

# Timber Rivet Connections – Design Process for a Hanger Connection

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## Introduction

Timber rivet connections have been used successfully in many structures over the past 30 years (Fig. 1). They are part of the U.S. (1) and Canadian (2) structural wood design codes, but unfortunately, there are few published design examples to aid designers (3-6). This paper is targeted to the connection designer and provides a short summary of the 2005 National Design Specification® (NDS®) for Wood Construction design process for timber rivet connections along with comments on design issues of common interest.

## Connection Strength

The main advantage of using timber rivets lies in their ability to develop very high loads compared to other conventional connectors in timber. As of this writing, the 2005 NDS limits the installation of timber rivets only into Douglas-fir-larch or southern pine structural glued laminated timber (manufactured in accordance with ANSI/AITC A190.1), whereas CAN/CSA-O86-01 permits rivet installation into other common structural wood products through the use of a listed factor,  $H$ , (10.7.2.2) applied to the factored lateral strength resistance of a connection loaded either parallel or perpendicular to grain (Table 1).

Test data (7-9) and rationale used for developing the CSA O86 provisions for timber rivets installed in solid sawn mate-

rial showed that timber rivet capacity for Douglas-fir-larch glulam and Douglas-fir-larch solid sawn lumber were comparable. The test failure modes were rivet yielding, but concern had been expressed that wood checks and splits are more prevalent in solid sawn material than glulam. So, although the test results were comparable, the  $H$  factor for solid sawn stated in the code is 50 percent of the  $H$  factor for glulam to allow for checks and splits in solid sawn.

Designers often ask: does it matter in which wood surface the timber connection is installed? From the research (3,7-12), there does not appear to be much difference in lateral load design capacity between face surface installations (surfaces 1 and 2 in Fig. 2) in parallel and perpendicular to the grain directions for timber rivet connections. Differences between surface 1 and 2 capacities are small in rivet yielding, but maybe more so for wood failure modes between each surface particularly in the perpendicular to grain direction. For design, these differences are small enough that installations in surfaces 1 and 2 can be treated the same. End grain (surface 3) installation design capacity, however, is considerably lower per NDS 13.2.5 and O86 10.7.2.7, roughly half of the perpendicular-to-grain surface 1 or 2 value for a 90° cut.

Timber rivet connections, originally developed for use with glulam construction, may be a viable option for use with structural composite lumber (SCL) products. Tests (13) have been conducted on small samples to assess the performance and predictability of timber rivet connections in parallel strand lumber (PSL) and laminated strand lumber (LSL). The test joint configurations were designed to ex-



Figure 1.—Typical timber rivet connection.

Table 1.—CAN/CSA-O86-01 (10.7.2.2) H Factor.

Douglas-Fir-Larch Glulam	1.0
Spruce-Lodgepole Pine – Jack Pine Glulam	0.80
Douglas-Fir-Larch Sawn Timber	0.50
Hem-Fir Sawn Timber	0.45
Spruce-Pine-Fir Sawn Timber	0.40
Northern Species Sawn Timber	0.35

hibit rivet failures in some combination of rivet yield and bearing deformation in the composite as opposed to brittle wood failure modes, such as block-shear tear-out or splitting. Results suggest that per-rivet design values should fall between 225 to 450 lb. (1 and 2 kN), depending on species and density of the composite and load direction with respect to grain of the composite strands. Timber rivets performed better in LSL than in PSL and better in yellow poplar PSL than in Douglas-fir or southern pine PSL; 1-1/2 in. (40 mm) rivets in yellow poplar LSL gave roughly equivalent performance to 2-1/2 in. (65 mm) rivets in yellow poplar PSL. Comparing rivet yield predictions following the *NDS* recommendations for round nails and the much simpler rule-of-thumb approach of using two-thirds the maximum load suggests that the latter approach provides a more consistently reliable evaluation of yield strength for timber rivets. Rivets in SCL are not currently addressed by the design codes. Additional study is still necessary to assess rivet connection performance in SCL when rivet density exceeds 1 rivet per square inch.

### Moisture Issues

Designers are often concerned about cross-grain movement of wood due to moisture changes, especially with respect to timber connections using wide side plates, or where fasteners are designed as large groups on a common plate. There is a belief that timber rivets on wide plates can accommodate wood cross-grain shrinkage/expansion due to moisture content (MC) change without inducing tension perpendicular stresses in the wood fiber around the rivet shanks; however, it is still a matter of the degree of MC change in the wood and the scale of the rivets, plate(s), and cross-grain dimension of wood member relative to each other. Standard connection practice is to never use a single steel splice plate on connections where the distance between outer rows of dowel connectors is greater than 5 in. (*NDS* Figure 11H). *CSA O86* Clause 10.7.1.10 states, “For wet fabrication conditions in sawn lumber, the maximum dimension perpendicular to grain over which a rivet group spans shall not exceed 200 mm (8 in.)” This accounts for shrinkage in the long term to

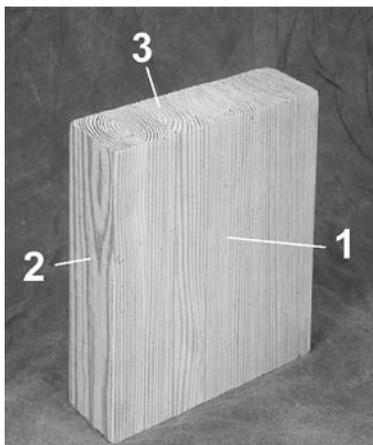


Figure 2.—Glulam surfaces.

prevent splitting, i.e., the timber shrinks but the steel plate does not, and tension perpendicular to grain develops which may result in splitting.

Timber rivets, on the other hand (as opposed to dowel-type connectors), have been tested and modeled for the group sizes indicated in the code tables. These tables in the *NDS* and *O86* limit the size of the rivet groups and were developed using analytical models confirmed with testing for groups within that range (see references). In terms of addressing the shrinkage/splitting concern with the drying of wet lumber, there were tests (5) that looked at various environmental conditions. Glulam, however, is made from kiln-dried material and should be much more stable than solid sawn, non-kiln-dried material. It would, therefore, be prudent, as a connection designer to keep this in mind when designing large groups of rivets, i.e., the material being specified, the anticipated service conditions, and the distance between the outermost rows of rivets on a common plate. Introducing more plates separated across the wood grain could alleviate this concern. In practice, however, the authors have never run into this issue as a problem.

Good practice is to always hot-dip galvanize metal components for corrosive or exposed environments and in situations where the structure may be exposed to the elements for long construction periods that might otherwise result in streaking stains on the wood that can be very difficult to remove (unsightly if the final structure is meant to be exposed for aesthetics). The *2005 NDS* specifies timber rivets made of mild steel (AISI 1035) and plates of A36 steel. Further, design provisions and values of the *2005 NDS* are applicable only to timber rivets that are hot-dip galvanized to ASTM A153. Plates also need to be hot-dip galvanized to ASTM A153 if the connection is in wet service. This is all described in *2005 NDS* 13.1.1.

### Design Process

The design process for timber rivets is illustrated through a worked example of a beam hanger connection using allowable stress design (ASD) and load resistance factor design (LRFD).

Essentially, there are four strength limit states to a timber rivet connection (Fig. 3); two parallel-to-grain (P-direc-

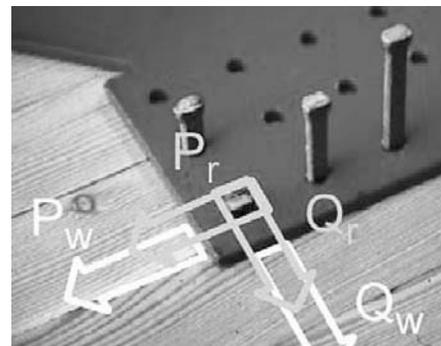


Figure 3.—Four strength limit states for a typical timber rivet connection.

tion) and two perpendicular-to-grain (Q-direction). For each grain direction either the rivet yields and controls or the wood fiber controls. If the load is applied only in the P-direction, or only in the Q-direction, then the number of strength limit states to check reduces to two: rivet yielding and wood fiber control. The lower strength will govern the design. The perforated plate is stiff and, although rarely an issue, should also be checked using appropriate steel code provisions (14,15). Although as mentioned previously, this potential limit state in the plate's tensile capacity could be added to the above states should one wish to assure connection ductility.

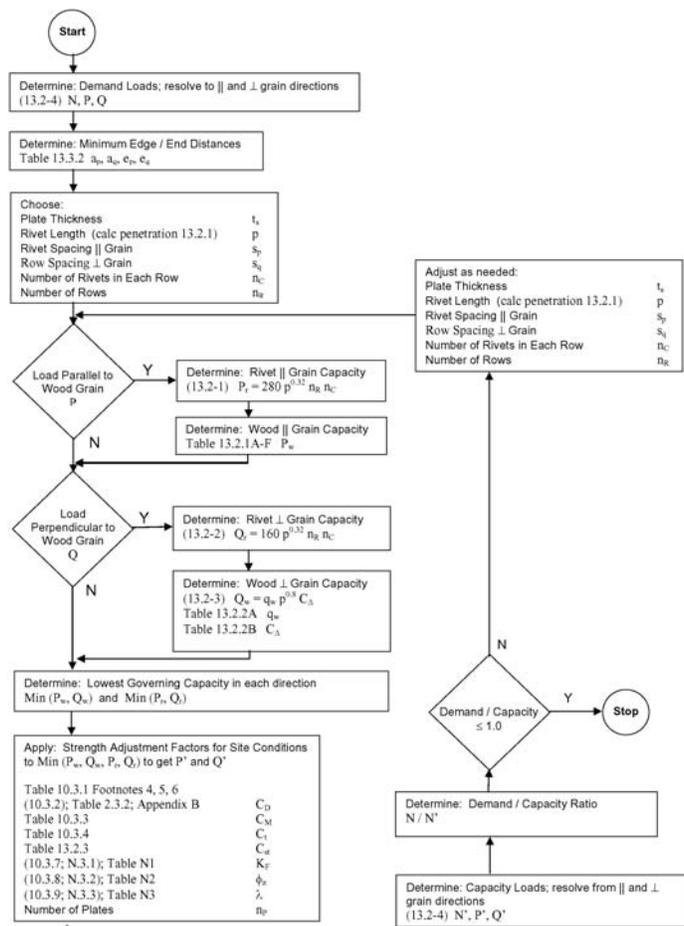
The design process is simple, regardless whether ASD or LRFD is used, and is best implemented using a spreadsheet, or other calculation software because of its recursive nature:

- Determine total loads that must be resisted (demand)
- Assume a trial design based on connection configuration geometry that will accommodate a grid of rivets, considering tabulated minimum edge and end distances. The main variables here are: plate thickness, rivet length, rivet spacing parallel-to-grain, number of rows of rivets, and number of rivets in each row.
- Check rivet yield failure – an equation is given for this based on the capacity of a single rivet through a single plate. There are two equations: one each for the P and Q directions respectively (NDS 13.2-1 and 13.2-2).
- Check wood failure parallel-to-grain (P-direction) – from a table based on rivets installed on faces of the connection. The tables are organized by rivet length and by plate thickness for typical rivet grid spacings. Footnotes to the table offer explanations of member width. The tables simplify the design process and allow the designer to avoid using complex equations for predicting wood failure in shear or tension. The equations were originally developed and verified by tests (7). For details on these equations, see the *2005 NDS Commentary*.
- NDS Tables 13.2.1A – 13.2.1F are for connections with steel side plates on opposite sides of the wood member. The reference design value in the table is for the capacity of *one* 1/4 in. side plate with associated rivets (NDS C13.2.1). Thus for a connection with plates on opposing faces, the designer would *double the table value* to determine the reference capacity of the connection. For connections with a single plate of rivets on one side of the wood member, the designer enters the table with *twice* the thickness of the wood member to get the correct reference capacity for a single-sided connection.
- Check wood failure perpendicular-to-grain (Q-direction) – an equation (NDS 13.2-3) is given for this based on the capacity of a single rivet through a single plate. The equation references two ta-

bles: one for the reference value (NDS Table 13.2.2A) based on one plate with rivets installed in one face of the connection and another for the Geometry Factor,  $C_{\Delta}$ , (NDS Table 13.2.2B). Again, the reference design value obtained from the equation is *doubled* for connections having *two* side plates.

- The lowest capacity of four failure checks will govern the capacity of the connection. If rivet yield governs, then ductility of the connection is assured. If wood capacity controls, the connection is likely to be less ductile.
- Adjust the determined capacity for site environmental conditions using adjustment factors.
- Calculate the demand:capacity ratio – a value less than 1.0 is safe. If the ratio is greater than 1.0, try adding more rivets and repeat the trial design. Off the table for number of rivets? Try increasing the rivet spacing parallel-to-grain and move to another table. No good? Try increasing the plate thickness. Still not enough? Try increasing the rivet length in increments to the maximum penetration permitted by the connection geometry, and repeat the trial.

A flowchart in **Figure 4** illustrates the design process as referenced in the *2005 NDS*.



**Figure 4.**—Flowchart of the design process as referenced in the *2005 NDS*. (A larger version of the flowchart is available in the online version at [www.awc.org](http://www.awc.org).) **[Appended to this document.]**

## Design Example – Hanger Connection

To illustrate the design process, an example of a typical beam hanger connection is provided. The example is worked in both ASD and LRFD and is based on the 2005 NDS timber rivet provisions found in Chapter 13.

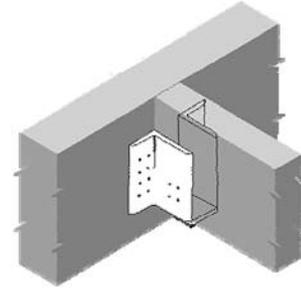
For this example, as shown in **Figure 5**, consider a beam-to-girder hanger connection with the hanger installed with rivets to one wood face of the girder and loaded in the girder perpendicular-to-grain direction. Here, two strength limit states are of interest: rivet strength and perpendicular-to-grain wood strength. The hanger is assumed to be structurally adequate. Notation in the example is defined in **Figure 4**.

### Summary

Timber rivets are a versatile means of making large scale timber connections functionally and aesthetically possible. Design of timber rivet connections using current code references is presented. An example is provided to illustrate the process and to aid the designer in this task.

### Disclaimer

It is intended that this paper be used in conjunction with competent engineering design. The authors, AF&PA, and



**Figure 5.**—Timber rivet hanger connection.

Equilibrium Engineering, Inc. assume no responsibility for errors and/or omissions in this paper, or for any engineering designs, plans, or construction prepared from it.

### References

References are available from the authors upon request.

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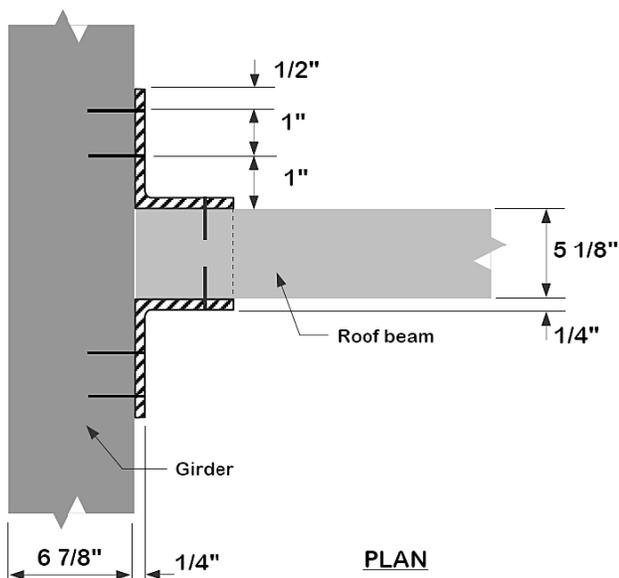
## Example: Beam Hanger Using Timber Rivets – ASD & LRFD

Design the beam hanger connection shown using timber rivets. The glulam beam and girder are untreated Douglas-fir glulam. The factored beam reaction is due to dead plus snow loads, and service conditions are wet. Use 2-1/2 in. rivets and 1/4 in. plate, and 2005 NDS provisions. Note that the designer must also confirm the bearing area for the roof beam support and consider possible uplift conditions in the design of this connection. See **Figure E1** for a plan view of the hanger.

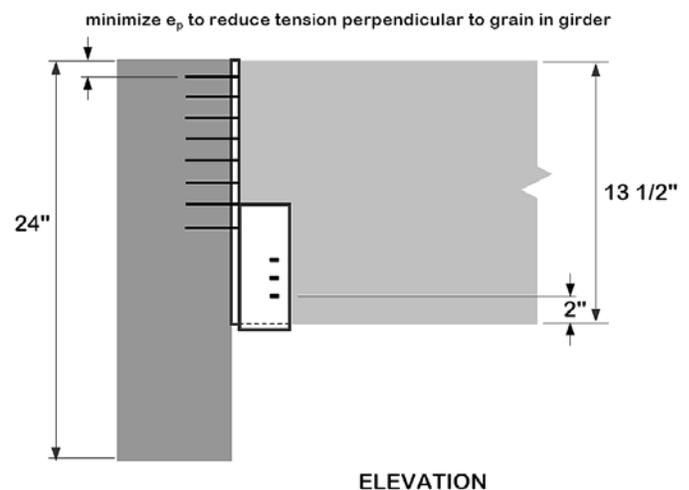
To reduce tension perpendicular stresses from connectors placed in the middle of the side of the girder, just as for larger fasteners, it is good practice to “hang” the load from the top of the girder.

This has additional benefits in terms of the number of rivets required, and ductility (see **Fig. E2**).

Note: This design solution has been developed using Mathcad® software by Parametric Technology Corporation® (PTC®). Therefore, formatting of certain variables and equations as shown in this example are unique to this software.



**Figure E1**



**Figure E2**

**Wood:**

$b_{\text{beam}} := 5.125 \text{ in.}$   
 $d_{\text{beam}} := 13.5 \text{ in.}$

**Rivet/Plate:**

$l_r := 2.5 \text{ in.}$   
 $t_p := 0.25 \text{ in.}$   
 $n_p := 2$  (treat hanger face as two plates, separated by carried beam)

**Modification Factors (NDS Table 10.3.1):**

$C_D := 1.15$  Load duration/Time effect  $\lambda := 0.8$   
 (ASD - NDS 10.3.2, LRFD - NDS Table N3; Snow Load)  
 $\phi_z := 0.65$  (NDS Table N2)  
 $K_F := \frac{2.16}{\phi_z} K_F = 3.323$  (NDS Table N1)  
 $C_M := 0.8$  Wet service (NDS Table 10.3.3)  
 $C_t := 1.0$  Less than 100°F (NDS Table 10.3.4)  
 $C_{st} := 1.0$  0.25 in. steel plate (NDS Table 13.2.3)

**Demand:**

	<b>ASD</b>	<b>LRFD</b>
D := 1500 lbf		
S := 4495 lbf		
	$R := D + S$	$R_f := 1.2D + 1.6S$
	$R = 5995 \text{ lbf}$	$R_f = 8992 \text{ lbf}$

**Capacity:**

Assume two rows in each plate, each side of hanger:

$s_p := 1 \text{ in.}$   
 $n_R := 2$

*Try 10 rivets each row, spaced at 1 in. perpendicular to grain.*

$s_q := 1 \text{ in.}$   
 $n_c := 10$

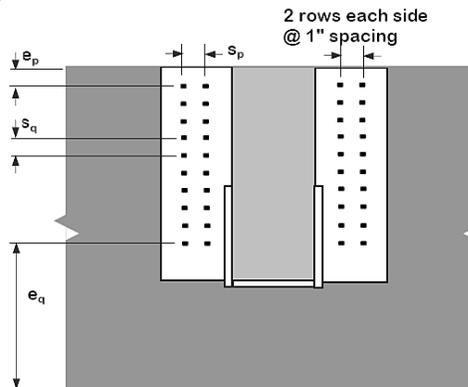
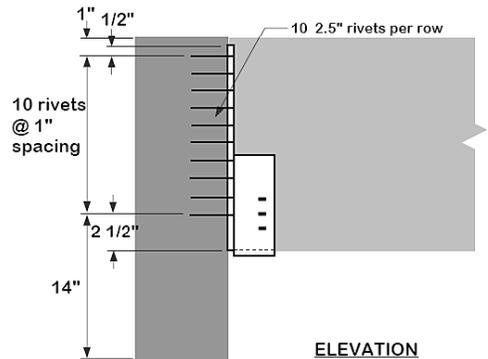
**Wood:**

From NDS Table 13.2.2A (load on wood is perpendicular to grain):

$q_w := 1173 \text{ lbf}$

Locate connection and check end and edge distances from NDS Table 13.3.2

$n_R = 2$      $a_p := 3 \text{ in.}$     End distance parallel to grain  
 $a_q := 2 \text{ in.}$     End distance perp to grain  
 $e_p := 1 \text{ in.}$     Rivet-to-unloaded-wood edge distance



**Figure E3**

$e_{ps} := 0.5 \text{ in.}$  Rivet-to-steel edge distance (Appendix M)  
 $e_q := 2 \text{ in.}$  Rivet-to-loaded-wood edge distance

$\frac{e_p}{(n_c - 1) \cdot s_q} = 0.111$

$C_{\Delta} := 5.48$  (NDS Table 13.2.2B interpolated)

penetration = rivet length – plate thickness – 1/8 in.

$p := \frac{(l_r - t_p - 0.125 \text{ in.})}{\text{in.}}$      $p = 2.125$

$Q_w := q_w \cdot p^{0.8} \cdot C_{\Delta}$      $Q_w = 11748.05 \text{ lbf}$     2005 NDS (13.2-3)

**Rivet (from NDS Equation 13.2-2):**

$Q_r := 160 \cdot p^{0.32} \cdot n_R \cdot n_c \cdot \text{lb}$      $Q_r = 4072.91 \text{ lbf}$     2005 NDS (13.2-2)

Take the minimum of wood or rivet capacity:

$Q := \min(Q_w, Q_r)$      $Q = 4072.91 \text{ lbf}$     Rivet controls

Adjust to conditions with rivet capacity governing (NDS Table 10.3.1, Footnotes 4 and 5). Note that  $C_D$  drops out of the ASD capacity when rivet yield controls (Footnote 4), yet  $\lambda$  remains on the LRFD side. For LRFD, the time effect factor,  $\lambda$ , applies to  $P_r$  and  $Q_r$  since the format conversion factor,  $K_B$ , for connections adjusts from a 10-year to a 10-minute load basis.  $C_D$  does not apply for ASD values of  $P_r$  and  $Q_r$  (Footnote 4) because “rivet bending capacity” was treated as a steel limit state in early research and implementation. The early assumption was that rivet bending capacity is unaffected by load duration. Load duration effects were specifically considered in checks of wood strength limit states, not steel strength limit states.

$n_p = 2$  (number of plates)

$Q' := Q \cdot n_p \cdot C_M \cdot C_t$

<b>ASD</b>	<b>LRFD</b>
$Q' = 6516.66 \text{ lbf}$	$\lambda \cdot \phi_z \cdot K_F \cdot Q' = 11260.78 \text{ lbf}$

**Demand – Capacity Ratios**

$\frac{R}{Q'} = 0.92$      $\frac{R_f}{(\lambda \cdot \phi_z \cdot K_F \cdot Q')} = 0.79 < 0.1 \text{ OK}$

**Solution – see Figure E3.**

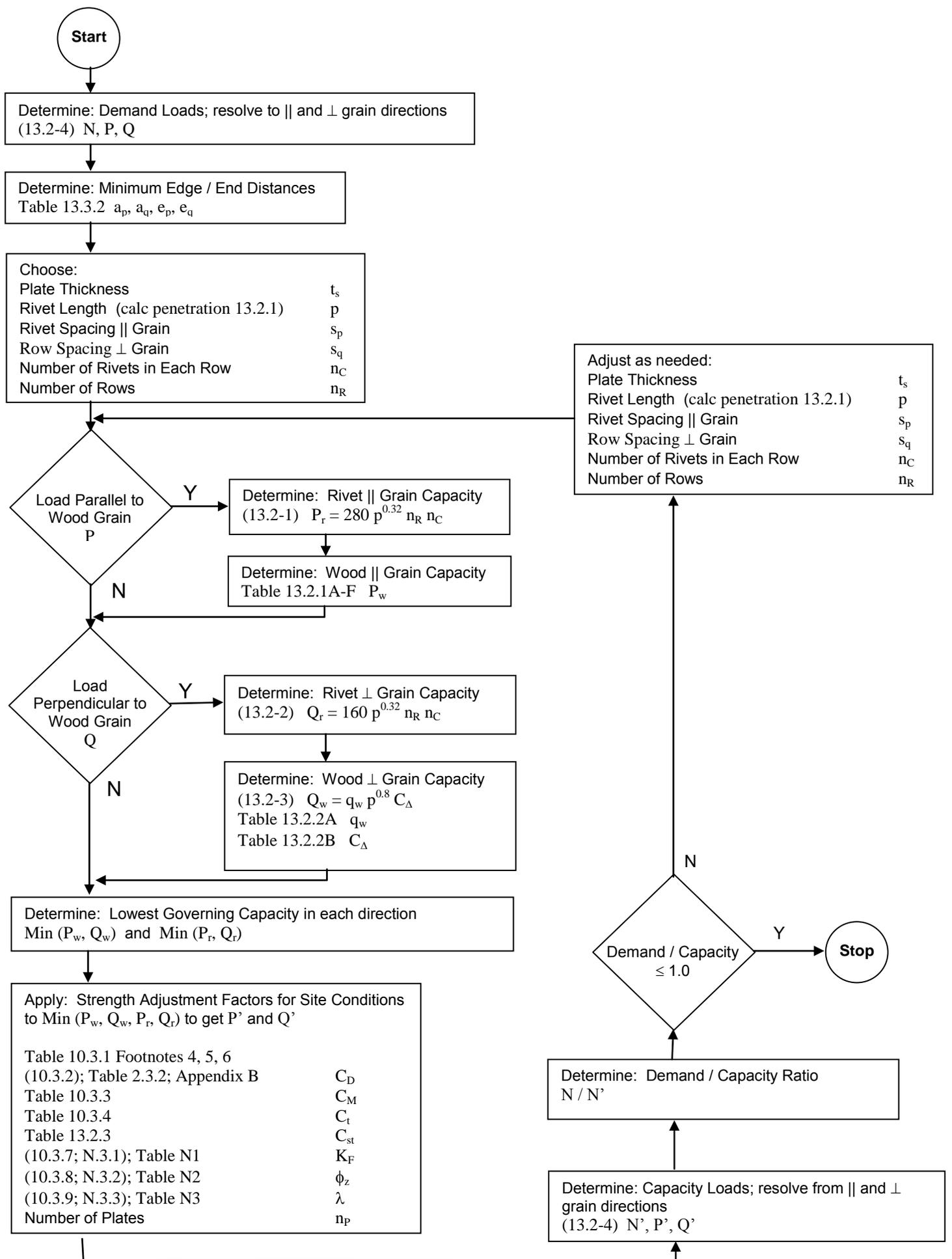


Figure 4: Timber Rivet Design Process Flowchart (referenced to 2005 NDS)