Structural reliability of nailed connections

D. I. MASSÉ[1] and J. J. SALINAS[2]

[1]*Engineering and Statistical Research Centre, Research Branch, Agriculture Canada, Ottawa ON, Canada K1A 0C6; and* [2]*Department of Civil Engineering, Carleton University, Ottawa, ON Canada K1S 5B6. Contribution no. I-935*[1]*, received 5 August 1987, accepted 31 January 1989.*

Massé, D. I. and Salinas, J. J. 1989. **Structural reliability of nailed connections**. Can. Agric. Eng. **31**: 195–203. The reliability of spliced, nailed, connections is assessed taking into account the random nature of the loads and material strength. The safety of each component (lumber, plywood and nails) is evaluated in terms of the reliability index, β, and reliability bounds on the system are evaluated and compared with target values. This approach makes it possible to identify those components which affect most significantly the safety of the connection. The reliability index for individual components was found to be above generally accepted levels. The lumber components were found to have the lowest reliability index and they have the largest effect on the overall system reliability. The level of safety associated with nailed connections designed using nail capacity values given by the Limit States Design Code is less conservative and more realistic than that associated with connections designed using Working Stress Design.

INTRODUCTION

The traditional approach for determining the factor of safety of nailed connections involves calculating the ratio of nominal ultimate capacity to design load. The approach, although easy to follow, may result in misleasing information concerning the reliability of the connection because it does not take into account the random nature of the loads and strength of the components. The methodology proposed in this study takes into consideration the uncertainties associated with the loads as well as the variability found in the material properties. The connections considered for this study have three basic components: lumber, plywood and nails. The structural reliability of each component can be evaluated separately and, at a later stage, unimodal bounds can be calculated for the overall system reliability. This approach is outlined here and can be used to assess the safety of current design assumption, and to evaluate new provisions or specifications.

The objective of this study is to evaluate the safety of nailed connections taking into consideration the random nature and the uncertainties associated with material strength and loads. Figure 1 shows the components of a typical bottom chord tension splice of the type commonly used in roof trusses for agricultural buildings. The connection has three basic components: lumber, plywood and nails, all of them with various degrees of variability in strength and subject to loads of uncertain magnitude or duration. The following material properties and loads will be considered in this investigation.

1. Material strength

(a) Lumber
  (i) Tensile strength parallel to grain (F_t).
  (ii) Compression parallel to grain. Embedding strength $(F_e)L$.

(b) Plywood
  (i) Tensile strength parallel to grain in face veneer (F_{tp}).
  (ii) Compression parallel to grain in face veneer. Embedding strength $(F_e)_P$.

(c) Nails
  (i) Yield strength (F_y).
  (ii) Plastic moment (M_y).

2. Loads

(a) Dead load, nominal design value (*Dn*)
(b) Dead load, actual value (*D*)
(c) Live load, snow, nominal design value (*Ln*)
(d) Live load, snow, actual value (*L*)
(e) Live/dead load ratio (*Ln/Dn*)
(f) Ratio of Lifetime maximum to nominal snow load (*L/Ln*)

COMPONENT RELIABILITY

The reliability analysis of a structural component can be formulated as a "supply demand" problem where the "supply" is characterized by the capacity of the component, represented by the resistance variable, R, and the "demand" is represented by the load variable, L. The performance function is given by:

$$Z = R - L \tag{1}$$

The limiting performance of the component is given by $Z = 0$ which defines a limit state in which there is no failure nor excess capacity in the component. The failure state is given by $Z < 0$ and the safe state by $Z > 0$.

The probability of failure can be calculated by determining the probability of occurrence of the failure state, $Z < 0$. Failure occurs when $R < L$ and for given load $L = u$ the failure event is the joint occurrence of $R < L$ and $L = u$. Consequently, the probability of failure is calculated by multiplying the probabilities associated with these two events. The probability of failure, for all $-\infty \leqq u \leqq +\infty$, is given by:

$$P_f = \int_{u=-\infty}^{u=+\infty} f_L(u) d_L \int_{R=-\infty}^{R=L=u} f_R(u) dR \tag{2}$$

Alternatively, the probability density function for Z, f_z, can be derived from the probability density function of the resistance, f_R, and the probability density function of the load, f_L. The reliability index, β, can be defined as the ratio of the mean to the standard deviation of the derived distribution for Z.

$$\beta \ \frac{m_z}{\sigma d_z} \tag{3}$$

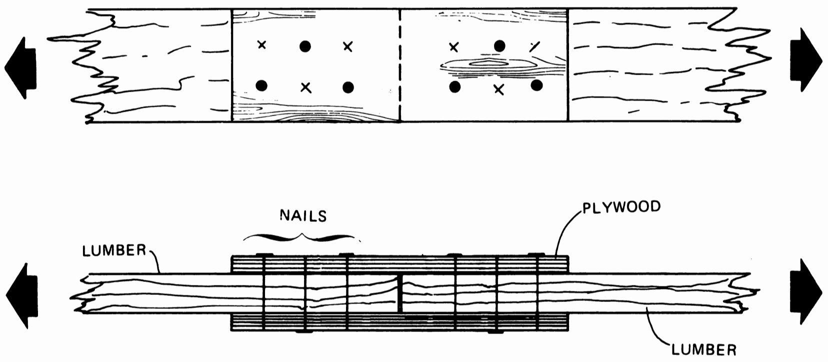

Figure 1. Typical bottom chord tension splice.

Table I. Target reliability index, β, for steel and concrete buildings, ultimate limit state, 30-yr life (from CSA special publication S408-1981)

	Type of failure	
Safety class	Gradual	Sudden
Not serious	2.5	3.0
Serious (normal buildings)	3.5	4.0
Very serious	4.0	4.5

Table II. Lumber tensile strength parallel to grain (*MP*a) (after Madsen and Nielsen (1978))

Species	Grade	Size	Mean	COV
D. fir	No. 2	38 × 89	31.2	0.40
		38 × 140	29.5	0.35
		38 × 184	29.7	0.44
		38 × 235	26.2	0.38
S-P-F	No. 2	38 × 89	23.9	0.36
		38 × 140	24.7	0.38
		38 × 184	21.1	0.33
		38 × 235	16.9	0.25

Table III. Plywood tensile strength parallel to grain in face veneer, (*MP*a) (after Stieda and Parasin (1985))

Species	Size	Mean	COV
D. fir	3 & 4 plies	53.4	0.40
	5 plies or more	50.0	0.30
Softwood	3 & 4 plies	40.4	0.40
	5 plies or more	40.4	0.40

Table IV. Nail yield strength (MPa)

Nail type	Size (mm)			
	Diameter	Length	Mean	COV
Common	3.25	64	750†	0.042
	3.66	76	750†	0.038
Concrete	3.60	64	2704‡	0.028
	4.40	76	2356‡	0.021

†Based on experimental work by Aune and Patton-Mallory (1986b).
‡Based on unpublished experimental work by Salinas and Massé (1986 unpubl.).

If F_z is assumed to be normal, the probability of failure and the reliability index are related by:

$$P_f = 1 - \Phi(\beta) \tag{4}$$

where $\Phi(\beta)$ = cumulative distribution function of the standard nominal variable, evaluated at β.

If the actual probability density functions for R and L are approximated by log-normal functions that fit in the significant tails, CSA SP S408-1981 (1981) suggests the following approximation for the reliability index:

$$\beta = \frac{\text{Ln}\,(m_R/m_L)}{\sqrt{v_R^2 + v_L^2}} \tag{5}$$

where m refers to the mean and V to the coefficient of variation (standard deviation divided by the mean). Recommended target values for β are given in Table I.

The analytical formulation of the function describing the reliability index is not a trivial problem. A more involved mathematical derivation is being currently considered for future publication.

MATERIAL PROPERTIES

Figure 1 shows a typical tension splice in the lower chord of a CPS truss. The strength of each component has been evaluated based on the material properties described in the following sections.

Lumber

Table II gives the mean tensile strength of S-P-F and Douglas fir, No. 2 lumber for various sizes. These values were obtained from in-grade test values reported by Madsen and Nielsen (1978). The high coefficient of variation (COV), ranging from 0.35 to 0.44, plays an important role in the evaluation of the reliability of this component.

Plywood

Table III gives the mean tensile strength of Douglas fir (D.fir) and Canadian softwood plywood. As can be observed the plywood exhibits higher tensile strength parallel to grain (face veneer) than the corresponding species for lumber. The COV is similar to that of lumber. These values were taken from studies by Stieda and Parasin (1985).

Nails

Table IV gives the yield stress for common wire and concrete nails. This measure of nail performance is used in determining the nail's bending strength required to assess its failure mode in a nailed connection. The values reported in Table IV were obtained from two independent experimental programs, and can

Table V. Nail embedding strength (MPa)†

Material	Species and grade	Nail size (mm)	Mean	COV
Lumber	D. fir/No. 2	3.25	41.6	0.22
		3.66	41.6	0.22
		4.10	41.6	0.22
		4.90	43.7	0.18
	S-P-F/No. 2	3.25	30.0	0.11
		3.66	30.0	0.11
		4.10	30.0	0.11
		4.90	30.0	0.11
Plywood	D. fir	3.25	51.6	0.18
		3.66	51.6	0.18
		4.10	51.6	0.18
		4.90	41.4	0.18
	Softwood	3.25	39.4	0.13
		3.66	39.4	0.13
		4.10	39.4	0.13
		4.90	39.4	0.13

†Based on experimental work by Aune and Patton-Mallory(1986b).

Table VI. Ratio of maximum lifetime to nominal snow load, α, 30-yr recurrence period, (after Sexsmith and Fox (1978))

Location	Mean $(m\ \sigma)$	COV $(V\ \sigma)$
Calgary	1.04	0.18
Montreal	1.08	0.18
St. John's	1.11	0.19
Toronto	1.23	0.17
Vancouver	1.21	0.22
Winnipeg	1.08	0.20

be seen to vary as a function of the nail type and diameter. Low COV is typical of steel.

Table V shows the embedding strength (compression parallel to grain) of lumber and plywood. This value is also needed to assess the failure mode of nailed connections. Although the COV is similar for both materials, plywood has a higher embedding strength. These values were obtained from the experimental work of Aune and Patton-Mallory (1986a,b).

LOADS

Dead and live loads used in the analysis correspond to those commonly used for CPS trusses. To account for the geographic variability associated with nominal snow loads, the results of a study by Sexsmith and Fox (1978) were incorporated in the analysis. Table VI gives the measured ratio of lifetime maximum to nominal snow load for six Canadian cities.

The effect of live to dead load ratio was studied over the range of values 2, 10 and 20, which covers service conditions commonly found in Canada.

RELIABILITY

Equation 5 was used to calculate the reliability index, β, for the connection components: lumber, plywood and nails. This equation makes use of the information gathered concerning the material strength and loads. The values obtained should be compared with the target values reported in Table I. A value of β in the neighborhood of 2.5, meeting the requirements for the 'not serious' safety class, appears to be adequate for farm buildings with low human occupancy, as defined in the Canada Farm Building Code (1983).

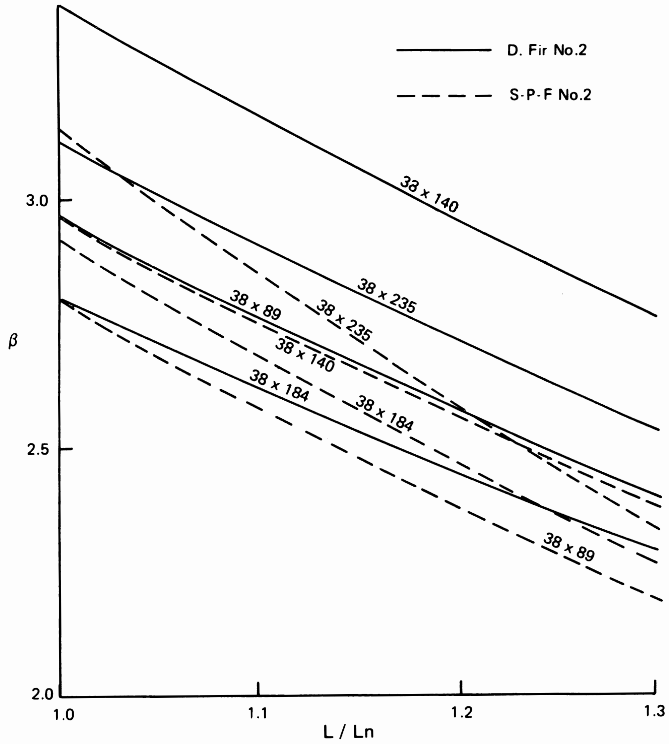

Figure 2. Component reliability, lumber. High human occupancy. Limit states design. $L_n/D_n = 10$.

RESULTS

The results index, β, for individual components was calculated and comparisons made for the following cases:

1. Occupancy
   (a) High human occupancy. Residential, industrial and commercial buildings.
   (b) Low human occupancy. Farm buildings and structures.
2. Design philosophy
   (a) Limit states design (LSD).
   (b) Working stress design (WSD).
3. Species/type
   (a) Lumber, S-P-F and D. fir, No. 2.
   (b) Plywood, D. fir and Canadian softwoods.
   (c) Nails, concrete and common wire nails.
4. Size
   (a) Lumber: 38 × 89, 38 × 140, 38 × 184 and 38 × 235.
   (b) Plywood: 12.5 mm and 18.5 mm. Four and five plies.
   (c) Nails: diameter — 3.25, 3.66, 4.1, 4.4 mm; length — 64 and 76 mm.

Lumber

Figure 2 shows the reliability index, β, for S-P-F and D. fir lumber when resisting tensile axial forces in a high human occupancy structure designed using limit states design philosophy. This index is evaluated for a range of values of lifetime maximum to nominal snow loads and for all commonly used lumber sizes. As can be observed in Fig. 2 there is an obvious size effect as well as a difference between the two species studied. A noticeable drop in the reliability index can be observed as the load ratio increases. For a lifetime maximum to nominal snow load ratio of 1.0 β ranges from 2.80 to 3.40.

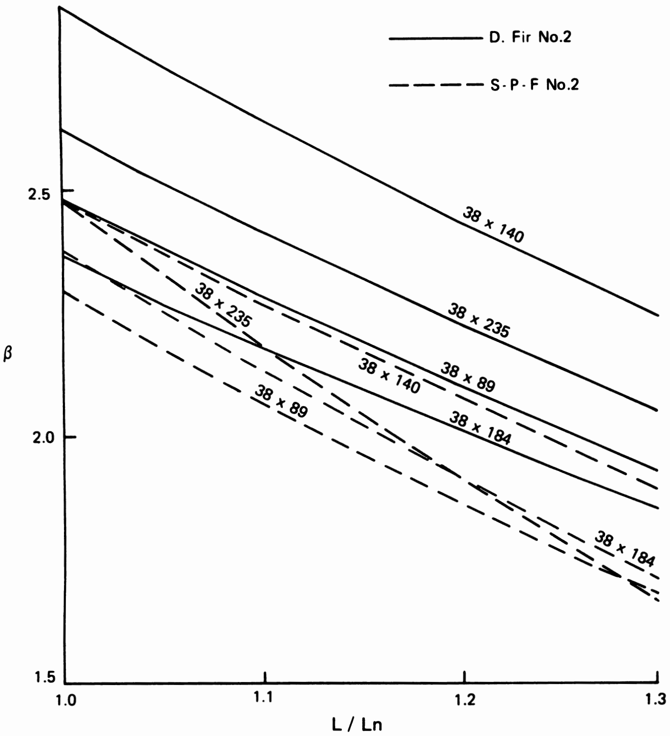

Figure 3. Component reliability, lumber. Low human occupancy. Limit states design. $L_n/D_n = 10$.

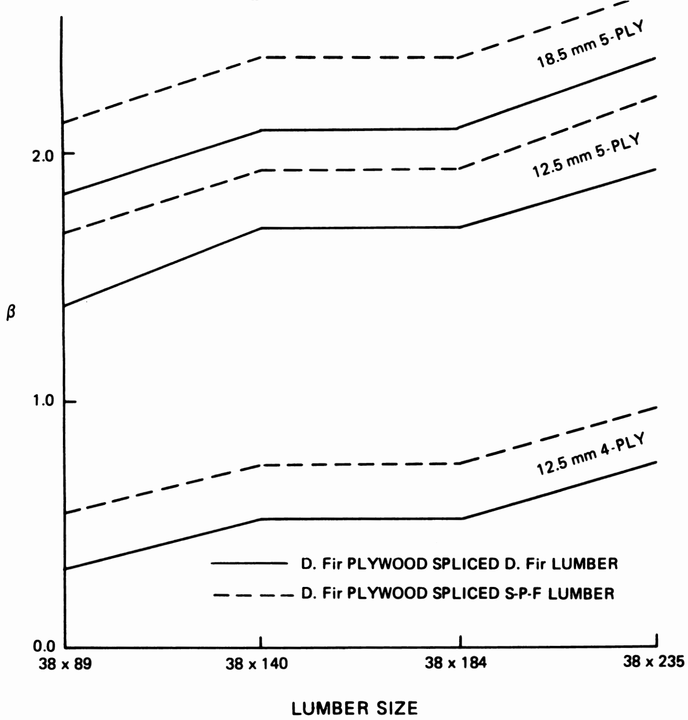

Figure 4. Component reliability, plywood (D. fir). Low human occupancy. Limit states design. Ratio of lifetime $L_n/D_n = 10$, $L/L_n = 1.12$.

For farm structures with low human occupancy an importance factor of 0.8 is commonly applied to the live load part, the reliability of the lumber components is reduced, as can be observed in Fig. 3. The corresponding values of β for a lifetime maximum to nominal snow load ratio of 1.0 range from 2.30 to 2.87.

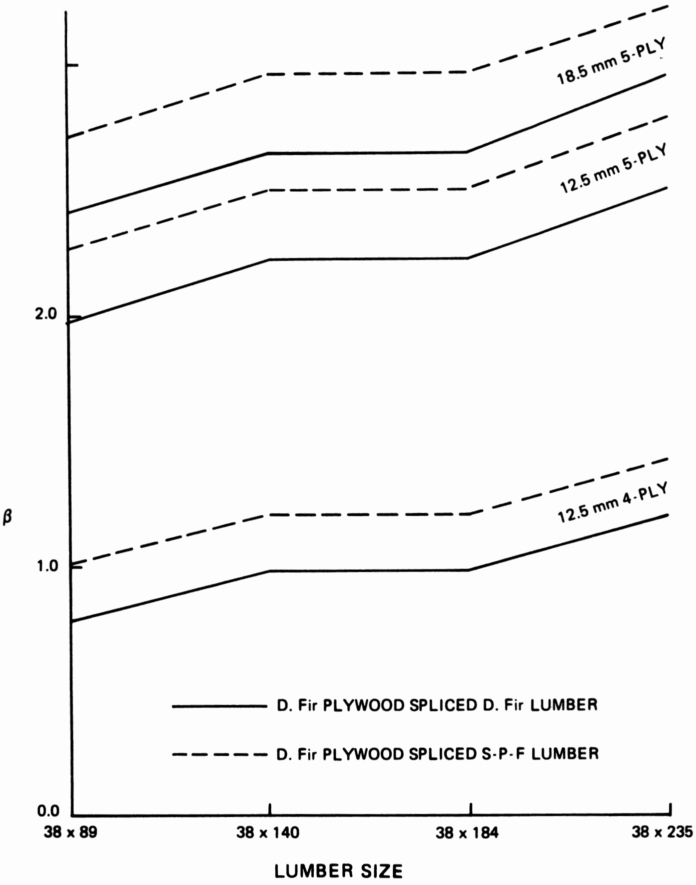

Figure 5. Component reliability, plywood (D. fir). High human occupancy. Limit states design. $L_n/D_n = 10$, $L/L_n = 1.12$.

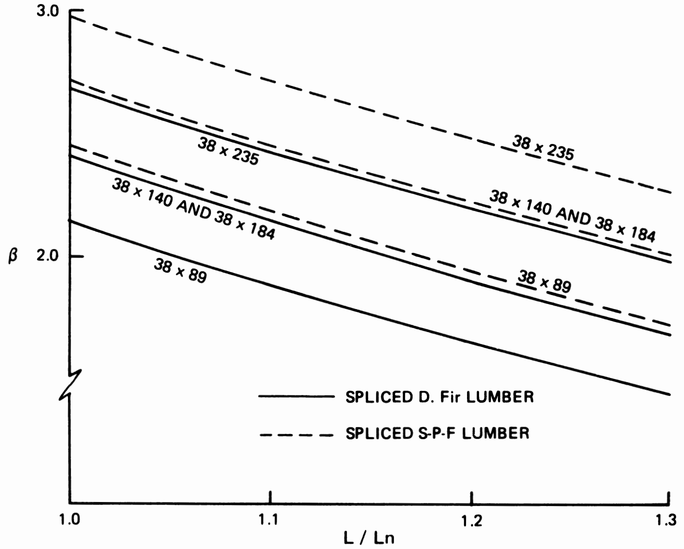

Figure 6. Component reliability, plywood (18.5 mm D. fir): Low human occupancy. Limit states design. $L_n/D_n = 10$.

Plywood

Where the width of plywood gussets is chosen to match that of the lumber components it splices, as in the lower chord of trusses, plywood thickness is chosen to develop a tensile strength as close as possible to that of the lumber. Figures 4 and 5 show the range of values for the reliability index, β, of D. fir plywood gussets splicing different lumber sizes and species for conditions of low human occupancy and high human occupancy.

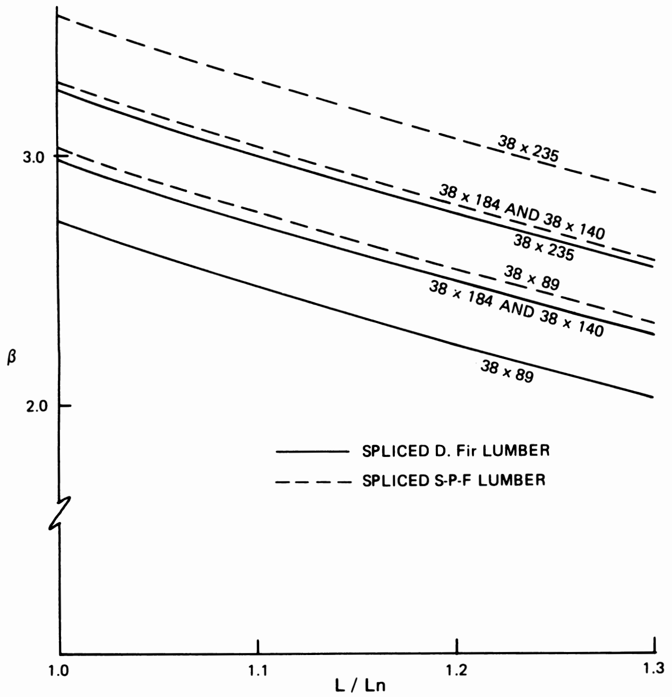

Figure 7. Component reliability. Plywood (18.5 mm D. fir): High human occupancy. Limit states design. $L_n/D_n = 10$.

A general trend can be observed showing an increase of plywood reliability as the lumber component size increases. This appears to be a direct result of the larger size (depth) factors recommended by CAN3-086.1-M84 (1984a) for the design of lumber components in tension parallel to grain. In order to develop the full tensile capacity of a lumber component, plywood gussets used with the smaller lumber sizes (38 × 89) are more stressed than those used for the larger lumber sizes (38 × 235).

Plywood gussets 12.5 mm thick and with four plies exhibit unacceptably low levels of reliability under all conditions. This situation improves for material with five plies with values of β between 2.28 and 2.80 for the 12.5-mm-thick material and between 2.74 and 3.24 for the 18.5-mm-thick plywood. Connections used under low human occupancy conditions exhibit reliability levels considerably lower than those under high human occupancy conditions. Figures 4 and 5 also show that the reliability levels associated with D. fir plywood splicing S-P-F lumber (No. 2) are higher than those associated with D. fir lumber components.

Figures 6 and 7 show the effect of the ratio of lifetime maximum to nominal snow load on the reliability of D. fir plywood gussets. A general trend observed, for all lumber sizes and for both occupancy conditions, is that reliability index values decrease as the load ratio increases. The values of the load ratio considered in this study are representative of climatic conditions found across Canada.

Nails

The behavior of a nailed connection depends on a large number of factors including the physical, mechanical and geometric properties of the components. In this study, the failure modes investigated include:

1. Crushing of the plywood under the nail.
2. Crushing of the lumber under the nail.
3. Nail yield in bending leading to localized crushing of plywood and lumber.

Figure 8 shows these three different failure modes. The theoretical capacity of the connection, F_u, is based on a model first proposed by Johansen (1949) and later refined by Larsen (1963) and verified experimentally by several investigators under different test programs.

Table VII shows the formulas to determine the lateral strength resistance of a nailed connection for the three failure modes. The failure mode resulting in the smallest capacity, F_u, controls.

Table VIII lists the failure modes controlling the behavior of the various splice configurations considered in this study. As can be observed in this table, mode 3 dominates the behavior of connections using common wire nails. The behavior of connections using the stronger and stiffer concrete nails is controlled by modes 2 and 1. Mode 2 is associated with D. fir lumber components, independently of plywood thickness or number of plies. Mode 1 is associated with S-P-F lumber components. The above comparisons are made for D. fir plywood gussets.

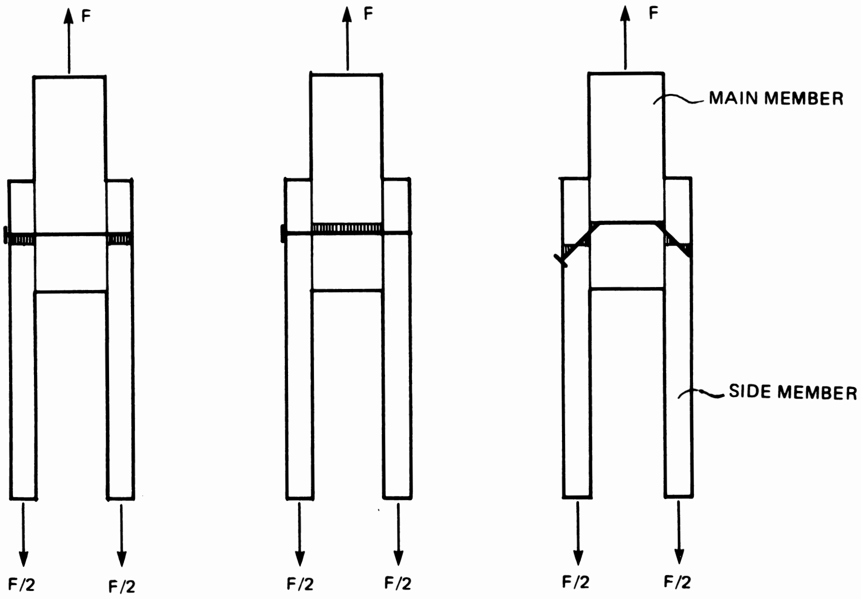

Figure 8. Failure modes investigated.

Table VII. Theoretical capacity of nailed wood joints

Failure mode	Capacity	
1 (Lumber)	$F_u=(F_e)_L\ D_n\ t_L$	
2 (Plywood)	$F_u=2(F_e)_P\ D_n\ t_P$	
3 (Nail yield)	$F_u=2(F_e)_P$	$\left[\sqrt{\dfrac{2\ (1+\beta)t_P^2}{(2+\beta)^2}+\dfrac{4\ \beta\ \gamma}{(2+\beta)}}-\dfrac{\beta t_P}{(2+\beta)}\right]$

$(F_e)_{L,P}$ = Embedding strength of lumber, plywood

D_n = nail diameter

$t_{L,P}$ = thickness, lumber, plywood

$\beta = \dfrac{(F_e)_P}{(F_e)_L}$

$\gamma = \dfrac{M_y}{(F_e)_L}$

M_y = nail yield moment.

Table VIII. Theoretical failure modes†

	D. fir plywood		Nails		Failure
Lumber	Thickness	Plies	Diam. (mm)	Type	mode
D. fir	12.5 mm	4	3.60	Concrete	2
		4	3.25	Common	3
		5	3.60	Concrete	2
		5	3.66	Common	3
	18.5mm	5	4.40	Concrete	2
S-P-F	12.5 mm	4	3.60	Concrete	1
		4	3.25	Common	3
		5	3.60	Concrete	1
		5	3.66	Common	3
	18.5 mm	5	4.40	Concrete	1

†Based on a yield model proposed by Aune and Patton-Mallory (1986a).

Table IX. Comparison of theoretical and experimental lateral strength resistance. Lumber: S-P-F No. 2, Plywood: D. fir/5-ply, Nails: concrete

Plywood	Nail	Capacity (kN)		Ratio
thick.	diam. (mm)	Theory†	Exp.‡	Theor./exp.
12.5 mm	3.60	4.10	3.91	1.04
18.5 mm	4.40	5.02	5.40	0.93

†Based on a theoretical yield model proposed by Aune and Patton-Mallory (1986a).

‡Based on experimental values obtained by Massé et al. (1986).

Table IX compares the capacity of connections using S-P-F lumber, D. fir plywood gussets, and concrete nails, for two different plywood thicknesses. First the capacity is calculated using the theoretical model proposed by Aune and Patton-Mallory (1986a). This theoretical capacity is then compared with values obtained experimentally by Massé et al. (1986). As can be observed, theoretical and experimental values are in good agreement. The analytical model was then used to predict the capacity of connections using common wire nails.

Figure 9 shows the variation in the reliability index, β, with the lifetime maximum to nominal snow load ratio, for connections using concrete and common wire nails, D. fir plywood gussets and two lumber species. These connections, designed using an LSD approach and under low human occupancy conditions, exhibit β values ranging from 3.54 to 5.18 for concrete nails. For common wire nails β values range from 3.98 to 5.54 for S-P-F lumber components. A significantly lower range, from 2.15 to 3.74, was found for D. fir lumber which favored a mode 3 type of failure and a higher design load. A significant decrease in the reliability index is noticeable for both types of nails for increasing load ratios.

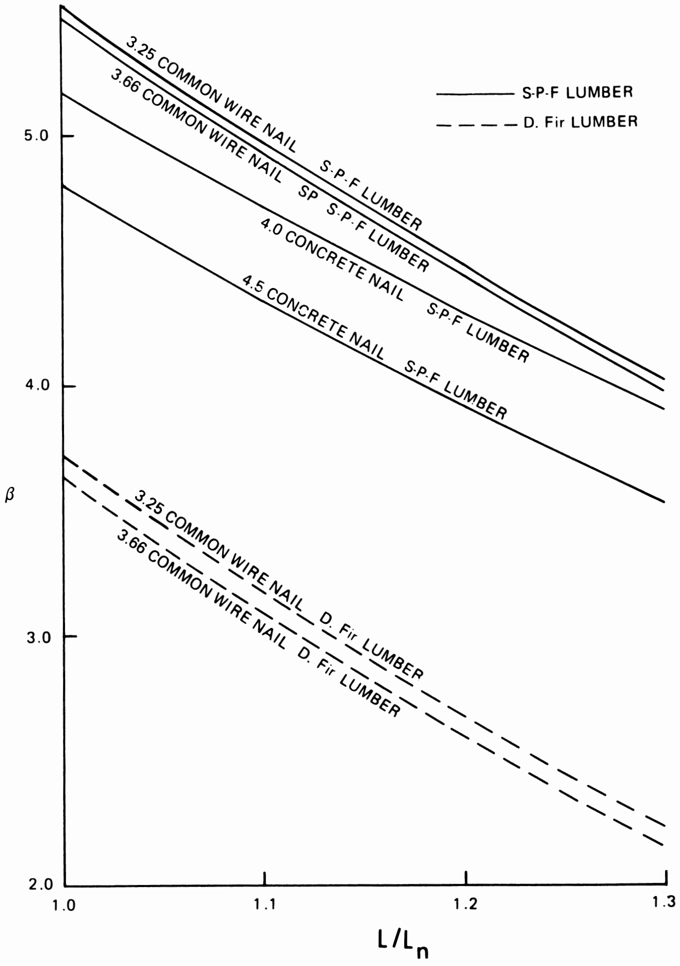

Figure 9. Component reliability. Nails: D. fir. Low human occupancy. Limit states design. $L_n/D_n = 10$.

Design philosophy: LSD vs. WSD?

Working stress design values for nailed connections have been derived from tests where the strength corresponding to the assumed proportional limit in the load-deformation curve was used as the basis for determining code design values. Limit states design values have been derived from an analytical model and calibrated with experimental results. In general, connections designed using LSD result in fewer nails than those designed using WSD. As a result, the nails in connections designed using LSD design values tend to be highly stressed and exhibit lower reliability indices. As shown in Figs. 10 and 11, the WSD philosophy is considerably more conservative than the LSD philosophy. Nonetheless, the values of the reliability index, β, associated with an LSD design philosophy are more realistic and more in line with values of reliability found in the other components.

Load effects

Figure 12 shows the effect of lifetime maximum to nominal snow load ratio, L/L_n, on the reliability index of the components of a typical nailed connection. As can be observed in this figure, the load ratio, L/L_n has a more pronounced effect on the nails than on the plywood or lumber components. This may be a direct consequence of the significantly lower levels of variability (COV) typical of nail yield capacities (see Tables II, III and IV). In this case, the variability of the load variable in Eq. 5 will have a more important effect.

Table X shows that the live/dead load ratio has a negligible effect on the reliability of the plywood and lumber components. However, it has a small but significant effect on the reliability of the nails. The reliability of the nails is more sensitive to the variability of the live load and when the live/dead load ratio increases the reliability index of the nails decrease.

SYSTEM RELIABILITY

Systems without redundancy, in which the failure of one or more components result in the failure of the system, are known as series systems or weakest link systems. The tension splices considered in this investigation are series systems and their survival depends on the survival of all components.

The series system consists of six components:

- Two pieces of lumber. Each piece must resist the full load acting on the splice. Their strengths are assumed to be statistically independent of each other.
- Two pieces of plywood. Each piece must resist half the load acting on the splice. To evaluate reliability bounds their strengths were first assumed to be statistically independent of each other and then fully correlated.
- Two groups of nails. Each group must resist the full load acting on the splice. Their strengths are assumed to be independent of each other.

Unimodal bounds

The reliability index for each component has already been calculated in the previous sections, for a number of cases. The corresponding probability of failure for each component can be calculated using Eq. 4.

It is possible to establish upper and lower bounds on the probability of failure of a series system, P_F, with k components, in terms of the individual failure probabilities, p_{FI}. Ang and Tang (1984) have derived the following relationship:

$$\max p_{FI} \leqq p_F \leqq -\prod_{i=1}^{k} (1-p_{FI}) \qquad (6)$$

The data from Fig. 12 have been used to calculate individual failure probabilities using Eq. 4 and substituted into Eq. 6 to calculate unimodal system failure probabilities which were

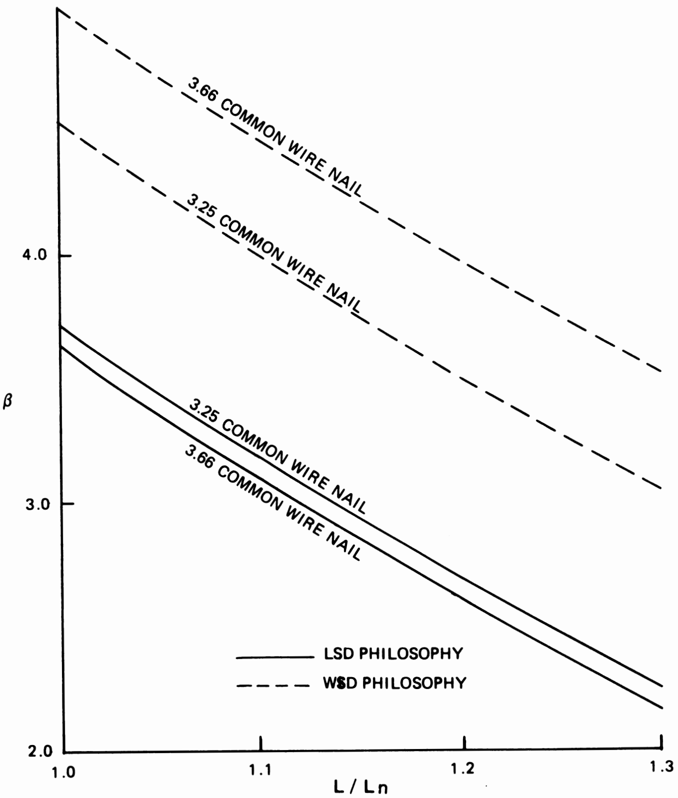

Figure 10. Component reliability. Nails. Comparison of two design philosophies. Plywood: D. fir. Lumber: D. fir. Low human occupancy. $L_n/D_n = 10$.

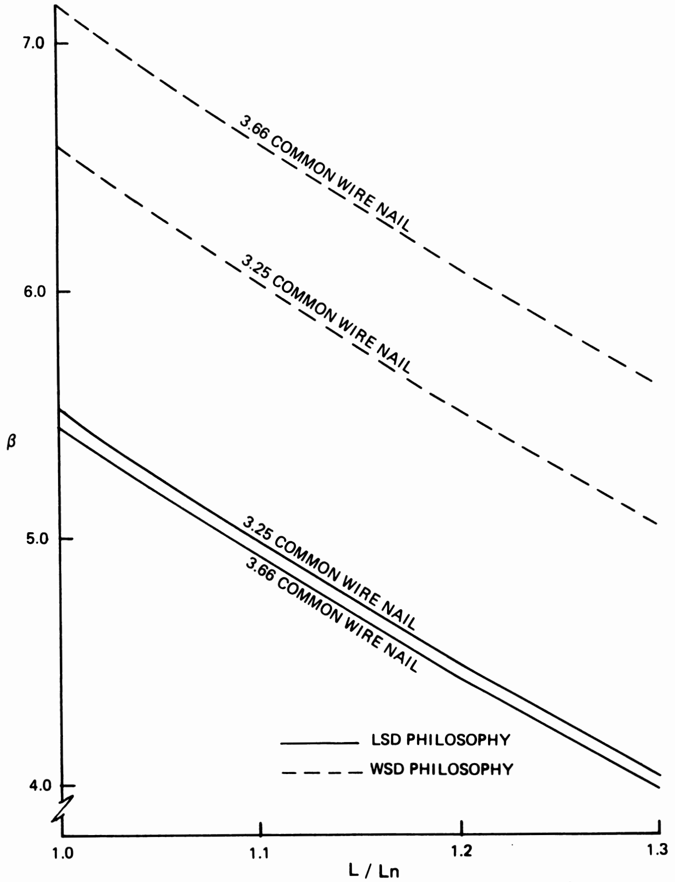

Figure 11. Component reliability. Nails. Comparison of two design philosophies. Plywood: D. fir. Lumber: S-P-F. Low human occupancy. $L_n/D_n = 10$.

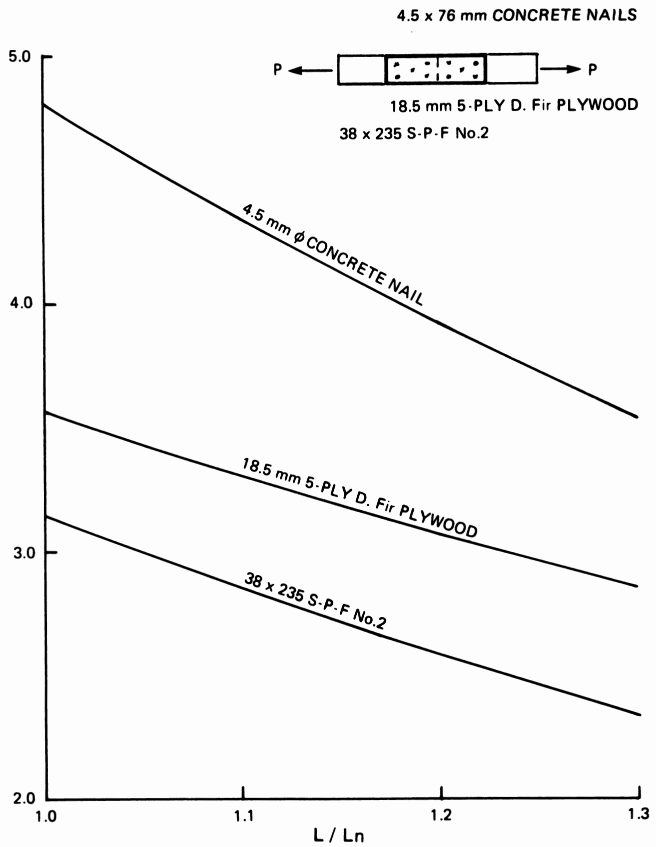

Figure 12. Comparison of component reliability. Low human occupancy. Limit states design. L_n/D_n = 10.

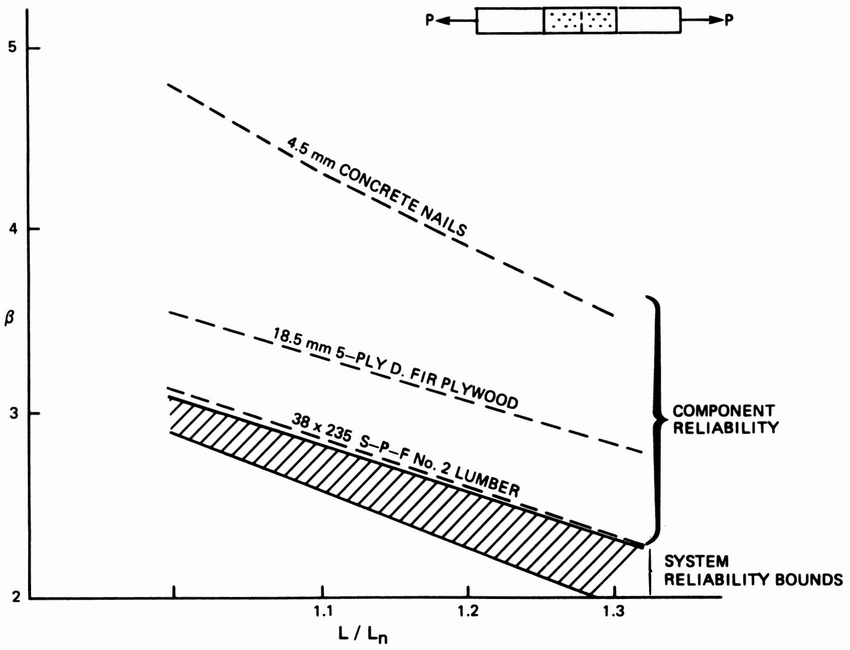

Figure 13. Comparison of component reliability and system reliability bounds. Low human occupancy. Limit states design. L_n/D_n = 10.

then converted back to reliability index, β, and plotted in Fig. 13.

As can be observed in Fig. 13, the high probability of failure (low reliability index, β) associated with the lumber components dominates the failure mode of the splice. The system reliability bounds define an area which is above the target reliability index of 2.5 for most regions across Canada where the lifetime maximum to nominal snow load ratio is below 1.2. Under conditions of perfect correlation between the strength of the plywood gussets, the unimodal system reliability bounds are still dominated by the lumber components and only a negligible change in the lower bound was detected. It is also clear that efforts aimed at improving the overall reliability of the system should be focused on increasing the individual reliability of the lumber which is the dominant component. This can be accomplished by using species, grades or sizes with higher strength and/or lower variability. Not suprisingly, this situation points towards the use of Machine Stress Rated lumber, which incorporates both traits.

Further improvement on these bounds is currently being investigated by considering bi-modal bounds obtained by evaluating the joint occurrence of failure in two components at a time and will be reported in future publications.

CONCLUSIONS

The methodology proposed takes into consideration the uncertainties associated with the loads as well as the variability found in the material properties. The proposed approach can be successfully used to assess the reliability of nailed connections by incorporating available information on strength and variability of the materials and the loads. This approach can also be used to assess the safety of a wide variety of currently used lumber connections and to evaluate new provisions or specifications being considered for adoption into the design standards.

A structured approach to the problem of determining failure probabilities also permits identifying those components which dominate the reliability of the nailed lumber/plywood connections. Upgrading these components would be the most efficient way to improve the safety of the connection.

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