Chapter 2 Literature Review

2.1 – Introduction

In timber structures, connections represent a major part of the overall strength and stiffness that resist a given set of loads. The ability of a connection to transfer loads over an extended period of time, or during a seismic event, has a direct affect on the safety, reliability, and durability of timber structures. Bolted timber connections are generally utilized in either a single-shear or double-shear configuration, indicating the number of shear planes or the number of contact surfaces between members. These simple and effective joints can be varied based upon member thickness, material, bolt diameter, number of bolts, and placement of bolts. Specifically bolts can be arranged in multiple rows with several bolts per row or bolts may be staggered. In general, multiple-bolt connections in wood are configured such that there is uniform spacing between bolts as well as rows.

A critical aspect to the understanding of multiple-bolt timber connections lies in the knowledge that strength and stiffness of single-bolt joints are not directly proportional to the strength and stiffness of multiple-bolt joints (Lantos, 1969). Individual bolts in a row do not share the applied load equally, resulting in high localized stresses which potentially cause fastener and/or wood yielding. These localized areas yield due to wood crushing or fastener bending before other areas have achieved substantial loading. This particular observation led to the development of the group action factor, which is presented in Equation (2.1).

$$P = n \times P_{Single} \times C_g \tag{2.1}$$

where:

P = connection strength,

n = number of bolts per row,

P_{Single} = single-bolt connection strength, and

C_g = group action factor.

The group action factor is a number less than or equal to one, to account for unequal load distribution among bolts in a row. Use of Equation (2.1), based on elastic deflection criteria, addresses a serviceability limit state to ensure that service loads do not produce inelastic connection response. Current design recommendations in the United States for multiple-bolt connections are based on this approach.

Development of the group action factor began in the steel industry when riveted joints failed due to fasteners at the beginning and end of rows attracting higher loads than at transitional locations (Salenikovich et al, 1996). Tests conducted by Stern(1940), Doyle(1964) and Isuymov(1967) initiated the development of the group action factor in timber connections based on similar performance characteristics. This implies that the inclusion of a group action factor in timber design is intended to addresses safety issues associated with bolted connection failure due to unequal load distribution among bolts in a row. An explanation for why a serviceability criterion is used to address a safety issue is unknown to the author at this time.

This chapter provides a review of the literature pertaining to the group action factor currently used in the design of bolted timber connections, and addresses the need for further research to ensure safety and reliability. At the completion of this chapter, the reader should be familiar with the terminology used to describe bolted timber connection performance as well as the group action factor and its influence on performance.

2.2 – Background and Definitions, Single Bolt Connections

To understand the performance of bolted timber connections, it is necessary to present some of the basic methods by which strength, yield, and failure are described. For bolted connections, these descriptions are based on load-displacement behavior and the mechanisms that describe elastic-inelastic response.

The roots of modern bolted timber connection design began with Trayer (1932). Up until this time, methods for computing safe design loads varied extensively due to a lack of physical test data that quantified connection strengths. As a result, Trayer ran several hundred tests on specimens of various configurations in an attempt to provide an understanding of and recommendations for the design of bolted connections. From test data, Trayer produced empirically based design formulae for bolted, double-shear joints and made recommendations about proper bolt spacing, end distance, alignment, and choice of bolt diameter. It was within this work that the term proportional limit stress was first introduced "as the average stress under the bolt when the slip in the joint ceases to be proportional to the load," (Trayer, 1932) (See Figure 2.1). Trayer's work was the basis of bolted timber connection design in the United States for many years (Moss, 1996).

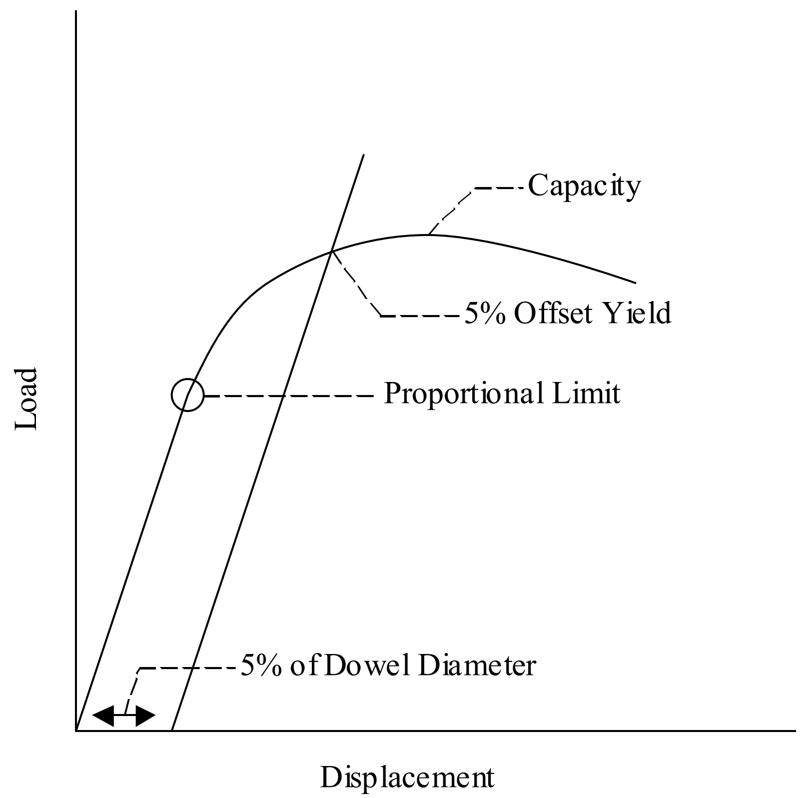

Figure 2.1: Typical monotonic load-displacement curve and associated design parameters (After AFPA, 1997).

Evolution of design specifications for bolted timber connections between the first edition of the *National Design Specification for Stress-Grade Lumber and its Fasteners* (NLMA, 1944) and the 1986 *National Design Specifications for Wood Construction* (NFPA, 1986) saw very little change in approach but significant extrapolation of Trayer's work, creating inconsistencies and confusion based on the interpretation required (McLain, 1991). As a result, the 1991 *National Design Specifications for Wood*

Construction (NDS) (NFPA, 1991) adopted an equation formatted, Yield Limit Model approach to connection design based on earlier work by European researchers. This approach significantly reduced the inconsistencies associated with the repeated extrapolation of earlier work and gave designers a simple, mechanics based, calculation for connection design values.

Development of a Yield Limit Model began in the late 1940's with Johansen (1949). His work utilized basic mechanics to predict yield strength of a single dowel-type fastener's resistance to bending and the resistance of wood to crushing. Continued refinements to the yield model and experimental verification by Mclain and Thangjtham (1983) and Soltis et al (1986) determined that the yield model could, with acceptable accuracy, predict the yield strength of bolted timber joints loaded parallel to grain. However, the onset of yielding in timber is not a well defined point on the load-deflection curve. Based on work by Harding and Fowkes (1984), the 5% offset yield was introduced and became the basis for the description of lateral strength in a single fastener connection. The 5% offset yield is defined as the point where the load-deflection curve is intersected by a line parallel to the linear region, but offset 5% of the dowel diameter (See Figure 2.1). With this criterion, yield strength is predicted based upon the assumed perfectly elastic-plastic behavior of both wood components and dowels. Knowledge of the dowel embedment strength, yield strength of the fastener and basic joint geometry leads to a predicted yield mode and lateral connection yield strength. Yield modes describe the mechanism by which the components of a timber connection are deformed beyond the elastic region. They are illustrated in Figure 2.2 and defined as follows (AFPA, 1997):

- Yield Mode I – Wood crushing in either the main member or side members. Fastener stiffness is greater than wood strength.
- Yield Mode II – Localized wood crushing near the faces of wood members based on the pivoting of rigid fastener about the shear plane.
- Yield Mode III – Fastener yield in bending at one plastic hinge point per shear plane and associated wood crushing.
- Yield Mode IV – Fastener yield in bending at two plastic hinge points per shear plane and associated wood crushing.

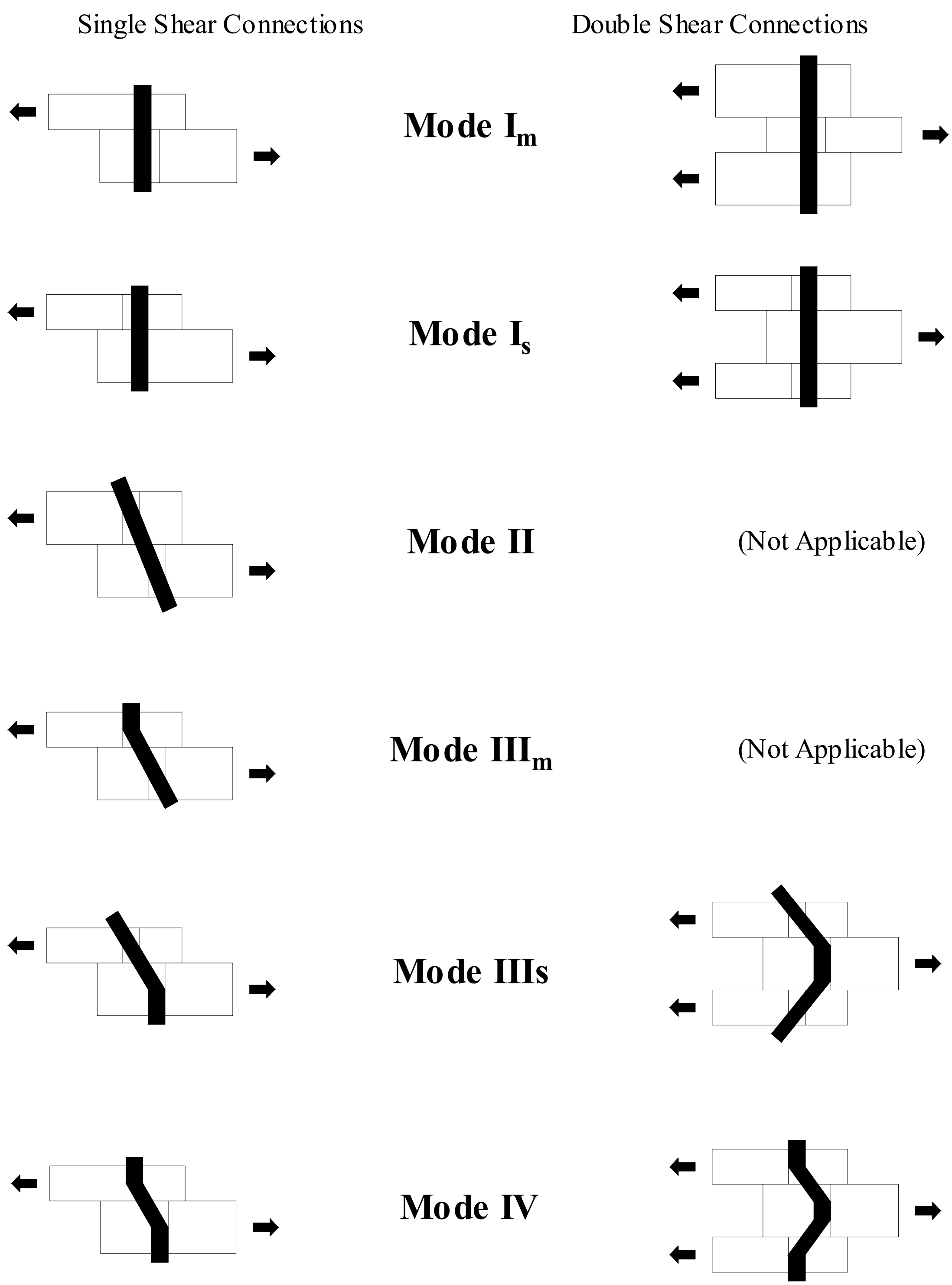

Figure 2.2: Connection yield modes for single and double shear connections (after AFPA, 1997).

The 1997 NDS requires the calculation of lateral connection design values for all applicable yield modes, the smallest value being the design value corresponding to the predicted yielding mechanism. Single-bolt lateral connection strength based upon yield mode for single and double shear connections, is as follows (AFPA, 1999):

Yield Mode I_m:

$$P_{Single} = q_m l_m \tag{2.2}$$

Yield Mode I_s:

Single-Shear $\quad$ Double-Shear

$$P_{Single} = q_s l_s \qquad\qquad P_{Single} = 2q_s l_s \tag{2.3}$$

Yield Mode II-IV:

Single-Shear $\quad$ Double-Shear

$$P_{Single} = \frac{-B+\sqrt{B^2-4AC}}{2A} \qquad\qquad P_{Single} = \frac{-B+\sqrt{B^2-4AC}}{A} \tag{2.4}$$

where:

Mode II, $\quad A = \dfrac{1}{4q_s} + \dfrac{1}{4q_m}$, $B = \dfrac{l_s}{2} + \dfrac{l_m}{2}$, $C = -\dfrac{q_s l_s^2}{4} - \dfrac{q_m l_m^2}{4}$

Mode III_m, $A = \dfrac{1}{2q_s} + \dfrac{1}{4q_m}$, $B = \dfrac{l_m}{2}$, $\quad C = -M_s - \dfrac{q_m l_m^2}{4}$

Mode III_s, $A = \dfrac{1}{2q_m} + \dfrac{1}{4q_s}$, $B = \dfrac{l_s}{2}$, $\quad C = -M_m - \dfrac{q_s l_s^2}{4}$

Mode IV, $\quad A = \dfrac{1}{2q_s} + \dfrac{1}{2q_m}$, $B = N.A.$, $\quad C = -M_s - M_m$

and: $\quad P_{Single}$ = single-bolt lateral connection strength, lbs.,

l_s = side member dowel bearing length, in.,

l_m = main member dowel bearing length, in.,

q_s = side member dowel bearing resistance, lbs./in., = $F_{es}D$,

q_m = main member dowel bearing resistance, lbs./in., = $F_{em}D$,

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

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

D = dowel shank diameter, in.,

F_b = dowel bending strength, psi,

M_s = side member dowel moment resistance, in-lbs. = $F_b\left(D^3/6\right)$,

M_m = main member dowel moment resistance, in-lbs. = $F_b\left(D^3/6\right)$.

(For complete derivation see AFPA (1999) and Heine (2001)).

Although current design procedure concerning bolted timber connections is based on the 5% offset yield, there is a desire to evolve into a capacity based design procedure. The current definition of capacity, or ultimate load, given by ASTM D 1761 (ASTM, 2001) is the load at which rupture occurs or the load at which connection displacement reaches 0.6 in., whichever occurs first. However, Gutshall (1994) defined capacity as either the ultimate load achieved or the load at a displacement of 1 inch. Differences in the definition of capacity stem from attempts to predict a realistic displacement at which load will be transferred to other elements within a structure. As a minimum, it is felt that knowledge of joint capacities will allow an improved understanding of the safety margin between current design values and actual bolted joint capacities. This knowledge would be beneficial in earthquake and hurricane prone regions where connections may be stressed beyond the yield point. In addition, it is likely that the margin of safety between 5% offset yield and capacity will not be uniform across varying bolted joint configurations, due to differing yield and failure modes. Currently, with the 5% offset method, the true margin of safety is not known. The Yield Limit Model can be used to predict the capacity of joints with ductile failure modes but the model overestimates capacity of joints with brittle failure modes such as splitting, plug shear, tension rupture and block shear (Jorissen, 1998).

Jorissen (1998) developed a fracture mechanics based model for bolted double shear timber connections with rigid dowel type fasteners. These joints, corresponding to Yield Mode I, typically produce brittle failure. This model produced a more reliable prediction of load carrying capacity than the Yield Limit Model currently used.

2.3 – Development of the Group Action Factor

A row modification factor, or group action factor, was not introduced in U.S. design codes until the 1973 edition of the *National Design Specifications for Stress-Grade Lumber and its Fasteners*. Until then it had been assumed that for a connection "the total allowable connector loads shall be the sum of the allowable connector loads given for each connector unit used" (NFPA, 1971). In other words the strength of a connection was simply the design value of a single fastener multiplied by the number of fasteners. Development of the Group Action Factor stemmed from observations of unequal load distribution in multiple-bolt connections. Cramer (1968) determined that the safe design of a multiple-bolt connection could not be based on the proportioning of a single-bolt connection. To account for this unequal distribution of forces, the group action factor is included to reduce the likelihood of a failure in multiple-fastener connections.

Observations of non-uniform load distribution among bolts in a row lead to the development of an analytical model by Cramer (1968) in which he developed a linear-elastic model of butt-type joints to predict the distribution of load among bolts in a row. He assumed friction to be negligible, and thus considered only load transferred by bolt shear and bearing. Cramer did recognize that non-uniform stresses occurred around bolt-holes, and accounted for this in his model. He validated his findings with several monotonic tests on perfectly machined joints but noted that a misalignment of bolt-holes may cause large shifts in load distribution among bolts. His findings also noted that, "Ultimate strength tests show some slight redistribution of load from the more heavily loaded end bolts to the less heavily loaded interior bolts when bolt bearing is the mode of failure"(Cramer, 1968).

The year of 1968 also saw the introduction of the Lantos model for load distribution in a row of fasteners. Similar to the work of Cramer, Lantos developed a linear-elastic model of a three-member joint (butt-type joint), but assumed that stresses are uniform across a cross-section. No experimental verification was performed to validate the model.

The Cramer and Lantos models are only valid in the linear-elastic range and only apply to loads acting parallel to grain. This criterion was confirmed by Wilkinson (1980) when he compared Cramer and Lantos model predictions to several connection studies in which bolts were used. Wilkinson found that the models were able to predict the proportional limit strength for a row of fasteners but overestimated capacity due to the linear-elastic assumptions of highly non-linear connections.

The Lantos model is used to determine the group action factor in the 1997 NDS (AFPA, 1997) based on an equation format introduced by Zahn (1991) (Equation (2.5)).

$$Cg = \left[\frac{m(1-m^{2n})}{n\left[(1+R_{EA}m^{n})(1+m)-1+m^{2n}\right]} \right] \left[\frac{1+R_{EA}}{1-m} \right] \tag{2.5}$$

where: Cg = group action factor,

n = number of fasteners in a row,

R_{EA} = the lesser of $\frac{E_s A_s}{E_m A_m}$ or $\frac{E_m A_m}{E_s A_s}$,

E_m = modulus of elasticity of main member, psi,

E_s = modulus of elasticity of side member, psi,

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

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

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

u = $1 + \gamma \frac{s}{2}\left[\frac{1}{E_m A_m} + \frac{1}{E_s A_s} \right]$,

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

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

(For complete derivation see Zahn (1991) and Heine (2001)).

Due to the inherent limitations of a linear-elastic model, it is assumed this derivation of the group action factor satisfies a serviceability limit state for the design of multiple-bolt timber connections. According to Salenikovich et. al. (1996), “In a multiple-bolted connection, nonlinear behavior causes the load distribution among the bolts to change during loading due to load redistribution. Furthermore, time-dependent viscoelastic deformation also reduces the stiffness of the connection and leads to the redistribution of the applied load among the fasteners." This model cannot accurately quantify the performance and safety of these connections beyond service level conditions.

2.4 – Influencing Parameters of Bolted Connection Performance

In an effort to determine what is known and not known about bolted connection performance, Soltis and Wilkinson (1987) did an extensive review of all previous single and multiple-bolt connection studies. Their findings suggest that, among other things, moisture content, spacing, end and edge distances, fabrication tolerances and fastener aspect ratio have a direct effect on the performance of bolted connections. The Yield Limit Model also requires information on the fastener bending strength and dowel embedment strength for a determination of overall joint performance. Other factors that may effect the strength characteristics of a bolted timber connection include friction and bolt tensioning effects.

2.4.1 – Moisture Content Effects

According to Soltis and Wilkinson (1987), a connection at 30 percent moisture content has a 40 percent reduction in proportional limit strength from a connection tested at 12 percent moisture content. This finding is based on research by Doyle and Scholten, (1963); Kunesh and Johnson, (1968); Longworth and McMullen, (1963). These studies were noted as having low fastener aspect ratios lending to the possibility that at higher aspect ratios, corresponding to yield modes II and III, the effect of moisture content may not be as dramatic (Soltis and Wilkinson, 1987).

2.4.2 – Spacing, End and Edge Distances

Originally based on recommendations from Trayer (1932) spacing, end and edge distances have remained relatively unchanged. Current design specifications from the 1997 NDS suggest an end distance of 4 times the bolt diameter for compressive loads and 7 times the bolt diameter for tensile loads to avoid plug shear and splitting type failures for parallel to grain loading. Edge distances and spacing between bolts in a row are still based on these original recommendations. There is currently no known research available that addresses the effect of spacing between rows of bolts, for either staggered or symmetric joints, on the performance of bolted timber connections loaded parallel to grain (Soltis and Wilkinson, 1987).

2.4.3 – Fabrication Tolerances

Current design specifications suggest, that for bolted connections, a hole oversize of between 1/32 in. and 1/16 in. to allow for ease of construction and to account for shrinkage should be provided (AFPA, 1997). Research has shown that fabrication tolerances may be the greatest indicator of performance in multiple-bolt connections (Wilkinson, 1980, 1986). The Lantos and Cramer models of multiple-bolt joints assume perfectly aligned holes thus they conclude that the bolts at the end of rows receive the greatest proportion of the load. Wilkinson has shown that in practice, holes are rarely in perfect alignment and that, due to these misalignments, any bolt in a row may transfer a significant portion of the load.

2.4.4 – Fastener Aspect Ratio

Fastener aspect ratio, referred to in the 1997 NDS, is a quantitative measurement of bolt bearing length, in both the main and side members, divided by bolt diameter; the lesser value governing. For simplicity, in a single shear joint the fastener aspect ratio is defined as:

$$\lambda = \frac{l_m \cdot or \cdot l_s}{D} \tag{2.6}$$

Fastener aspect ratio is used to give an indication as to what affect fastener yield strength and dowel embedment strength have on the overall performance of the joint. Smaller values for fastener aspect ratio indicate a rigid dowel where wood crushing is likely to be the controlling yield mechanism. Larger values indicate slender dowels where bolt bending is likely to be the controlling yield mechanism. Obviously, intermediate values indicate a probable combination of the two yielding mechanisms to varying degrees.

2.4.5 – Fastener Bending Strength

Fastener bending strength is used within the Yield Limit Model to determine if, when, and to what degree a fastener will be stressed into the inelastic region. Increasing aspect ratios in a timber connection indicate that bolt bending will be a yielding mechanism, thus adequate information on fastener yield is required for accurate performance predictions. The Yield Limit Model assumes perfectly elastic-plastic behavior of the fastener, so the full plastic bending stress is calculated based on the plastic section modulus ignoring the effects of strain hardening (Heine, 2001). In the United States, fastener bending strength is based on the 5% diameter offset method.

2.4.6 – Dowel Embedment Strength

Dowel embedment strength is a timber connection property that measures the wood's resistance to crushing when loaded by a dowel type fastener (Wilkinson, 1991). The Yield Limit Model utilizes this property to characterize joint performance. The 1997 NDS suggested design values for dowel embedment strength are based on empirical equations developed by Wilkinson, 1991 (Equation 2.7).

$$F_e = 11{,}200 * G \tag{2.7}$$

where: F_e = parallel to grain dowel embedment strength, psi,

G = specific gravity based on oven dry weight and volume.

The empirical derivation of Equation 2.7 is based on load-deflection data of half-hole test specimens and makes use of the 5% offset method to determine bearing stress. For further discussion on dowel embedment strength see Heine (2001).

2.4.7 – Friction

During construction, individual bolts are typically drawn tight with washer and nut, which introduces tension forces in the bolts and compression forces at the member interfaces. When laterally loaded, these interfaces will develop frictional forces. It is known that, over time, relaxation of the material around the bolt hole will reduce the initial tension and thus reduce the influence of frictional forces. Tests on bolted timber connections are generally run with the nuts "finger-tight" to simulate in-situ conditions (Trayer, 1932, Johansen, 1949). Logic dictates that beyond the proportional limit, when bolts begin to bend or when significant displacement causes washers to be drawn tight to timber members, individual members will be drawn tight and frictional forces will be introduced (Heine, 2001). These frictional forces are neglected in the Yield Limit Model.

2.4.8 – Bolt Tensioning

Directly related to friction at member interfaces, bolt tensioning occurs when fastener end restraints prevent the transverse movement of the bolt in relation to the lateral movement of the timber members. As a result, tensioning of the bolt may coincide with bolt bending. At higher displacements, as long as wood splitting does not occur, it may be possible for significant tension to develop such that necking of the bolt at the location of the plastic hinges occurs. Currently, there is no means to address this issue in the design of bolted timber connections.

2.5 – Test Methods

The most commonly used testing procedure to evaluate mechanical properties of bolted timber connections has been the static-monotonic method, standardized by the American Society of Testing and Materials (ASTM, 2001). Monotonic testing is generally referred to as pseudo-static, in that it simulates a static load, over a small increment of time, by applying the load at a slow and constant rate. Static-monotonic testing has generally been used because of the ease with which test apparatus can be set up and the efficiency with which multiple tests can be run. Current research indicates that this type of testing procedure does not yield sufficient information about connection performance in earthquake prone regions (Foliente, 1996). Many regulatory agencies, such as the Office of State Architect of California (OSACA), International Conference of Building Officials (ICBO) and National Earthquake and Hazards Reduction Program (NEHRP), now require performance characteristics of structural elements be determined by either cyclic or dynamic test methods.

Dynamic testing is conducted to determine the force-displacement behavior of structural elements exposed to earthquake conditions. Earthquake ground motions input an oscillatory motion on structures and structural elements in such a manner that loads and displacements are cyclic. Load-displacement plots are typically hysteretic in nature because of inelastic behavior (Chopra, 1995). Behavior of systems exposed to dynamic loading is partially defined by inertial effects that occur when the system mass is exposed to time dependent accelerations. For this reason, dynamic testing requires sophisticated test equipment, software and analysis procedures.

Currently, a considerable amount of structural assemblies are tested using pseudo-static (a.k.a. pseudo-dynamic), cyclic methods. This method utilizes cyclic excitation in a pseudo-static manner allowing the inertial forces to be neglected in the analysis but still mimicking earthquake conditions by fully reversing the direction of imposed displacements. Furthermore, these tests can be displacement driven similar to the seismic excitation of structures.

At the present time there is no standardized procedure for the development of pseudo-static, cyclic displacement protocols although draft standards are under consideration. The ASTM draft standard (7^{th} Draft: cyclic test standard, 1999), based on the Sequential Phased Displacement (SPD) Protocol, the CEN proposed standard (prEN 12512 Draft 1996), ISO proposed standard (Draft 1998), and ST04 – Draft New Zealand Standard are all currently under review. For a complete overview of these standards see Heine (2001). Current interest is on the newly developed CUREE Displacement and Force Controlled Quasi-Static Cyclic Protocol because it is the only protocol that is based statistically on previously recorded earthquake ground motions.

2.5.1 – Mohammad and Queeneville

In an effort to determine the influence of cycled loads on the strength and stiffness of bolted connections, Mohommad and Queeneville (1998) developed a force driven cyclic protocol applied at 1 cycle per minute to both parallel and perpendicular to grain specimens. The materials tested were Spruce-Lodgepole Pine glue laminated timber connected with Grade 2 bolts. Test configurations included single and two row, multiple-bolt patterns, up to eight bolts per row, with ten replications per series. The loading sequence required monotonic tests to be performed to determine the monotonic capacity of each connection. Loads were then applied to the specimens in a cyclic manner such that 10% of the monotonic capacity was reached in tension followed by a reversal of load direction until the same load was reached in compression. Consequent cycles included an additional 10%, of the monotonic capacity, increase until failure was reached.

Parallel to grain test specimens were loaded in a double-shear configuration utilizing steel side plates. The test fixture was set up such that deformation was measured at one location transverse to load direction with two LVDT's.

The conclusions from these tests state that similar modes of failure can be expected for cyclic and monotonic loaded specimens. For parallel to grain connections the mean residual strength of cyclically loaded specimens can improve by a factor of 1.16 over monotonic values.

2.5.2 – CUREE Displacement Controlled Quasi-Static Cyclic Protocol

Development of the CUREE Displacement and Force Controlled Quasi-Static Cyclic Protocol addressed the need for a common testing protocol for component tests outlined in the CUREE-Caltech Woodframe Project; a project funded primarily by the Federal Emergency Management Agency (FEMA) (Krawinkler et. al., 2001).

> "The development of loading histories is based on results of nonlinear dynamic analysis of representative hysteretic systems subjected to sets of ordinary and near-fault ground motions. Cumulative damage concepts are employed to transform time history responses into representative deformation and force controlled loading histories." (Krawinkler et. al., 2001).

In particular, a deformation controlled protocol is developed that represents the loading history for ordinary ground motions, where the probability of exceedance in 50 years is 10 percent. It is based on twenty ground motion records in the California region and utilizes a rainflow counting method to determine sequence.This specific case was implemented in this study for all bolted connections exposed to deformation controlled, pseudo-static cyclic loading where capacities are to be determined. A detailed review of the protocol generation is given in Chapter 3.

2.6 – Summary

Based on the preceding review of literature concerning bolted timber connections and the group action factor, the following statements can be made:

- Some questions exist as to whether the current group action factor provides an adequate margin of safety in overloaded connections.
- Current methods for determining a group action factor are unconservative when determining multiple-bolt monotonic connection capacity.
- The European Yield Model overestimates the strength of single-bolt connections, loaded monotonically, when the failure mode is brittle in nature.
- Monotonic loading may not accurately represent the strength and ductility requirements of structural components in areas prone to seismic disturbances.
- Cyclic loading protocols should be based on seismic event analysis to provide a rational and reliable estimation of connection capacity.