*A_s))\}$

s = center to center spacing between adjacent fasteners in a row, in.

γ = load/slip modulus for a connection, lbs/in.

$\gamma = 180{,}000^*D^{1.5}$ for dowel-type fasteners in wood-to-wood connections

$\gamma = 270{,}000^*D^{1.5}$ for dowel-type fasteners in wood-to-metal connections

D = diameter of bolt, in.

The research which developed the group action factor utilized monotonic loading only and did not consider the effects of cyclic loading on a multiple-bolt connection. Also, the research did not consider ultimate loading conditions. Anderson (2002) concluded that a step function group action factor could be developed for multiple-bolt connections at ultimate, but that the current group action factor should not be used at ultimate loading conditions.

2.5 Effects of Bolt Spacing

In 1932, George Trayer published research in a document titled, *The Bearing Strength of Wood Under Bolts*. The results of his research are the basis of the modern timber connection design method. He ran several hundred tests and produced design methods based on the empirical results. He introduced the term proportional limit (the point at which the test material changes from linear-elastic to non-linear behavior) and made recommendations for end margin (end distance), alignment, and proper bolt spacing for both perpendicular and parallel to grain loading. He also made recommendations on choice of bolt diameter and the combined action of multiple bolts in a connection. Trayer’s design methods continued to be used and extrapolated to account for various design conditions until the Yield Limit Model was introduced in the U.S. design code. His recommendations for design details such as end distance and bolt spacing are still part of the U.S. design codes today (Soltis and Wilkinson, 1987).

In a section of his 1932 report titled *Details of Design*, Trayer found that the center-to-center spacing of bolts for loading acting parallel to grain should be at least four times the bolt diameter (4D). He states that the spacing should actually increase as the bolt length to depth ratio increases if the maximum capacity of the connection is to be developed because the shear stress in the wood is not uniform across the thickness but rather concentrates on the edges. However, he explains that the maximum loads for length to depth ratios greater than 6 will not be much below the maximum obtained with the most favorable spacing. Also the loads at failure will be greater than 2.25 times the safe loads so a spacing of four times the bolt diameter is given as applicable for all cases (Trayer, 1932).

Research performed by Jorissen (1998), which utilized monotonic loading examined the effects of several parameters on the EYM developed by Johansen. Jorissen (1998) performed nearly 1000 tests on single and multiple-bolt connections to examine design details such as spacing, end distance, slenderness ratio, hole clearance, number of fasteners, and number of rows. His results showed that the spacing of the bolts in a multiple-bolt connection under monotonic loading is more or less independent of bolt slenderness, but that spacing is very important for small spacings with the influence reducing with increasing spacing. Load carrying capacity increased with increasing spacing (Jorissen, 1998).

Currently, the NDS (AF&PA, 2001) states that spacing between multiple bolts in a single-shear connection should be four bolt diameters (4D). However, recent studies by Heine (2001) and Anderson (2002) indicate that the 4D recommendations may cause failure in the connection before the calculated yield limit is reached under reverse cyclic loading conditions. Heine (2001) developed a model that uses genetic algorithms to characterize the hysteretic performance of dowel connections. The model has been validated through experiments performed by Anderson (2002). More testing is needed to validate and improve Heine’s model so that the behavior of materials in a connection can be predicted.

2.6 Cyclic Loading

A structure experiences reverse cyclic loading during natural disasters such as earthquakes and hurricanes. The oscillatory movement of the earth during a seismic event causes a structure’s components to alternate between tension and compression loading. During high winds, a structure also experiences changes in loading between tension and compression. To better understand the effects of reverse cyclic loading on connections with multiple bolts, tests must be performed under this loading condition on connections with multiple bolts. The majority of previous research has been performed using monotonic loading. However, monotonic tests do not provide adequate information to evaluate the expected performance of timber joints under reverse cyclic loading.

Much of Trayer's research in the 1932 publication was on joints where the load was applied monotonically, however, he did perform some tests where the load was removed and reapplied. This type of loading would be representative of cyclic loading, but not reverse cyclic loading which includes equal loading in the opposite direction. The results from Trayer's research showed that if the load was kept below the proportional limit than no significant difference was found between the cyclic loading and the monotonic loading. There was a slight increase in deflection when the load was at 25% over the proportional limit load. When the load was 50% over the proportional limit load, the repeated loads soon produced harmful effects. Additional tests showed that there was an increase in deflection in tests where an initial load of 50% over the proportional limit load was applied and removed and subsequent loads were 25% over the proportional limit load. The increase was slight, but indicated that the repetition of loading does cause an increase in deflection versus a single constant loading pattern. Trayer concluded that no harmful change in deflection in the joint results from repeating loading at or below the proportional limit load, but that an increase in deflection occurs when the load is above the proportional limit load.

Mohammad et al. (1998) performed research on bolted connections subjected to ramped cyclic loading. Ramped cyclic loading involves loading a specimen in tension and compression through zero with increasing magnitude for sequential cycles. Results showed that similar modes of failure occurred for the monotonic and cyclic loading, but that more ductility was exhibited before failure in the cyclic loading. The mean residual strength was increased under the cyclic loading while the maximum deformation was reduced. Single-bolt connections were found to be less sensitive to the type of loading than multiple-bolt connections. Multiple-bolt connections exhibited higher mean strength values than single-bolt connections which could be contributed to the redistribution of load. Less deformation and higher stiffness values occurred with the multiple-bolt connections (Mohammad et al., 1998).

Popovski et al. (2002), tested heavy timber connections subjected to monotonic and quasi-static cyclic load. The bolts in the monotonic tension tests formed plastic hinges and yielded, however, failure always occurred by splitting of the wood at the maximum load. In the monotonic tests the connections with slender bolts (ratio of the width of the wood member to the bolt diameter) carried a significant portion of the load after splitting of the main member had occurred for an extended period. In the cyclic tests, the connections with lower slenderness exhibited higher flexural rigidity thus causing more rigid connections with high stresses in the wood which caused sudden wood splitting and loss of bearing capacity. Connections that had smaller bolt diameters (larger slenderness ratios), allowed for more wood crushing to occur before fracture although splitting of the wood was the eventual failure mechanism. Bolted connections with smaller bolt diameters also showed higher energy dissipation, thus, it is reasonable to conclude that for seismic design, slender bolts are more desirable. The cyclic test connections experienced wood crushing at small displacements and transitioned to bolt bending at larger displacements. Generally, the average curves for the monotonic tests showed higher values for maximum load and ductility versus the average envelope curves for the cyclic tests. The deformation for the monotonic tests was also found to be larger as compared to the cyclic tests. Thus, the monotonic test results would be an overestimation or non-conservative estimation of cyclic tests. Popovski et al. state, “ results from monotonic tension tests should be used with caution when determining the seismic properties of a timber connection or component.”(Popovski et al., 2002).

Recently, the Consortium of Universities for Research in Earthquake Engineering (CUREE) developed a testing protocol for deformation controlled quasi-static cyclic testing that considers the non-linear behavior of wood structures (Krawinkler, et al., 2000). This protocol cycles the load through zero and fully reverses the force in the member from tension to compression. The protocol allows the researcher to evaluate the performance of components subjected to ordinary ground motions where the probability of exceedance in 50 years is 10 percent (10/50 hazard level). The 10/50 hazard level is chosen because it is indicative of ordinary ground motion.

Structures have a higher probability of experiencing ordinary ground motions during a seismic event than near-fault ground motions. The loading history does not consider near-fault ground motions, but does consider ground motions due to smaller events that may precede the capacity level event. The basis of the protocol is a set of twenty performance assessment records taken in the 10/50 hazard level near Los Angeles, CA where the ground motion was considered ordinary. More information about the CUREE loading pattern is given in the testing methods section in chapter 3 along with a figure.

The CUREE protocol is based on loading that may be experienced during an earthquake. The effects of high winds on a structure would create a similar type loading, therefore, a separate cyclic loading history does not need to be produced for wind effects.

The International Standards Organization published a new standard in 2003 titled, "Timber Structures- Joints made with mechanical fasteners- Quasi-static reversed-cyclic test method" (ISO, 2003). The standard presents an alternate cyclic displacement schedule than that which was developed by CUREE, however, the schedule is still based on results from monotonic tests. The cyclic displacement schedule is based on an ultimate displacement versus a yield displacement. Foliente, et al. in a paper which discusses the international standard state that because yield displacement is a fictitious parameter, a variety of ways of determining the parameter exist thus producing a variety of values. Therefore, an ultimate displacement is used because the determination is rather straightforward. (Foliente, et al., 1998). The standard recommends creating several load-displacement envelope curves for the cyclic results and reporting the maximum load and ultimate displacement where the ultimate displacement is defined as the displacement at 80% of the maximum load in the descending portion of the envelope curve. Both positive and negative values are reported. The standard states that stiffness, yield displacement, and ductility can be determined according to the definitions adopted in different jurisdictions. No guidance is given for the determination of energy dissipation, but recommends storing the data for future reference (ISO, 2003).

To expand the scientific understanding of the expected response of a wood structure, more research is needed to ensure the safety and competitiveness of wood in the construction market particularly under natural hazard loading conditions. Designers need to understand how the current design practices were developed and whether the methods accurately predict the behavior and strength of multiple-bolt wood connections.