DEVELOPMENT OF STRENGTH AND STIFFNESS DESIGN VALUES FOR STEEL-CLAD, WOOD-FRAMED DIAPHRAGMS

By

DAVID AGUILERA

A thesis submitted in partial fulfillment of
The requirements for the degree of

MASTER OF SCIENCE IN CIVIL ENGINEERING

WASHINGTON STATE UNIVERSITY
Department of Civil and Environmental Engineering

August 2014

To the Faculty of Washington State University:

The members of the Committee appointed to examine the thesis of DAVID AGUILERA find it satisfactory and recommend that it be accepted.

Donald A. Bender, Ph.D., Chair

James D. Dolan, Ph.D.

Vikram Yadama, Ph.D.

AKNOWLEDGEMENT

I would like to express my deepest appreciation to my committee chair, Dr. Don Bender, for his guidance, mentorship and patience throughout this research project. I would like to thank my committee members, Dr. Dan Dolan and Dr. Vikram Yadama for their support and agreement in being a part of my committee. I would like to express a special thanks to Dr. Gary Anderson from South Dakota State University for generously providing literature and test reports needed to complete this project. I cannot thank him enough for investing his time and sharing his expert knowledge on this topic.

DEVELOPMENT OF STRENGTH AND STIFFNESS DESIGN VALUES FOR STEEL-CLAD, WOOD-FRAMED DIAPHRAGMS

ABSTRACT

By David Aguilera, M.S.
Washington State University
August 2014

Chair: Donald A. Bender

Lateral loads are primarily resisted by roof diaphragms and shear walls in post-frame buildings. While a variety of sheathing materials can be used, most often corrugated steel sheets are fastened to the wood frame to form shear walls and diaphragms. The strength and stiffness of these components must be known for proper analysis and design of the building, and are typically derived through small-scale panel tests. This method is costly and may not accurately estimate the strength of full-scale diaphragm assemblies. A mathematical model, typically referred to as the *modified MCA procedure*, allows analytical predictions of diaphragm strength and stiffness for steel-clad, wood-framed (SCWF) diaphragms. This model is the most accurate procedure to-date for predictions of design values, but the time investment to learn the model and complexity are significant barriers to implementation.

The objective of this research project is to provide an independent validation of the modified MCA procedure, and implement the model to develop a full matrix of shear strength and stiffness design values for common constructions of SCWF diaphragms. Predictions from the modified MCA procedure are compared to design unit shear strength and effective shear modulus values obtained from seven different SCWF diaphragm tests and nine different SCWF shear wall tests. For diaphragms the ratio of predicted to tested design unit shear strength averaged 0.97 and the ratio of predicted to tested effective shear modulus averaged 1.07. For shear walls the ratio of predicted to tested design unit shear strength averaged 0.81 and the ratio of predicted to tested effective shear modulus also averaged 0.81. The modified MCA procedure was used to develop design values for 168 different diaphragm constructions allowing a variety of corrugated steel profiles to be used. Adjustment formulas are provided to more accurately predict the strength of full-scale diaphragms. The design tables and formulas provide a simplified and useful resource for structural engineers to design safer and more economical post-frame buildings.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER 1

INTRODUCTION TO LATERAL DESIGN OF POST-FRAME BUILDINGS

INTRODUCTION

Post-frame buildings typically employ a wood-frame system with corrugated steel sheathing to form shear walls and diaphragms. Gravity loads are carried to the foundation using timber posts spaced several feet apart and are typically either embedded in the ground with concrete footings or surface mounted to a concrete foundation. Girts span across posts to form the wall frame. Trusses are attached directly to posts and purlins span across the trusses to form the roof frame. Various cladding materials can be used, but the most common is corrugated steel attached to framing members on the exterior walls and roof with structural fasteners. Figure 1.1 illustrates the various components in a post-frame building.

While corrugated steel cladding is used as an exterior finish, it also serves as the main lateral force resisting system through diaphragm action. Timber frames with embedded posts also aid in resisting lateral loads, but only carry a small portion because the diaphragm is significantly stiffer, attracting the large majority of lateral loads.

CURRENT DESIGN PROCEDURES

ANSI/ASAE EP484.2 (2012) is the engineering standard for diaphragm design of post-frame buildings. Distribution of lateral loads to frames, diaphragms, and shear walls are typically considered when frames are assumed fixed at the base. This requires the strength and stiffness for each building component to be known. Adjustment factors have been tabulated in load distribution tables to determine the maximum forces in the diaphragm and critical frame. The most highly stressed frame is usually at mid-length of the building where maximum eave deflections occur. Load distribution tables are only applicable when the building does not have intermediate shear walls and frame stiffness, shear wall stiffness, diaphragm stiffness, and eave loads are each constant values. When these values are not constant or intermediate shear walls are present, the force distribution method (Anderson et al, 1989) or computer program DAFI (Bohnhoff, 1992) can be used to determine load distribution between building components. Frame stiffnesses can be easily computed using statics or a structural analysis program; however, shear wall and diaphragm stiffnesses are typically estimated from small-scale panel tests in accordance with ASAE EP558 (2014).

METHODS FOR DETERMINING DIAPHRAGM STRENGTH AND STIFFNESS

Steel-clad, wood-framed (SCWF) diaphragms and shear walls have a number of components that influence strength and stiffness. These parameters include the number of fasteners, fastener location, size of fasteners, framing member properties, framing geometry, steel cladding properties, and the location and properties of blocking. Diaphragm behavior can be complex, therefore ANSI/ASAE EP484.2 requires shear strength and stiffness values be obtained from full-scale building tests, validated structural models, or small-scale panel tests.

The most common method for estimating diaphragm design values is small-scale panel tests conducted in accordance with ASAE EP558 using either a cantilever or simple beam configuration. The design values determined from small-scale testing can then be extrapolated for the full-scale shear wall or diaphragm used in the post-frame building being designed. It is assumed that shear strength and stiffness are a linear function of diaphragm length. If the structural engineer wishes to change a single parameter, an entirely new specimen configuration must be tested to determine new strength and stiffness values. This method for determining strength and stiffness is costly, time consuming, and limits flexibility in the substitution of materials and constructions of shear walls and diaphragms.

While strength and stiffness can be obtained from full-scale building tests, this is an impractical means of establishing design values, and only a handful of full-scale building tests have been conducted. Past attempts to develop structural models using finite element analyses also proved impractical for design purposes due to complexity and computational requirements. The most accurate model developed to-date is an analytical procedure originally developed by Luttrell and Mattingly (2004) for SCWF diaphragms and modified by Leflar (2007) and Anderson (2011) for use with SCWF diaphragms. This mathematical model is often referred to as the *modified MCA procedure*.

MODIFIED MCA PROCEDURE

A mathematical model for determining strength and stiffness of steel-clad, steel-frame (SCSF) diaphragms was been published by the Metal Construction Association (MCA) and is titled *A Primer on Diaphragm Design* (2004). Design procedures for SCWF diaphragms were provided in the design manual, but the stiffness was over predicted in the MCA validation. Several features unique to post-frame construction were not considered in the MCA procedure such as the stiffness of purlins and shear blocks that are usually incorporated into the diaphragm stiffness. Modifications to the procedure were made by Leflar and Anderson to include these elements in the computation of the effective shear modulus. Amendments were also made to screw flexibility formulas that are more accurate for connections used in SCWF diaphragms. In the testing of small-scale SCWF diaphragms screws are typically placed into the blocking on loaded rafters. The contribution of these addition screws was addressed in the strength prediction. In accordance with the modified MCA procedure, diaphragm strength is governed by panel buckling, fasteners at the corners of steel sheets, and fasteners used in the field.

In Leflar's validation, shear strength and stiffness values obtained from 26 different diaphragm constructions tested in accordance with ASAE EP558 were compared to those obtained by the modified MCA procedure. The strength prediction using the modified MCA procedure averaged 98% of tested strength with a 16% coefficient of variation. The calculated stiffness averaged 97% of the tested stiffness with a 23% coefficient of variation (Leflar, 2008).

RESEARCH OBJECTIVES

The modified MCA procedure is valuable tool that can be used to determine strength and stiffness of SCWF diaphragms with less reliance on expensive diaphragm tests. While the model provides a reasonably accurate means of analytically predicting design values for test panels, the model is complex and the required time investment to learn the model are significant barriers to implementation. As a means of simplifying the implementation, design tables and accompanying adjustment formulas can be developed for diaphragm constructions commonly used in the post-frame industry. The objectives of this study are to:

1) Provide an independent validation of the modified MCA procedure.
2) Determine possible differences between test panels and full-scale diaphragms.
3) Provide a simplified approach to analytically derive strength and stiffness for common SCWF diaphragm constructions using the modified MCA procedure.

Objective 1 is discussed in Chapter 2 and Objectives 2 and 3 are addressed in Chapter 3. The ultimate goal of this research is to provide a simple, flexible and robust approach to develop design values in lieu of small-scale diaphragm testing. Currently, the database for diaphragm design values is limited, and the addition of design tables will provide engineers with a variety of diaphragm constructions to meet a broad range of applications. This would likely result in more economic designs and greater market acceptance for post-frame buildings.

REFERENCES

Anderson, G. A. 2011. Modification of the MCA Procedure for Strength and Stiffness of Diaphragms used in Post-Frame Construction. Frame Building News (June): 22-25.

Anderson, G. A., D. S. Bundy, and N. F. Meador. 1989. The force distribution method: Procedure and application to the analysis of buildings with diaphragm action. Transactions of the ASAE 32(5):1791-1796.

ASAE. 2012. ANSI/ASAE Standard EP484.2. Diaphragm Design of Metal-Clad, Wood-Frame Rectangular Buildings. ASABE Standards, Engineering Practices, and Data. St. Joseph, Mich.: ASABE.

ASAE. 2014. ANSI/ASAE Standard EP558 Load Tests for Metal-Clad Wood-Frame Diaphragms. ASABE Standards, Engineering Practices, and Data, American Society of Agricultural and Biological Engineers, St. Joseph, MI.

Bohnhoff, D. R. 1992. Expanding diaphragm analysis for post-frame buildings. Applied Engineering in Agriculture 35(4):509-517

Leflar, J.A. 2008. A mathematical Model of Steel-Clad Wood-Frame Shear Diaphragms. Unpublished M.S. thesis, Department of Civil and Environmental Engineering, Colorado State University, Fort Collins, CO.

Luttrell, L.D. and J.A. Mattingly. 2004. A Primer on Diaphragm Design. Metal Construction Association, Glenview, IL

Post-Frame Building Design Manual, 1999. National Frame Builders Association, Lawrence, KS.

FIGURES

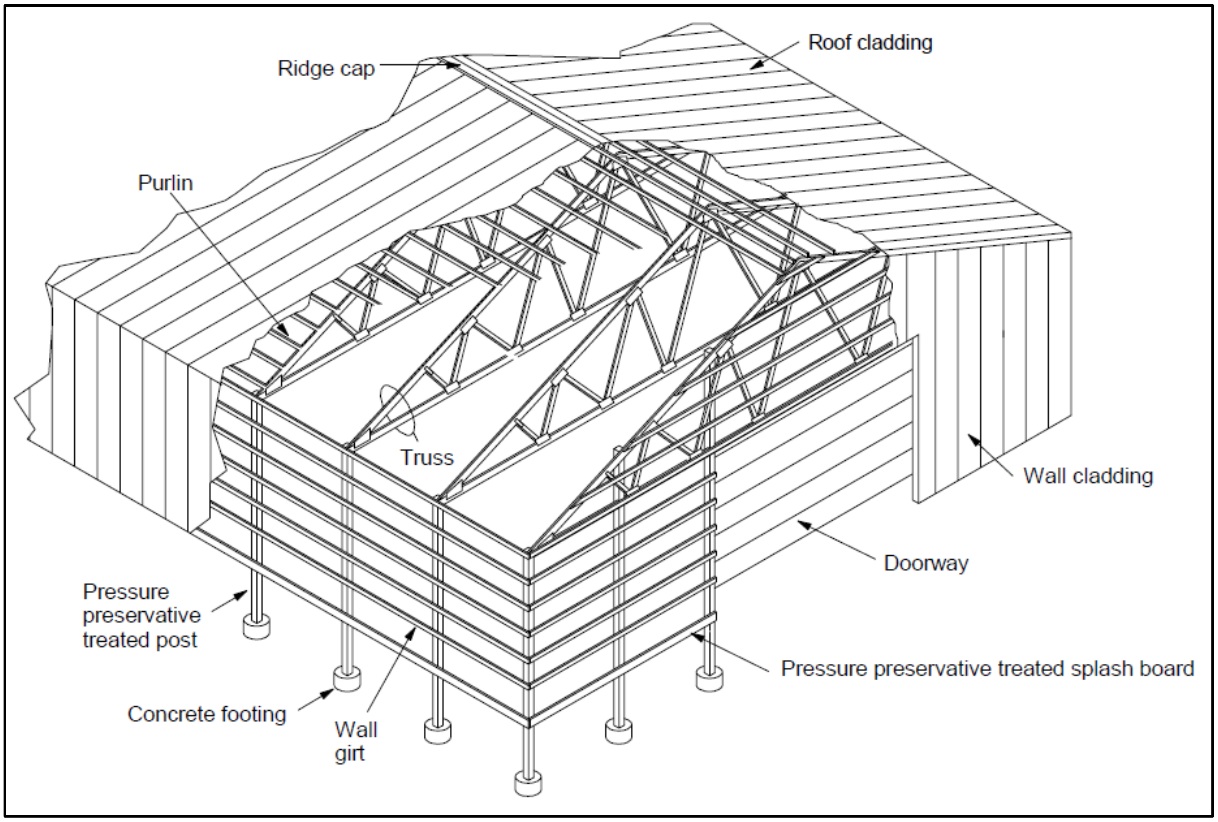


Figure 1.1: Components of a typical post-frame building (Post-Frame Building Design Manual, 1999)

CHAPTER 2

VALIDATION OF MODIFIED MCA PROCEDURE FOR PREDICTING STEEL-CLAD, WOOD-FRAME DIAPHRAGM AND SHEAR WALL STRENGTH AND STIFFNESS

ABSTRACT

A mathematical model for predicting shear strength and stiffness of steel-clad, steel-frame diaphragms (SCSF) published by the Metal Construction Association (MCA) was previously modified for applicability to steel-clad, wood-framed (SCWF) diaphragms. The model, referred to as the *Modified MCA Procedure*, allows design values to be predicted analytically with less reliance on expensive diaphragm testing. The modified MCA procedure was independently validated by comparing predicted unit shear strength and effective shear modulus values to those obtained from seven different SCWF diaphragm tests and nine different SCWF shear wall tests. An explanation of the model and how it was used in the validation is presented. Design values predicted by the modified MCA procedure for diaphragms were found to be in good agreement with tested values with conservative differences in most cases. The ratio of predicted to tested strength averaged 0.97, and the ratio of predicted to tested effective shear modulus averaged 1.07. Design values for shear walls were less accurately predicted by the modified MCA procedure, but were found to be conservative in most cases. For shear walls the ratio of predicted to tested design unit shear strength averaged 0.81, and the ratio of predicted to tested effective shear modulus also averaged 0.81.

INTRODUCTION

Post-frame construction is a viable option for the many low-rise building applications, offering a system that is durable, easy to construct, and generally less expensive than conventional light-frame wood construction. The post-frame industry was valued at $6.4 billion in 2011 and lumber represents $860 million of the material costs. Post-frame buildings, traditionally used in agricultural markets, have excellent potential for commercial and industrial markets due to the relative low cost and open, versatile spaces created by clear span trusses.

Light-gauge corrugated steel panels are typically fastened to the frame as an exterior finish, but also serve as the primary lateral resisting system for the building through diaphragm action. While the primary function of frames is to carry gravity loads, they also carry a small portion of lateral load. The strength and stiffness of diaphragms and shear wall assemblies must be known for proper analysis and lateral design of the building; however, a standardized design database is lacking.

Strength and stiffness for shear wall and diaphragm assemblies are commonly derived through small-scale tests conducted in accordance with ASAE EP558 (2014). Whenever a different combination of construction materials, framing geometry, and fastener pattern are used new tests are required to determine design values for the specific construction. Design values obtained from small-scale tests are not a practical option for building design, and are limited due to their high cost and the considerable time required to perform tests.

The Metal Construction Association (MCA) published a mathematical model, titled *A Primer on Diaphragm Design,* for the prediction of shear strength and stiffness of SCSF diaphragms. The model, developed by Luttrell and Mattingly (2004), is widely used in the steel building industry.

Leflar (2008) and Anderson (2011) modified the MCA procedure to predict shear strength and stiffness of SCWF diaphragms. The model was validated by Leflar and is suggested as a feasible alternative to small-scale diaphragm tests. The model is being considered for use in developing design tables for strength and stiffness of SCWF diaphragms; however, an independent validation of the model provides further credibility to the model.

The objective of this study is to independently validate the modified MCA procedure by comparing predicted values to seven different SCWF diaphragm tests and nine different SCWF shear wall tests.

LITERATURE REVIEW

In post-frame buildings, resistance to lateral loads is dependent on diaphragm action in roofs and shear walls. For the structural analysis and design, strength and stiffness of the diaphragm must be known. Small-scale diaphragm test panels have been the primary method of determining strength and stiffness of SCWF diaphragms. Test panels are representative of the exact diaphragm construction in the building being designed, and are typically 9 ft x 12 ft cantilever tests. Strength and stiffness of test panels are extrapolated for larger diaphragms by assuming both strength and stiffness increase linearly with length.

Another possible means of establishing strength and stiffness is through full-scale building tests. Full-scale building test data are limited, and due to cost are impractical for design purposes of a range of building configurations and constructions. Hurst and Mason (1961) conducted full-scale post-frame building tests on two 45 ft long by 36 ft wide buildings. The intent of these tests was to determine if the steel cladding contributed to lateral stiffness of the building. It was established from these tests that corrugated steel cladding used in post-frame buildings is effective in resisting lateral loads through diaphragm action. Johnston and Curtis (1984) conducted a full-scale building test to compare predicted building performance to design procedures developed by Hoagland and Bundy (1983). A 117 ft long by 45 ft wide full-scale building with an intermediate shear wall was tested. The effects of different fasteners and the use of knee bracing on lateral stiffness of the building were also investigated. McFadden (1991) conducted tests on a 45 ft long by 24 ft wide building to determine eave deflections when various parts of the building were sheathed. He also sought to compare eave deflections from the building with procedures developed by Hoagland and Bundy (1983) and modified by Gebremedhen et al. (1986).

Gebremedhen et al. (1992) also conducted tests on an 80 ft long by 40 ft wide building to determine eave deflections during various stages of construction. These full-scale building tests demonstrated that the corrugated steel cladding significantly increased building lateral stiffness. The most recent full-scale building tests were conducted by Bohnhoff et al. (2003) in which a 200 ft long by 40 ft building was tested with hundreds of different load cases. Frames were spaced 10 ft apart, and individual frames could be locked to prevent lateral displacement allowing simulation of shorter length buildings. The building was instrumented to measure frame loads, chord forces, horizontal eave displacements, and major and minor axis bending of purlins and girts.

Validated structural models may also be used in determining strength and stiffness of SCWF diaphragms. Prediction of design values using finite element analysis has been the subject of research in the past, but these models are not feasible for use in building design due to complexity and computational requirements. Wright (1993) modeled half of an 8ft. by 12ft. a SCWF diaphragm using finite element analysis software. The model contained a total of 11,644 nodes, 11,515 elements, and 69,864 degrees of freedom. The finite element model predicted the panel ultimate strength within 3% and shear stiffness to within 28%. Keener and Manbeck (1996) sought to simplify Wright's finite element analysis model with less elements and degrees of freedom. The model only accurately predicted the linear portion of the load-deformation curve. The model predicted shear stiffness to within 20%, and design shear strength was predicted to within 51%.

OVERVIEW OF MODIFIED MCA PROCEDURE

The modified MCA method utilizes mechanics based formulas along with empirically derived equations for screw strengths. Input variables, which are primarily related to geometric and material properties of the diaphragm, are first determined. Structural and seam screw strengths are then determined, as diaphragm capacity is dependent on screw capacities. The contribution of individual screws to shear strength of the diaphragm is dependent on the distance of the screw from the centerline of the panel. Once screw contributions have been determined the diaphragm can be evaluated for strength based on the limit states of field screws, corner screws, and panel buckling. Post-frame buildings are primarily designed using ASD design methods, thus an ASD factor of safety is applied to the governing limit-state.

The effective shear modulus is determined by first computing the in-plane stiffness of the cladding. The in-plane stiffness is dictated by deformations resulting from shear strain of the steel, warping of the major corrugations, and fastener slip. Formulas for screw stiffness are given and the contribution of each screw to fastener slip is determined by its distance from the centerline of the panel. Warping of major corrugations is determined through a series of formulas that account for the number of spans and number of screws next to major corrugations at panel ends. Purlin and blocking connections are then combined with the in-plane stiffness to compute an effective shear modulus of the entire diaphragm assembly. The general procedure is outline below in 10 steps.

Step 1: Determine input variables for strength

A number of input variables are required to use the model. Material properties for the corrugated panels, fasteners, and lumber must be known. Geometric properties of the cladding and diaphragm layout are also required along with the location of screws relative to centerline of the cladding. Figure 2.1 depicts the cladding geometry and Figure 2.2 shows a typical screw pattern with distances from the centerline labeled. These input values are listed below.

<u>Corrugated panel geometric and material properties:</u>

w = panel width, in.

p = major rib corrugation pitch, in.

d_d = height of major rib, in.

w_t = top width of major rib, in.

w_c = bottom width of major rib, in.

t = base metal thickness, in.

F_y = yield strength, ksi

F_u = ultimate strength, ksi

E = elastic modulus, ksi

I_x = moment of inertia, in^4/ft

n = number of corrugations per cladding width

$n = w/p$

w_b = corrugation flat width, in.

$w_b = p - w_c$

w_w = web width, in.

$$w_w = \sqrt{d_d^2 + \left[(w_c - w_t)/2\right]^2}$$

s = flat width to form one pitch, in.

$s = w_b + w_t + 2w_w + 1/8$

Diaphragm properties:

a = rafter spacing, ft

b = L = sloped roof length parallel to corrugations, ft

L_v = purlin spacing, ft

G = purlins specific gravity

n_{rsb} = number of shear blocks per rafter

n_{spans} = number of spans

$n_{spans} = L/L_v$

n_{ip} = number of interior purlins

$n_{ip} = n_{spans} - 1$

n_{shts} = number of cladding sheets between rafters

$n_{shts} = 12a/w$

Screw properties and locations relative to panel centerline:

x_{pa} = interior purlin field screw location relative to panel centerline, in.

x_{pb} = shear block screw location relative to panel centerline, in.

x_e = end purlin field screw location relative to panel centerline, in.

N_e = number of end purlin screws and elevated sidelap structural fasteners across the sheet width

n_{purlin} = number of elevated sidelap structural screws in one seam overlap

n_{stitch} = number of stitch screws in one seam overlap

n_{ssb} = number of screws per shear block

d_{fs} = diameter of field screw, in.

L_f = length of field screw, in.

d_{ss} = diameter of seam screw, in.

n_s = total stitch and elevated sidelap structural screws in one seam overlap

$n_s = n_{purlin} + n_{stitch}$

n_{pip} = number of phantom interior purlins per sheet between rafters

$n_{pip} = n_{rsb}\ n_{ssb}/n_{shts}$

N = number of screws per unit length at end of panel

$N = N_e/(w/12)$

Step 2: Determine the strength of field, stitch, and elevated sidelap structural screws

Field screw strength:

The strength determination is dependent upon fastener capacity and fastener location relative to the panel centerline. Structural fastener capacity is limited by wood crushing, or steel tearing. For wood failure the strength of the fastener, Q_{f_wood}, is determined by Equation 2.1. The screw must penetrate the purlin 8 *root* diameters or more for full strength. Since screws are usually specified by *crest* diameter, which is approximately 20% larger than the root diameter, the required penetration depth is 6.4 *crest* diameters. If the penetration depth is between 6.4d and 4d, the strength must be decreased linearly with the reduction factor, R. No strength is allotted for screws with penetration depth less than 4d.

$$Q_{f\_wood} = 32.0 d_{fs}^2 G \cdot R \tag{2.1}$$

where:

$R \quad = n_{dia} / 6.4 \leq 1.0$

$n_{dia} \quad = L_f / d_{fs}$ (must be ≥ 4)

The fastener strength limited by steel tearing, Q_{f_stl}, is determined by Equation 2.2 as determined from MCA research.

$$Q_{f\_stl} = \begin{cases} 2.22 d_{fs} F_u t & \text{for No. 8, 9, and 10 screws} \\ 1.25 F_y t (1.0 - 0.005 F_y)\sqrt{t/0.028} & \text{for No. 12 and 14 screws when } t < 0.028 \\ 1.25 F_y t (1.0 - 0.005 F_y) & \text{for No. 12 and 14 screws when } t \geq 0.028 \end{cases} \tag{2.2}$$

The structural screw strength, Q_f, is the minimum value determined from the two failure modes of wood crushing and steel tearing. That is,

Q_f = minimum of Q_{f_wood} and Q_{f_stl}

<u>Stitch screw strength:</u>

Stitch screws are used for steel-to-steel connections at the panel overlaps as shown in Figure 2.3. The strength of No. 8, 10, 12, and 14 screws is determined from Equation 2.3.

$$Q_s = \begin{cases} 115 d_{ss} t \sqrt{t/0.028} & \text{when } t < 0.028 \\ 155 d_{ss} t & \text{when } t \geq 0.028 \end{cases} \tag{2.3}$$

Strength for elevated sidelap structural fasteners anchored in wood:

Elevated sidelap fasteners penetrate the panel overlap and purlin below as shown in Figure 2.3. Anderson (2011) suggests the use of similar equations used for structural fasteners limited by steel tearing. These formulas are shown in Equation 2.4.

$$Q_s = \begin{cases} 2.22 d_{ss} F_u t & \text{for No. 9, and 10 screws} \\ 1.25 F_y t (1.0 - 0.005 F_y) & \text{for No. 12 and 14 screws} \\ 1.5 d_{ss} F_u t & \text{for 0.145 in. nail} \end{cases} \qquad (2.4)$$

While elevated sidelap structural fasteners are similar to stitch screws, the end of the fasteners penetrates the purlin preventing the fastener from tilting. This provides a stronger connection than stitch screws. Although the fastener is elevated, it bears on the steel in the same manner as typical structural fastener. This justifies the use of similar equations used for structural fasteners limited by steel tearing. A minimum penetration depth of 2 diameters is recommended for elevated sidelap structural fasteners (Anderson, 2011).

Step 3: Determine diaphragm factors and fastener contribution factor

Diaphragm factors account for the shear contribution of each fastener by considering the location of fasteners relative to the panel centerline. The contribution of each fastener is assumed to vary linearly from zero at the panel centerline to the fastener capacity at the panel edge. Derivation of the diaphragm factor equations are provided in Appendix A. The diaphragm factors for field screws, blocking screws, and end purlin screws are computed as shown below.

Determine diaphragm factors for field screws:

$$\alpha_{ap} = \Sigma x_{pa}/w$$

$\alpha_{ap}^2 \quad = \Sigma(x_{pa}/w)^2$

Determine diaphragm factors for screws into blocking:

$\alpha_{bp} \quad = x_{pb}/w$

$\alpha_{bp}^2 \quad = (x_{pb}/w)^2$

Determine weighted average for field and blocking screws:

$n_{pip} \quad = n_{rsb}n_{ssb}/n_{shts}$

$\alpha_p \quad = (n_{ip}\alpha_{ap} + n_{pip}\alpha_{bp})/(\, n_{ip} + n_{pip})$

$\alpha_p^2 \quad = (n_{ip}\alpha_{ap}^2 + n_{pip}\alpha_{bp}^2)/(\, n_{ip} + n_{pip})$

Determine diaphragm factors for end screws:

$\alpha_e \quad = \Sigma x_e/w$

$\alpha_e^2 \quad = \Sigma(x_e/w)^2$

Determine diaphragm factor for seam screws:

$\alpha_s \quad = Q_s/Q_f$

Determine fastener contribution factor:

$n_p \quad = n_{ip} + n_{pip}$

$B \quad = n_s\alpha_s + 2n_p\alpha_p^2 + 4\alpha_e^2$

Step 4: Calculate the nominal strength for the three limit-states

Limit State 1: Field fasteners

The diaphragm unit shear capacity governed by field fasteners is determined by Equation 2.5 which considers the contribution of all fasteners used in the field, seam screws, end purlin screws, and screw placed into blocking. The four corners of the panel are subject to forces acting perpendicular and parallel to corrugations, producing an eccentric resultant force. Two of the corners will be in compression, and the edge-most corrugation at those corners may not be able to resist the full eccentric compression force. Thus, a corner fastener strength reduction factor, λ, is conservatively applied to all four corners screws and shall not be less than 0.7.

$$S_{u\_f} = Q_f [B + 2(\lambda - 1)]/L \tag{2.5}$$

where:

$$\lambda = 1 - d_d L_v/(240\sqrt{t}) \geq 0.7$$

<u>Limit State 2: Corner fasteners:</u>

The shear force acting parallel to end purlins along the sheet edge is shared equally among the end purlin screws. The screws at the panel corners are the most heavily loaded experiencing shear forces both parallel and perpendicular to the end purlins. The diaphragm unit shear capacity governed by corner screws is computed using Equation 2.6

$$S_{u\_c} = Q_f \sqrt{N^2 B^2/(L^2 N^2 + B^2)} \tag{2.6}$$

<u>Limit State 3: Panel buckling:</u>

The diaphragm unit shear capacity governed by buckling of the panels is determined by Equation 2.7.

$$S_{u\_b} = (3250/L_v^2) \cdot (I_x^3 t^3 p/s)^{1/4} \tag{2.7}$$

The diaphragm unit shear capacity is governed by the smallest value of the three limit-states. That is,

$$S_u = \text{minimum of } S_{u\_c}, S_{u\_f}, \text{and } S_{u\_b}$$

Step 5: Apply ASD factor of safety

The design unit shear strength is determined by Equation 2.8 where a safety factor of 2.5 is used for strength governed by screws, and 2.0 for buckling.

$$S_{allow} = S_u/\Omega \tag{2.8}$$

where:

Ω = 2.5 for strength governed by field or corner fasteners

= 2.0 for strength governed by panel buckling

Step 6: Determine input variables for stiffness

n_1 = number of corrugations per sheet with one screw at each end corrugation

n_2 = number of corrugations per sheet with two screws at each end corrugation

K_{rp} = stiffness of a single rafter-purlin connection

K_{sb} = stiffness of a single shear block connection

n_{rb} = total number of purlins connected to one rafter

Step 7: Determine screw flexibilities

The flexibility of field screws and stitch screws has been modified from the original MCA method (Leflar, 2008). The flexibility of structural field screws, S_f, is taken as a constant for all sizes of screws. The flexibility of stitch, S_{stitch}, and elevated sidelap structural fasteners, S_{purlin}, are determined using the equations shown below. If a combination of stitch and elevated sidelap structural fasteners are used, the weighted average of the screw flexibilities, S_s, is used.

S_f = 0.20 in/kip

$S_{purlin} = 3/(1000\sqrt{t})$

$S_{stitch} \quad = 4/(1000\sqrt{t})$

$S_s \quad = (n_{purlin} S_{purlin} + n_{stitch} S_{stitch})/(n_{purlin} + n_{stitch})$

Step 8: Determine stiffness coefficients

The panel edge reduction factor, K, depends on the type of sidelap (lap-up or lap-down), and cladding and purlin materials. For SCWF diaphragms with lap-up seams K = 0.5. The coefficient, ϕ, accounts for the number of spans and is assigned a value as shown below.

$$\phi = \left| \begin{array}{ll} 1.0 & \text{when } n_{spans} = 1 \\ 1.0 & \text{when } n_{spans} = 2 \\ 0.9 & \text{when } n_{spans} = 3 \\ 0.8 & \text{when } n_{spans} = 4 \\ 0.71 & \text{when } n_{spans} = 5 \\ 0.64 & \text{when } n_{spans} = 6 \\ 0.58 & \text{when } n_{spans} > 7 \end{array} \right.$$

<u>Panel warping coefficient</u>

The panel warping coefficient, D_n, accounts for the distortion of the end corrugations which causes a decrease in the in-plane diaphragm stiffness. Corrugations with screws on both sides and adjacent to major ribs at the end purlins offer the best restraint against end panel warping. For the cases with one or two screws placed next the major corrugations, Equation 2.9 is used to determine the panel warping coefficient. A D_{ni} value is computed for each end corrugation.

$$V = 1.6 \quad \text{for 1 screw per corrugation} \tag{2.9}$$

$$= 1.0 \quad \text{for 2 screws per corrugation}$$

$$D_{ni} = \begin{cases} [d_d w_t^2/(25L)]\cdot(1/t)^{1.5} & \text{for } V = 1 \\ (0.94p/w_t)\cdot(V^2)\cdot[d_d w_t^2/(25L)]\cdot(1/t) & \text{for } V = 1.6 \end{cases}$$

$$D_n = (1/n) \sum_{i=1}^{n} D_{ni}$$

<u>Fastener Flexibility Coefficient</u>

Fastener slip is accounted for by considering the flexibility contributions of each fastener depending on the fastener location relative to the panel centerline. The fastener flexibility coefficient is determined by Equation 2.10.

$$C = (EtS_f/w)\cdot[24L/(2\alpha_e + n_p\alpha_p + 2n_s(S_f/S_s)] \quad (2.10)$$

Step 9: Determine in-plane stiffness modulus

The in-plane stiffness modulus of the panel is determined using Equation 2.11. Accounted for in this equation are displacements due to shear strain of the steel panels, panel end warping, and fastener slip.

$$G' = EtK / [2.6(s/p) + \phi D_n + C] \quad (2.11)$$

Step 10: Determine effective shear modulus for diaphragm assembly

The stiffness of purlin and blocking connections are combined with the in-plane stiffness of the diaphragm to determine the effective shear modulus of the entire diaphragm assembly. The effective shear modulus is determined by Equation 2.12.

$$G'_{net} = (a/b)/[a/(G'b) + 2/K_R] \quad (2.12)$$

where:

$K_R = n_{pur}K_{rp} + n_{rsb}K_{sb}$

MATERIALS AND METHODS FOR DIAPHRAGMS

The following is a summary description of the diaphragm tests used for independent validation. Additional details can be found in the test report by Bender and Aguilera (2013). SCWF diaphragms were tested in accordance with ASAE EP558 (2004) using a simple beam configuration with monotonic loading. All diaphragms were nominally 24 ft wide by 12 ft long with three bays. Rafters were 2 x 8 Douglas-fir Select Structural lumber. Purlins were 2 x 4 Spruce-Pine-Fir 1650 Fb-1.5E lumber spaced 2 ft on center. Different diaphragm constructions were tested and are designated as Diaphragm Types. Figure 2.4 shows the general configuration for Diaphragm Types 1, 2, 2A, 3, 3A, and 6 which utilize on-edge purlins running on top of rafters with a single 60d ring shank nail for purlin-rafter connections. Splice connection were made using 3” x 0.131” smooth shank nails. Blocking of the same lumber as purlins was placed on-edge between all purlins and connected to the rafter with two 60d ring shank nails.

Diaphragm Type 5 utilized recessed purlins supported by hangers with two 10d toe-nails placed into every hanger support as shown in Figure 2.5. All diaphragms utilized the field screw pattern shown in Figure 2.6 with #10 x 1” structural screws placed one side of every major corrugation at interior purlin locations, and #10 x 1” screws place on both sides of every major corrugation at the two end purlins. Seam screw type, seam screw spacing, and steel cladding profile varied for each diaphragm and details are listed in Table 2.1.

The set up for the simple beam test is shown in Figure 2.7. The rafters were loaded in tension, and string potentiometers were used to measure rafter displacements. The net deflection, Δ_T, was calculated by Equation 2.13.

$\Delta_T = (\Delta_2 + \Delta_3 - \Delta_1 - \Delta_4)/2$ (2.13)

where:

Δ_1 = displacement of support rafter, in

Δ_2 = displacement of loaded rafter, in

Δ_3 = displacement of loaded rafter, in

Δ_1 = displacement of support rafter, in

The deflection resulting from bending, Δ_b, and chord slip, Δ_c, were subtracted from Δ_T as shown by Equation 2.14

$\Delta_s = \Delta_T - \Delta_b - \Delta_c$ (2.14)

The deflection at third points resulting from bending is given by Equation 2.15.

$\Delta_b = 5P(3a)^3/162EI$ (2.15)

where:

P = applied load, lbs

3a = width of diaphragm (285.25 in.)

E = modulus of elasticity of the purlins (1.4 x10^6 psi)

I = moment of inertia of all the purlins resisting moment and acting as deep beam (82065 in^4)

String potentiometers were placed at the location of purlin splices on the outermost purlins to measure chord slip. The deflection at third points resulting from chord slip is given by Equation 2.16.

$\Delta_c = (1/3b)\cdot\Sigma\Delta_{ci}x_i$ (2.16)

where:

b = length of diaphragm, 144 in.

Δ_{ci} = the individual chord slip measured by deflection gauges

x_i = the distance of the purlin splice to the nearest support rafter

Connection tests were conducted to determine the flexibility of the screws used in diaphragm tests. Specimens were cut out from the edge-most corrugation of used sheets that were located in the center bay of tested diaphragms. These sheets were in a zero shear zone and did not sustain damaged during testing. Specimens were 16 in. long and overlapped by 8 in. with a screw placed at the center of the overlapped portion. A 2 kip Instron universal testing machine was used to apply tension loading. A string potentiometer was used to measure the slip between the overlapped portions. The three different seam connections used in tested diaphragms were tested. The connections were fabricated using a Grandrib 3 with #12 x 0.75” stitch screw, Grandrib 3 with #12 x 1.5” elevated sidelap structural screw, and a Wick panel with #12 x 0.75” stitch screw. The test configuration is shown in Figure 2.8. Note, only the test with an elevated sidelap structural screw had a purlin supported behind the panels. A total of 10 replications were conducted for each. Screw stiffness was determined using a secant modulus at 40% of the ultimate load to correspond with the diaphragm stiffness, which was also determined at 40% of ultimate capacity.

MATERIALS AND METHODS FOR SHEAR WALLS

The following is a summary description of the shear wall tests used for comparison with values obtained from the modified MCA procedure. Additional details can be found in the test report by Bender and Aguilera (2012). SCWF shear walls were tested in accordance with ASAE EP558 (2004) using a cantilever configuration with monotonic loading. All shear walls were nominally 16 ft wide by 12 ft high with two 8-ft bays. The posts were 3-ply nail-laminated posts comprised of 2 x 6 pressure preservative treated (PPT) Hem-Fir No.2 and Douglas-Fir Larch Select Structural lumber. Girts were 2 x 4 Spruce-Pine-Fir 1650 Fb-1.5E lumber. The skirt board was 2 x 8 PPT Hem-Fir No.2 and the simulated truss chord at the top of the wall was 2 x 6 Douglas-Fir Larch Select Structural lumber. Secondary framing members were oriented flat on the posts and connected using 3.5” x 0.162” ring shank nails. Different shear wall constructions were tested and are designated as Shear Wall Types. Figure 2.9 shows the general configuration for all the shear wall tests. The screw patterns used for each shear wall are shown in Figure 2.10. The cladding type, girt spacing, field screws at the overlap locations, seam screw type, and seam screw spacing for each shear wall differed and details are provided in Table 2.2.

Locations of string potentiometers used to measure deflections are shown in Figure 2.11. The net deflection at the top of the wall, Δ_T, was calculated by Equation 2.17.

$$\Delta_T = \Delta_3 - \Delta_1 - (a/b)(\Delta_2 + \Delta_4) \quad (2.17)$$

where:

Δ_1 = horizontal displacement of the wall base measured by deflection gage 1 (in)

Δ_2 = vertical displacement of post measured by deflection gage 2 (in)

Δ_3 = horizontal displacement of the truss measured by deflection gage 3 (in)

Δ_4 = vertical displacement of post measured by deflection gage 4 (in)

a = height of shear wall (144 in. for all walls)

b = width of shear wall (192 in. for all walls)

The deflection resulting from bending, Δ_b, was subtracted from Δ_T as shown by Equation 2.18

$$\Delta_s = \Delta_T - \Delta_b \quad (2.18)$$

The deflection resulting from bending is given by Equation 2.19.

$$\Delta_b = Pa^3/3EI \quad (2.19)$$

where:

P = applied load

a = length of shear wall (144 in. for all shear walls)

E = modulus of elasticity of laminated posts (taken as the average E for the two species of lumber used: 1.6 $x10^6$ psi)

I = moment of inertia for the two outermost posts resisting moment, acting as flanges on a deep beam (456,317 in^4)

Note that Equation 2.19 yields identical results as the first term in the shear wall deflection equation found in ANSI/AWS Special Design Provisions for Wind and Seismic (SDPWS)-2008.

VALIDATION, RESULTS, AND DISCUSSION

Predictions of unit shear strength and effective shear modulus using the modified MCA procedure were compared to those obtained from seven simple beam diaphragm tests. Several deviations were made to the modified MCA method in calculating design values, as will be described herein.

Diaphragm Type 1 utilized elevated sidelap structural fasteners at the overlaps, and use of Equation 2.4 considerably overestimated diaphragm shear strength. Equation 2.3 was used in the diaphragm design example by Leflar (2008) for computing the strength of elevated sidelap structural fasteners. The validation in the study reported herein used Equation 2.3 as it provided better predictions of diaphragm strength, and is recommended that it be used in lieu of Equation 2.4 for elevated sidelap structural fasteners. The model is sensitive to changes in the flexibility of stitch and elevated sidelap structural screws. The flexibility of screws determined from testing was used in the validation because they were judged to better represent connection flexibilities of the diaphragms tested. Table 2.3 provides a comparison of screw flexibilities obtained from tests and predicted by the modified MCA equations.

The moment of inertia specified by the panel manufacture for negative bending (as panels will buckle upwards away from purlins), is typically used in Equation 2.7 to determine the buckling capacity. The buckling capacity for diaphragms with observed buckling failure was over predicted substantially. For heavily stitched diaphragms, the cladding begins to exhibit minor buckling in the flat regions between major ribs at relatively low loads. However, the diaphragm can sustain much larger loads until the major ribs begin to buckle. Since diaphragm failure resulting from panel buckling only occurs once the major ribs have buckled, the moment of inertia was reduced to account only for the major ribs and an effective width of the flat region between major ribs. By considering the major ribs as stiffened elements, the effective width of the flat region between major ribs was computed using Equation 2.20 (AISI, 2007)

$b_{eff} = \rho\ w$ when $\lambda > 0.673$ (2.20)

$b_{eff} = w$ when $\lambda \leq 0.673$

where:

$\rho = (1-0.22/\lambda)/\lambda$

$\lambda = (1.052/k)(w/t)\sqrt{f/E}$

k = 4.0 for web stiffened on both sides

w = flat web width

t = base thickness

f = stress in compression element

E = modulus of elasticity

The safety factors assumed in the modified MCA procedure are 2.5 for limit states involving screw failures and 2.0 for panel buckling. In the validation presented herein, 2.5 was used for all limit states because 2.5 has been used in the post-frame industry for tested diaphragms and is in accordance with ASAE EP558.

Using the modifications mentioned, a comparison of tested and predicted design unit shear strength for diaphragms is shown in Table 2.4. The predicted design unit shear strength for the three limit states of field screws, corner fasteners, and panel buckling are listed. Note, for panel buckling two design values are shown for comparison: (1) the predicted design unit shear strength computed using the moment of inertia specified by the panel manufacture for negative bending, and (2) using the reduced moment of inertia which only considers the major rib and an effective width of the flat region between ribs. The moment of inertia computed using the reduced section provided closer agreement with tested values where buckling was the primary failure mode. As shown in Table 2.4 the panel buckling was governing limit state predicted for Diaphragm Types 2, 2A, and 5 and was also the observed failure mode.

The highlighted values are the governing design unit shear strength used in determining the ratio of calculated to tested values. The predicted design unit shear strengths are in good agreement with test values with only slight conservative differences in all but one diaphragm. For diaphragms with more than one repetition, predicted values fell within the range of tested minimum and maximum values for all but one diaphragm. The ratio of predicted to tested design unit shear strength averaged 0.97 with a coefficient of variation of 6%. Table 2.5 provides a comparison of tested and predicted effective shear modulus for the diaphragms. The ratio of predicted to tested effective shear modulus averaged 1.07 with a coefficient of variation of 18%.

An example calculation of the modified MCA procedure as used in the validation is given in Appendix B for Diaphragm Type 2.

Predictions from the modified MCA procedure were also compared to tests from nine different SCWF shear wall constructions. The comparison was made to determine if the modified MCA procedure could also accurately predict design unit shear strength and effective shear modulus of shear walls. Table 2.6 provides a comparison of predicted and tested design unit shear strength for the shear walls. The predicted design unit shear strength for the three limit states of field screws, corner fasteners, and panel buckling are listed. Once again, two predictions for panel buckling are shown; one for the case in which the moment of inertia from panel manufacturers is used and one for the case where a reduced section was used to calculate the moment of inertia. For all shear wall types the strength is conservatively under predicted. Shear Wall Types 7 and 8 were substantially under predicted. These shear wall constructions utilized a 3 ft girt spacing, and use of the reduced section caused buckling to become the governing failure mode. However, the observed failure mode was primarily field and corner fastener failure with the presence of minor buckles in the cladding. This shows that the use of a reduced section may under predict the buckling capacity when secondary members are spaced larger than 2 feet apart. For shear walls the ratio of predicted to tested design unit shear strength averaged 0.81 with a coefficient of variation of 15%. The difference in predicted and tested design unit shear strength may be attributed to the fact that loading was perpendicular to the corrugations instead of parallel to the corrugations.

The modified MCA procedure was used to predict the effective shear modulus of shear walls. Lacking, however, were the girt-to-post connection stiffness required for use with the model.

Leflar (2008) determined the stiffness of a 2 x 4 purlin oriented on-edge and connected with a single 60d rink shank nail. Leflar also determined the stiffness of a 2 x 4 shear block oriented on-edge and connected with 2-60d ring shank nails. The purlin and shear block connections were determined to be 1.0 and 10.0 kip/in, respectively. The girt connections are likely considerably stiffer than a purlin connection. However, the blocking connection may provide a better approximation of girt connection stiffness. Blocking in the tested diaphragms utilized 2-60d ring shank nails with an average diameter of 0.178 in. The girt-to-post connections in shear wall tests utilized three ring shank nails with an average diameter of 0.149 in. In the absence of required stiffness values for the girt-to-post connection, the girt connection was assumed to be the same stiffness as a blocking connection used in diaphragms (10 kips/in). With use of this assumed stiffness value, a comparison of predicted to tested effective shear modulus is presented in Table 2.7. The ratio of predicted to tested effective shear modulus was 0.81 with a coefficient of variation of 39%. The effective shear modulus is under predicted for most cases. The notable differences in tested and predicted effective shear modulus values may be attributed to the post carrying additional moment since the post supports at the base were in between a pinned and fixed connection. Also, the girt-to-post connection stiffness assumed to be 10 kip/in may not be valid and should be verified with connection testing.

SUMMARY AND CONCLUSIONS

An independent validation of the modified MCA procedure was made by comparing predicted strength and stiffness values to tested values obtained from seven different SCWF diaphragm constructions and nine different SCWF shear walls constructions. The comparison in Table 2.4 shows that unit shear strength for diaphragms was in good agreement with tested values. The ratio of predicted to tested design unit shear strength averaged 0.97. The effective shear modulus had larger differences in predicted and tested values with the ratio of predicted to tested effective shear modulus averaging 1.07. The results are within reason when one considers that some diaphragm types had only one replication, and large variability of stiffness results is typical from testing due to the inherent variability in wood properties. For shear walls the ratio of predicted to tested design unit shear strength averaged 0.81. The ratio of predicted to tested effective shear modulus also averaged 0.81 but had larger variability. The design unit shear strength was under predicted for all shear wall types, and the effective shear modulus was under predicted in most cases. The modified MCA procedure does not appear to be as accurate for shear walls compared to diaphragms, however the predictions are conservative. It should also be noted that the girt-to-post connection stiffness was assumed to be 10 kips/in which may not be valid.

Slight modifications were made in computing design values. It is recommended that elevated sidelap structural screws be treated as stitch screws by using Equation 2.3 when computing fastener strength. The equation for the limit state of panel buckling over predicts the design strength obtained from testing when using the moment of inertia from panel manufacturers. It is recommended that a reduced moment of inertia be computed using Equation 2.20 in which only the major rib and a flat portion between the major ribs are considered effective. The use of a reduced moment of inertia produces good agreement with diaphragms and shear walls utilizing 2 ft member spacing when buckling was the observed failure mode.

Although this study concluded the modified MCA procedure provides an accurate means to analytically predict the design values for diaphragms, the time investment to learn the procedure and its complexity are significant barriers to implementation. As a means of simplifying the implementation of the modified MCA procedure, the following chapter will utilize the model to develop design values for common constructions of SCWF diaphragms used in the post-frame industry.

REFERENCES

Anderson, G. A. 2011. Modification of the MCA Procedure for Strength and Stiffness of Diaphragms used in Post-Frame Construction. Frame Building News (June): 22-25.

AISI. 2007. North American Specification for the Design of Cold-Formed Steel Structural Members, Washington, D.C.

ASAE. 2012. ANSI/ASAE Standard EP484.2. Diaphragm Design of Metal-Clad, Wood-Frame Rectangular Buildings. ASABE Standards, Engineering Practices, and Data. St. Joseph, Mich.: ASABE.

ASAE. 2004. ANSI/ASAE Standard EP558 Load Tests for Metal-Clad Wood-Frame Diaphragms. ASABE Standards, Engineering Practices, and Data, American Society of Agricultural and Biological Engineers, St. Joseph, MI.

ASAE. 2014. ANSI/ASAE Standard EP558 Load Tests for Metal-Clad Wood-Frame Diaphragms. ASABE Standards, Engineering Practices, and Data, American Society of Agricultural and Biological Engineers, St. Joseph, MI.

Bender, D.A. and D. Aguilera. 2012. Development of Design Data for Steel-Clad, Wood-Framed Shear Walls. CMEC Technical Report 12-004. Washington State University, Pullman, WA.

Bender, D.A. and D. Aguilera. 2013. Development of Design Data for Steel-Clad, Wood-Framed Diaphragms. CMEC Technical Report 12-030. Washington State University, Pullman, WA.

Bohnhoff, D. R., P. A. Boor, F. A. Charvat, M.H. Gadani. 2003. UW & LBS full-scale metal-clad wood-frame diaphragm study. Report 3: Building Load, Configurations, Load Cases and Data Analysis Methods. ASAE Paper No. 034004. St. Joseph, Mich.: ASAE

Gebremedhin, K. G., E. L. Bahler, and S. R. Humphreys. 1986. A modified approach to post-frame design using diaphragm theory. Transactions of the ASAE 29(5): 1364-1372.

Gebremedhin, K. G., J. A. Bartsch and M. C. Jorgensen. 1992. Predicting roof diaphragm and endwall stiffness from full-scale test results of a metal-clad, post-frame building. Transactions of the ASAE 35(3):977-985.

Hoagland , R. C. and D. S. Bundy. 1983. Post-frame design using diaphragm theory. Transactions of the ASAE 26(5): 1499-1503.

Hurst, H. T. and P. H. Mason, Jr. 1961. Rigidity of end walls and cladding on pole buildings. Journal of the ASAE 42(4):188-191.

Johnston, R. A. and J. O. Curtis. 1984. Experimental verification of stress skin design of pole-frame buildings. Transactions of the ASAE 27(1):159-164.

Keener J.D. and H.B. Manbeck. 1996. A Simplified Model for Predicting the Behavior of Metal-Clad, Wood-Framed Diaphragms. ASAE Paper No. 95-4456, St. Joseph, MI.

Leflar, J.A. 2008. A mathematical Model of Steel-Clad Wood-Frame Shear Diaphragms. Unpublished M.S. thesis, Department of Civil and Environmental Engineering, Colorado State University, Fort Collins, CO.

Luttrell, L.D. and J.A. Mattingly. 2004. A Primer on Diaphragm Design. Metal Construction Association, Glenview, IL

McFadden, V. J., D. S. Bundy and A. D. Lukens. 1991. Evaluation of the wind loading of a post-frame structure. Transactions of the ASAE 34(4):1860-1865.

Wright, B.W. and H.B. Manbeck. 1993. Finite Element Analysis of Wood-Framed, Metal Clad Diaphragms. American Society of Agricultural and Biological Engineers, St. Joseph, MI.

TABLES

Table 2.1: Cladding, screw type, and spacing used for each diaphragm

Diaphragm Type	Reps	Cladding Type	Seam Screw	O.C. Spacing (in)
1	3	Grandrib 3	#12x1.5" elevated sidelap structural screw	24
2	3	Grandrib 3	#12x0.75" stitch screw	8
2A	1	Wick Panel	#12x0.75" stitch screw	8
3	2	Grandrib 3	#12x0.75" stitch screw	24
3A	1	Wick Panel	#12x0.75" stitch screw	24
5	2	Grandrib 3	#12x0.75" stitch screw	8
6	1	Grandrib 3	#12x0.75" stitch screw	12

Table 2.2: Construction properties for each shear wall

Shear Wall Type	Reps	Cladding Type	Girt Spacing (ft)	#10x1" structural fasteners adjacent to the overlap rib in flats	#12x1.5" elevated sidelap structural fasteners	#12x.75" stitch fastener
1	1	Grandrib3	3	1 side	----	----
2	3	Grandrib3	3	Both sides	----	----
3	1	Grandrib3	2	Both sides	----	----
4	3	Grandrib3	2	1 side	24” o.c.	----
5	2	Grandrib3	2	1 side	----	8" o.c.
6	2	Grandrib3	2	1 side	----	24” o.c.
7	3	Grandrib3	3	1 side	----	18” o.c.
8	1	Wick	3	1 side	----	18” o.c.
9	1	Wick	2	1 side	----	8" o.c.

Table 2.3: Comparison of seam screw flexibilities

Profile	Screw	Tested Flexibility (in/kip)	Flexibility per Modified MCA Procedure (in/kip)
Grandrib 3	#12x3/4"	0.054	0.034
Grandrib 3	#12x1.5"	0.043	0.025
Wick	#12x3/4"	0.036	0.033

Table 2.4: Comparison of tested and predicted design unit shear strength for diaphragms

Diaphragm Type	Tested (lb/ft)	Predicted design capacities per modified MCA procedure (lb/ft)[a]				Ratio: Calc/Test	Failure Observed in Testing[b]
		Field Fastener	Corner Fastener	Buckling (I_x=manufact.)	Buckling (I_x=reduced section)		
1	123	125	122	373	213	0.99	1
1	118	125	122	373	213	1.03	1
1	115	125	122	373	213	1.06	1
2	218	251	219	373	213	0.98	2
2	212	251	219	373	213	1.01	2
2	222	251	219	373	213	0.96	2
2A	223	246	198	323	191	0.85	2
3	129	125	122	373	213	0.94	1
3	132	125	122	373	213	0.92	1
3A	117	112	107	323	191	0.92	1
5	209	251	219	373	213	1.02	2
5	219	251	219	373	213	0.97	2
6	179	188	174	373	213	0.97	1
					Average =	0.97	
					COV =	6%	

[a] Highlighted values indicate the governing design unit shear strength predicted by the modified MCA procedure.
[b] Failure modes:
1. Primarily field and corner fastener failure. Minor buckling of panels was also observed.
2. Panel buckling

Table 2.5: Comparison of tested and predicted effective shear modulus for diaphragms

Diaphragm Type	Tested (kips/in)	Calculated per Modified MCA Procedure (kips/in)	Ratio: Calc/Test
1	6.3	8.5	1.34
1	7.9	8.5	1.07
1	6.2	8.5	1.36
2	10.9	11.9	1.09
2	13.4	11.9	0.89
2	14.7	11.9	0.81
2A	13.6	13.1	0.97
3	6.8	7.7	1.14
3	7.4	7.7	1.04
3A	7.3	9.0	1.24
5	8.8	N/A[a]	N/A[a]
5	8.1	N/A[a]	N/A[a]
6	13.4	10.5	0.79
		Average =	1.07
		COV =	18%

[a] Effective shear modulus could not be computed because connection stiffness for recessed purlins supported by hangers was not known.

Table 2.6: Comparison of tested and predicted design unit shear strength for shear walls

Shear Wall Type	Tested (lb/ft)	Predicted design capacities per modified MCA procedure (lb/ft)[a]				Ratio: Calc/Test	Failure Observed in Testing[b]
		Field Fastener	Corner Fastener	Buckling (Ix=manufact.)	Buckling (Ix=reduced section)		
1	74	66	67	166	95	0.89	1
2	85	85	85	166	95	0.99	1
2	111	85	85	166	95	0.76	1
2	105	85	85	166	95	0.80	1
3	119	117	115	373	214	0.96	3
4	144	127	124	373	214	0.86	3
4	137	127	124	373	214	0.90	3
4	154	127	124	373	214	0.80	3
5	239	253	221	373	214	0.90	2
5	244	253	221	373	214	0.88	2
6	147	127	124	373	214	0.84	3
6	138	127	124	373	214	0.90	3
7	140	133	129	166	95	0.68	3
7	150	133	129	166	95	0.64	3
7	151	133	129	166	95	0.63	3
8	149	124	118	144	86	0.58	3
9	256	248	200	323	192	0.75	2
					Average =	0.81	
					COV =	15%	

[a] Highlighted values indicate the governing design unit shear strength predicted by the modified MCA procedure.
[b] Failure modes:
1. Combined field and corner fastener failure
2. Panel buckling
3. Primarily field and corner fastener failure. Minor buckling of panels was also observed.

Table 2.7: Comparison of tested and predicted effective shear modulus for shear walls

Shear Wall Type	Tested (kips/in)	Calculated per Modified MCA Procedure (kips/in)	Ratio: Calc/Test
1	3.0	1.9	0.63
2	4.5	2.2	0.49
2	6.6	2.2	0.33
2	5.7	2.2	0.38
3	4.5	2.9	0.65
4	5.5	8.7	1.56
4	7.2	8.7	1.21
4	8.9	8.7	0.97
5	14.2	12.3	0.86
5	15.1	12.3	0.82
6	7.6	7.9	1.04
6	7.7	7.9	1.02
7	13.6	7.7	0.57
7	10.9	7.7	0.71
7	9.0	7.7	0.87
8	14.9	8.8	0.59
9	11.9	13.6	1.14
		Average =	0.81
		COV =	39%

FIGURES

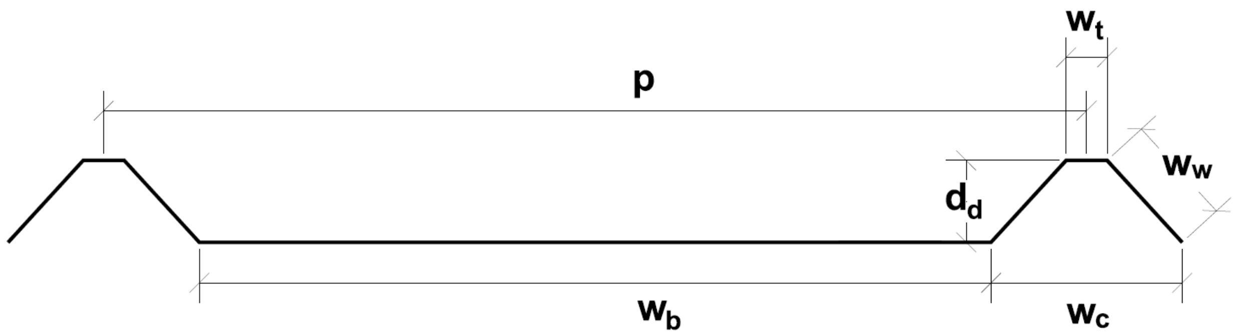


Figure 2.1: Geometric properties of cladding profile

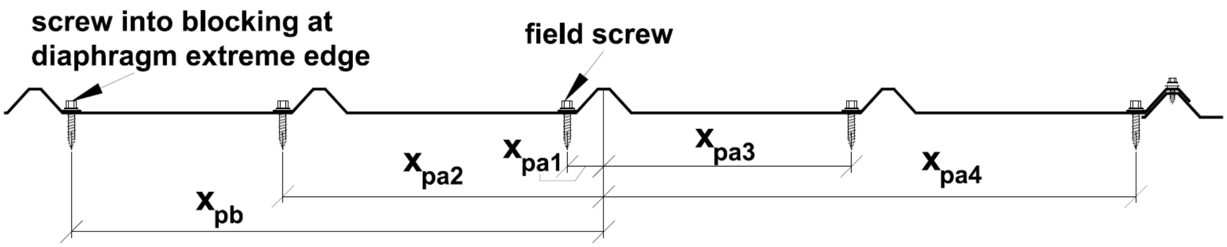


Figure 2.2: Screw distances relative to panel centerline for typical screw pattern

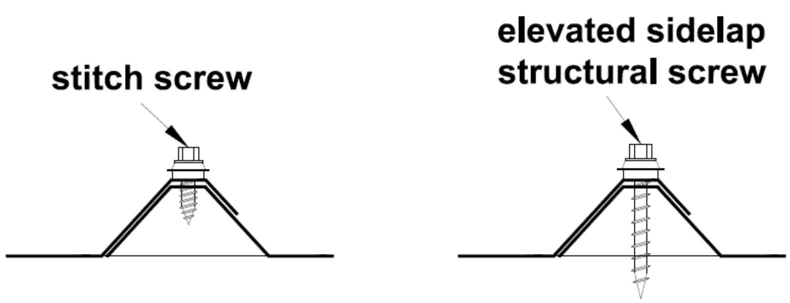


Figure 2.3: Stitch and elevated sidelap structural screws

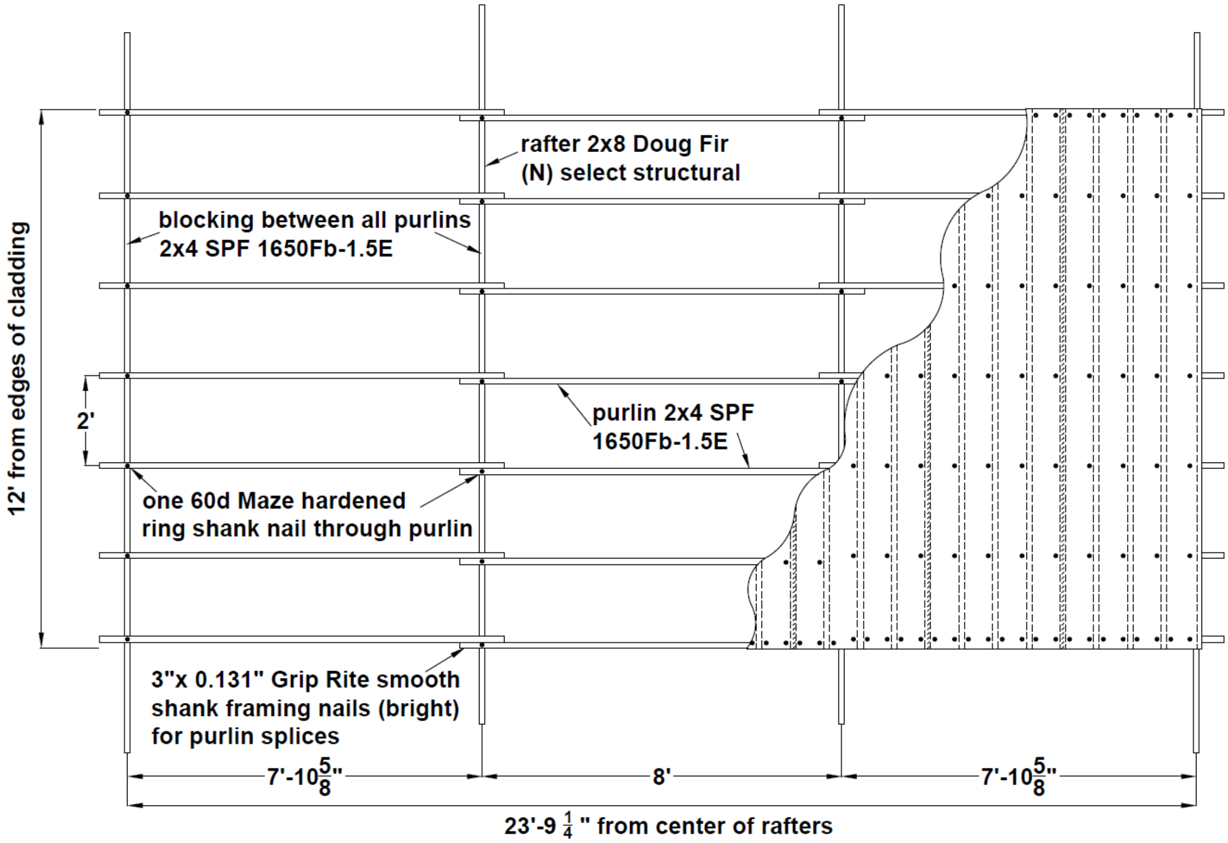


Figure 2.4: Diaphragm construction with purlins on top of rafters

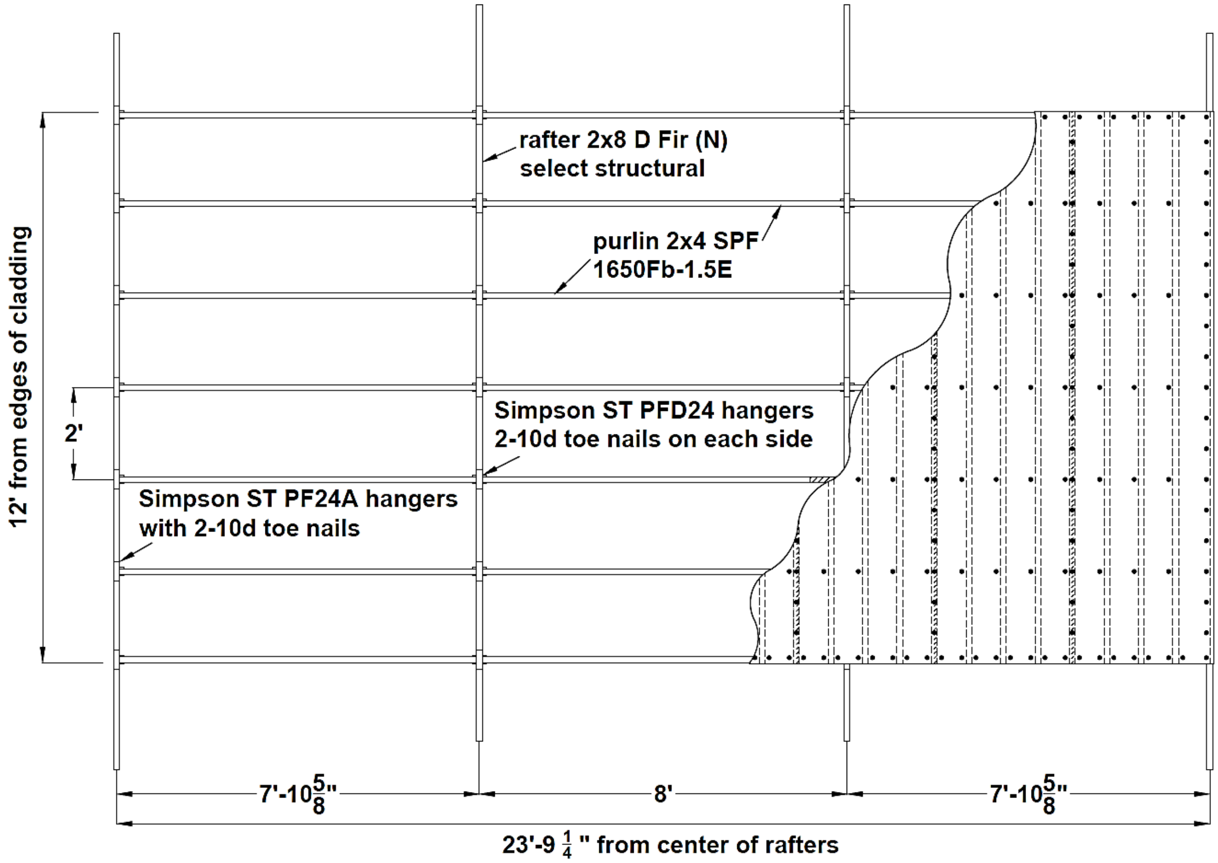


Figure 2.5: Diaphragm construction with recessed purlins

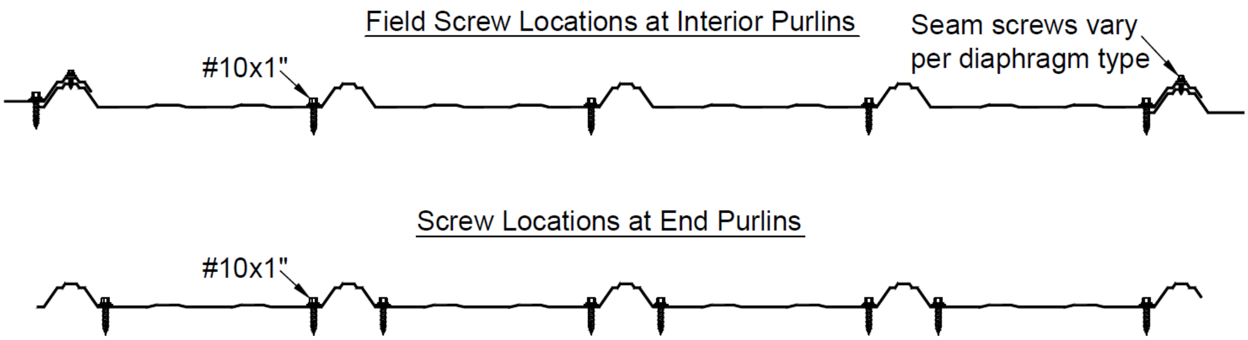


Figure 2.6: Diaphragm screw pattern

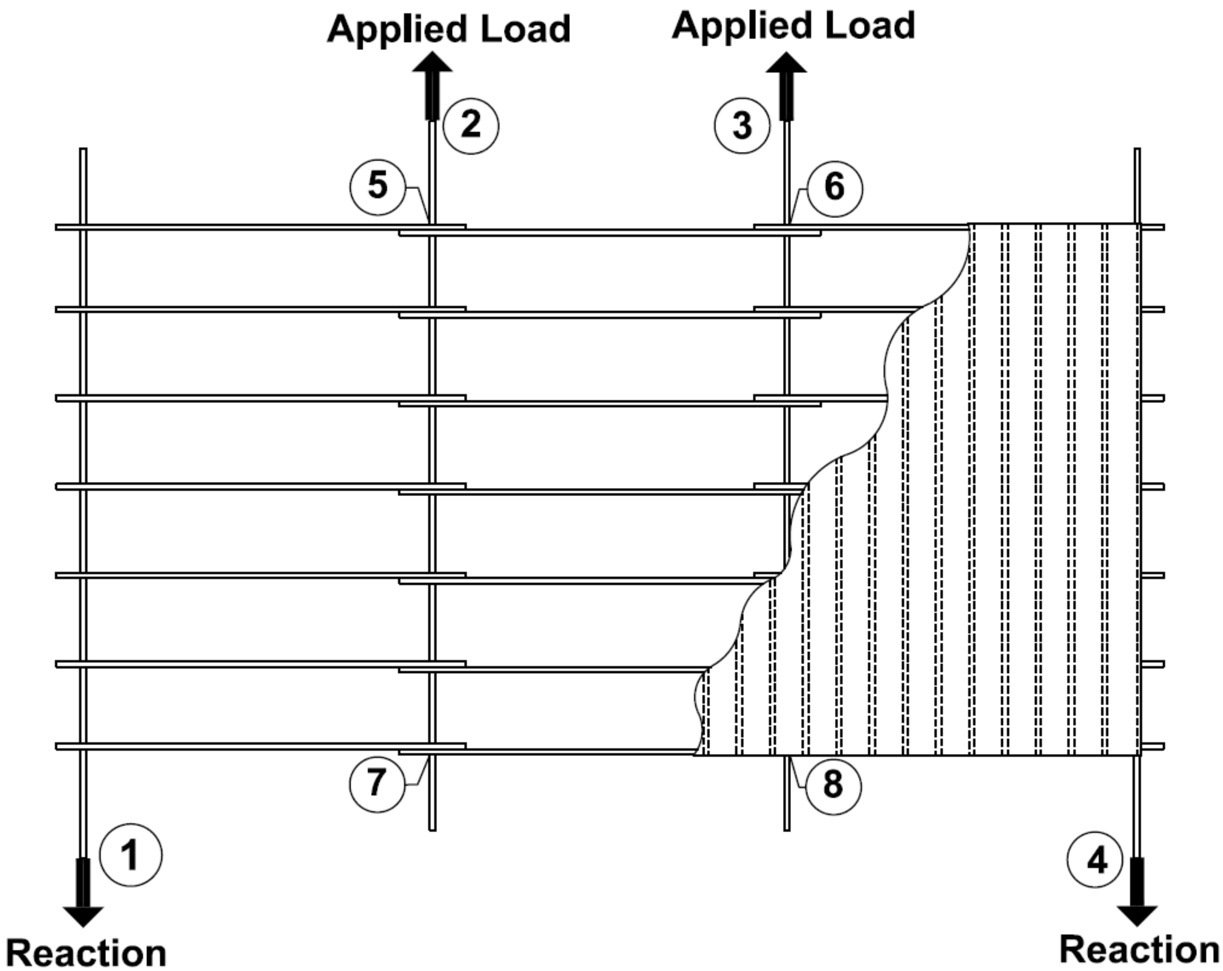


Figure 2.7: Location of string potentiometers for simple beam diaphragm test

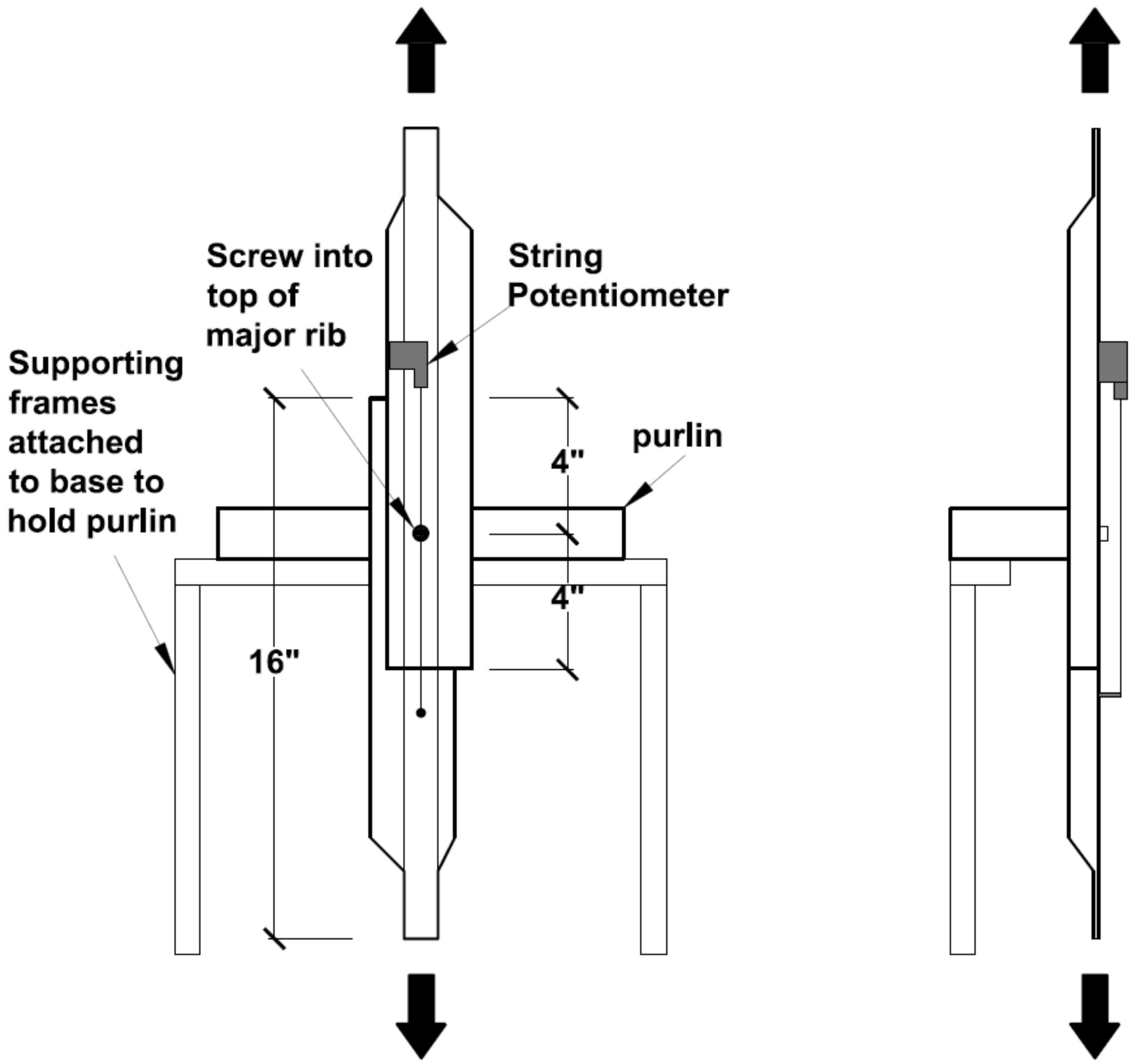


Figure 2.8 Front and side view of screw test configuration

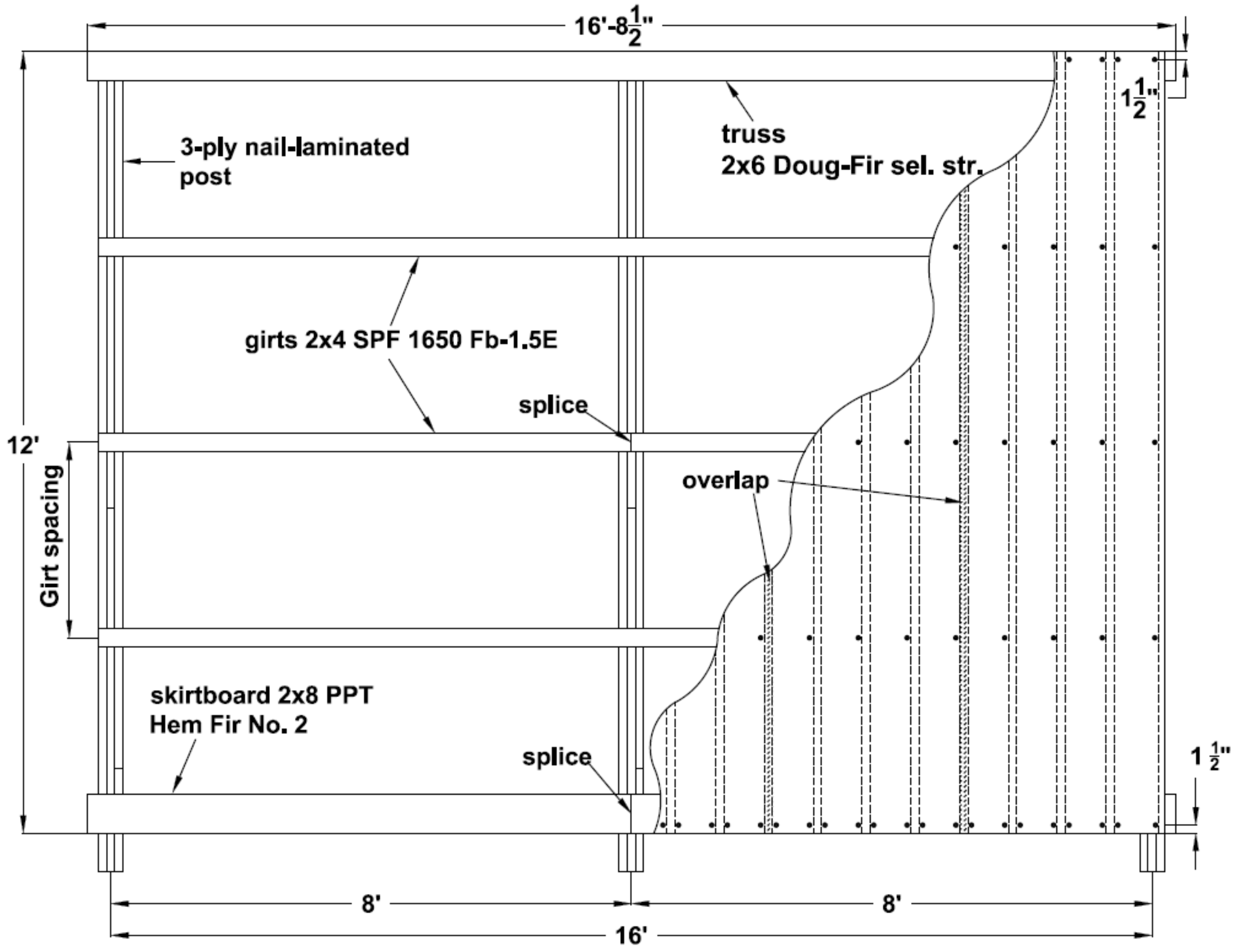


Figure 2.9: Shear wall construction

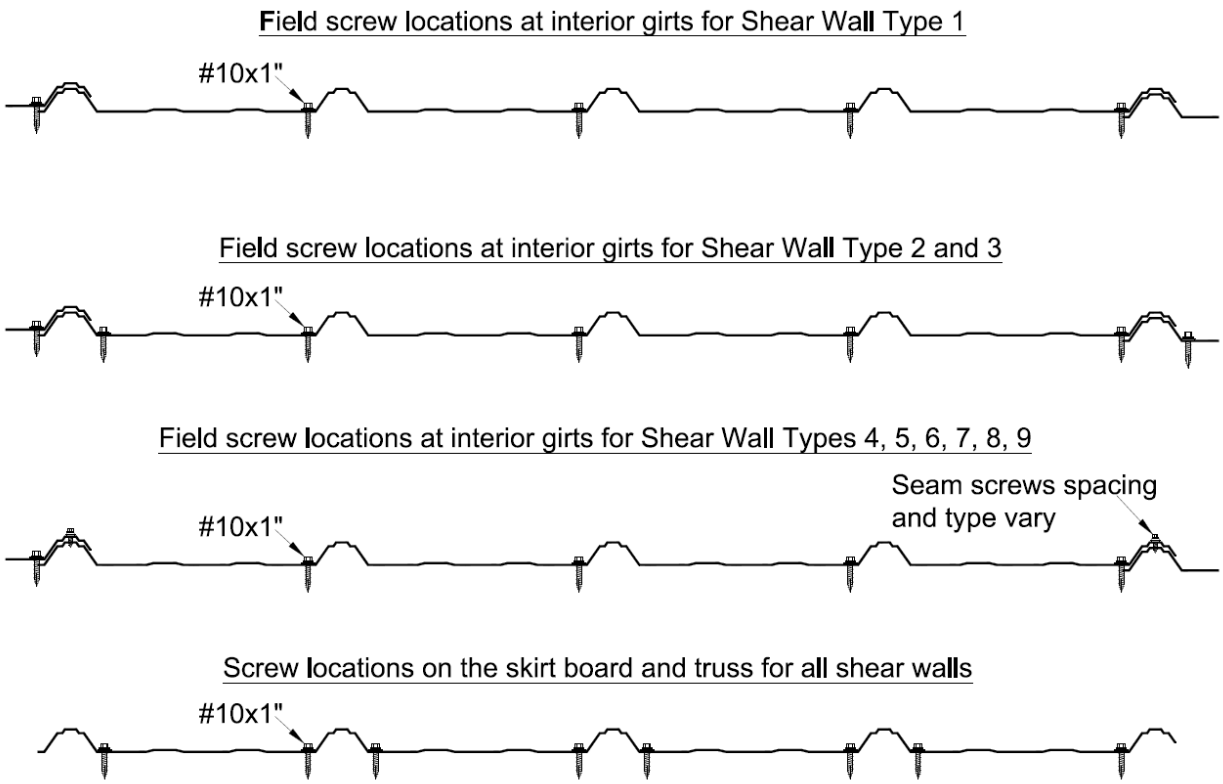


Figure 2.10: Shear wall screw patterns

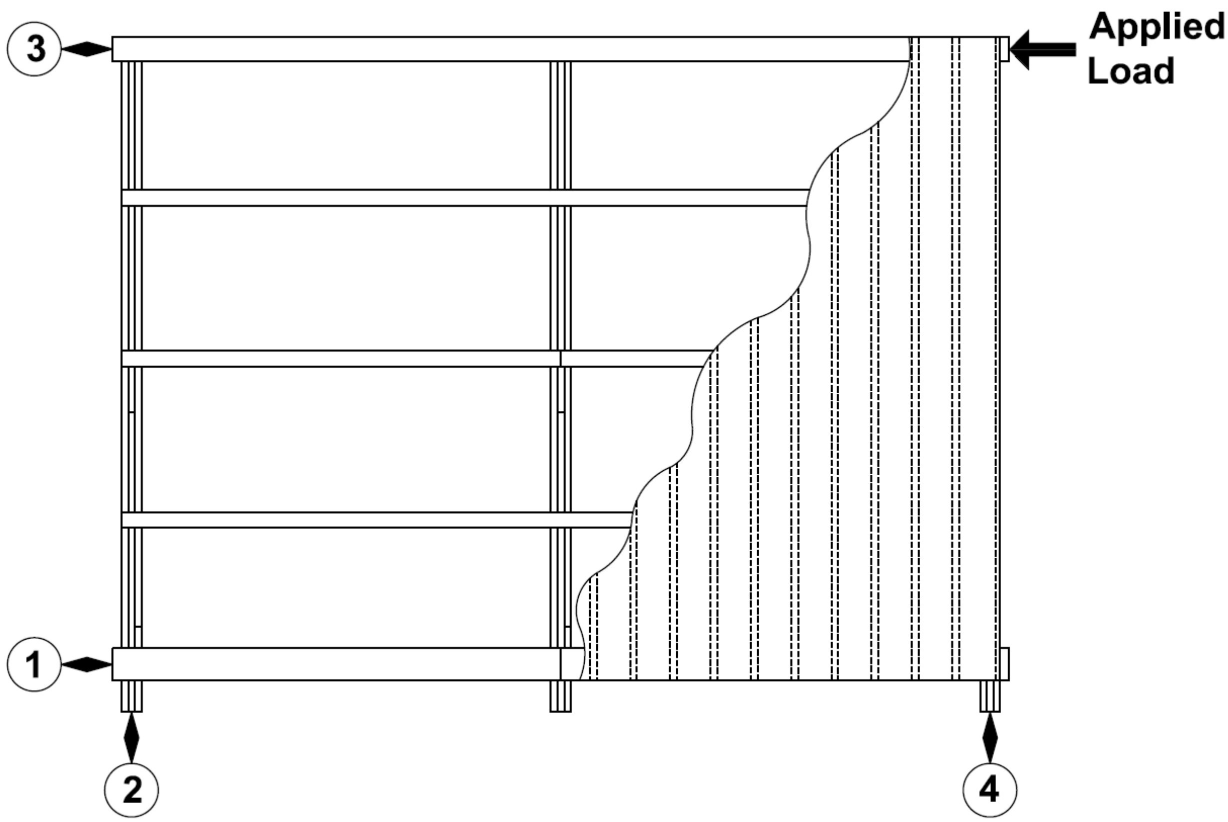


Figure 2.11: Location of string potentiometers for cantilever shear wall test

CHAPTER 3

DETERMINATION OF DESIGN UNIT SHEAR STRENGTH AND EFFECTIVE SHEAR MODULUS VALUES FOR STEEL-CLAD, WOOD-FRAMED DIAPHRAGMS

ABSTRACT

Post-frame buildings are typically sheathed with light gauge corrugated steel cladding that functions as the main lateral resisting system through diaphragm action. For proper design and structural analysis, designers must know the strength and stiffness of these components. A mathematical model, often referred to as the *Modified MCA Procedure*, is the most accurate procedure to-date for analytical prediction of design values for steel-clad, wood-framed (SCWF) diaphragms. The model has provided good agreement with small-scale diaphragm tests, but the time investment to learn the procedure and its complexity are significant barriers to implementation. This study utilizes the modified MCA procedure to provide a simplified approach for prediction of strength and stiffness of common constructions of SCWF diaphragms used in post-frame buildings. Design tables and accompanying adjustment formulas are provided for the case in which purlins are oriented on-edge and nailed to the top of rafters. These tables are intended as a convenient and credible tool for diaphragm design of post-frame buildings.

INTRODUCTION

Small-scale diaphragm tests have long been the primary method of establishing unit shear strength and effective shear modulus values for SCWF diaphragms. Anytime a construction variable is changed (i.e. screw pattern, cladding profile, screw sizes, member spacing, etc.) a new test must be conducted. This is an expensive and cumbersome method for development of design values, and as a result, the database for tested diaphragm constructions is limited.

The Metal Construction Association (MCA) published a mathematical model, titled *A Primer on Diaphragm Design,* for the prediction of shear strength and stiffness of steel-clad, steel-frame (SCSF) diaphragm. The model, developed by Luttrell and Mattingly (2004), has been accepted and widely used in the steel building industry. Leflar (2008) and Anderson (2011) modified the MCA procedure to predict shear strength and stiffness of SCWF. The model was validated by Leflar and is suggested as an alternative to small-scale diaphragm tests. The design procedure is outlined in the flowchart in Figure 3.1.

Shear strength and stiffness values are tabulated in the ANSI/AWS Special Design Provisions for Wind and Seismic (SDPWS) for wood-sheathed, wood-frame diaphragms and shear walls allowing for a simple design approach that many structural engineers are very familiar with. The development of similar tables for SCWF diaphragms would prove beneficial by simplifying the design process, saving time, and providing a sense of familiarity to design engineers. The modified MCA procedure can provide the basis for developing similar tables those in the SDPWS.

One of the main concerns of using small-scale diaphragm tests to predict unit shear strength design values is they do not necessarily mimic the responses of larger test panels. This effect is largely due to different screw configurations being used for end purlins and interior purlins. Often screws are placed on both sides of the major rib at end purlins, while only a single screw is placed on one side of major ribs at interior purlins. The end purlin contains twice as many screws as an interior purlin, thus providing approximately twice the strength. This can result in a larger unit shear strength developed for a smaller panel than would actually be developed for a diaphragm test panel of larger length. However, the modified MCA procedure predicts this decrease in unit shear strength with increase length of the panel. This length effect will be addressed in design tables developed in this study. For clarification, the term panel length or diaphragm length as used in this study corresponds to the dimension parallel to the major corrugations of the cladding and parallel to the rafters, which is the eave to ridge distance for a diaphragm in an actual building. Further, the modified MCA procedure includes the stiffness of purlins and blocking which have traditionally been incorporated in the stiffness of SCWF diaphragms. The original MCA method only predicted the in-plane stiffness of the diaphragm which accounts for deflections resulting from shear strain of the steel, end panel warping, and fastener slip between the framing and cladding. The deflections from purlins and blocking are accounted for in the amended model.

The objective of this study is to utilize the modified MCA procedure to provide a simplified approach to analytically derive strength and stiffness for common SCWF diaphragm constructions. Further, possible differences between test panels and full-scale diaphragms in regards to strength will be addressed.

LITERATURE REVIEW

ANSI/ASAE EP484.2 (2012) is the diaphragm design standard for post-frame buildings. The distribution of lateral loads between the frames and diaphragm are considered. The three methods for determining load sharing are with the use of load distribution tables, the force distribution method, and DAFI software. Diaphragms used in the post-frame industry have additional elements that were not accounted for by the original MCA model. Purlins and blocking are incorporated into the diaphragm stiffness because the structural analog used to develop the diaphragm design procedures is a two-element spring analog as shown in Figure 3.2. The stiffness of frames/shear walls is denoted as k, and the stiffness of the diaphragm between frames is denoted as C_h. While horizontal loads are uniformly distributed along the building length as shown in Figure 3.3, they are applied as concentrated loads at frames and shear walls locations at the building eave for use with the structural analog. The structural analog requires the stiffness of only *two* elements: the frame/shear walls and the diaphragm. However, the load path to the diaphragm is such that loads enter the building at frame locations and the load transfers from frames, through the purlin connections, to the diaphragm; *three* elements must be accounted for. The purlin connection should not be neglected in structural analysis because it forms the load path from frames to the diaphragm.

Traditionally, the purlin connection has been accounted for by incorporating purlins (and blocking if used) into the diaphragm stiffness for small test panels. ASAE EP558 (ASABE, 2014), stipulates that deflection measurements shall be taken on loaded rafters, and in doing so the resulting stiffness will be one which incorporates purlins, and blocking if used, into the stiffness of the diaphragm. This allows use of procedures in EP484.2 without additional consideration of the purlin or blocking components. The modified MCA procedure takes the same approach in incorporating the purlins and blocking into the diaphragm stiffness. Bohnhoff et al. (1999) have suggested the use of a more complex three-element spring analog that requires the stiffness of all three elements to be known. This could yield more accurate predictions of eave deflections, however, current design procedures do not reflect an adoption of such an analog.

Limited tests have been conducted to investigate the effect of diaphragm length on strength and stiffness. Bohnhoff and Williams (1999) conducted cantilever tests on 10, 20, and 30 ft diaphragm lengths with a 9 ft rafter spacing. Results showed that diaphragm unit shear strength was constant with increases in length. However, there was no indication that end purlins utilized more screws or a different screw pattern than interior purlins in the tests. Bohnhoff and Williams determined effective shear modulus values computed on the basis of both rafter and cladding displacements. Results showed the effective shear modulus was constant when rafter displacement was measured. However, the effective shear modulus computed on the basis of cladding displacement increased at an increasing rate as the diaphragm length increased.

Lukens (1988) conducted cantilever tests on 6, 8, 12, 16, and 20 ft. diaphragm lengths with a 9 ft. rafter spacing. The screw pattern at the end purlins contained twice as many screws as the interior purlins. Results showed that the design unit shear strength decreased as diaphragm length increased. Research by Lukens will be used in this study to compare tested and predicted unit shear strength predictions made by the modified MCA procedure for diaphragms of longer length.

DIAPHRAGM CONSTRUCTION PROPERTIES

The modified MCA procedure was used to establish design strength and stiffness values. The goal was to provide design tables inclusive of common diaphragm construction used in industry. The modified MCA procedure takes into account a large amount of variables primarily relating to geometric and material properties of the cladding, geometric properties of the diaphragm assembly, fasteners sizes, and the location of fasteners relative to the panel centerline. The development of tables first required establishment of the types of materials, their properties, and common corrugated sheet profiles used in the post-frame industry.

Corrugated Sheets

Post-frame buildings utilize light-gauge, cold-formed steel sheets with major and minor ribs that can vary in size per manufacturer. While minor ribs can contribute to increased bending strength, they were conservatively ignored in analysis assuming that regions between major ribs were flat. The major ribs were simplified as trapezoidal in shape, conservatively ignoring features such as additional bumps on the ridge of major ribs.

It was difficult to determine common dimensions for cladding profiles because many panel manufacturing websites do not provide all panel dimensions required for analysis. The corrugated steel panels used to establish the range of major rib dimensions are shown in Table 3.1. Based on the information found, the range of major rib dimensions included in the analysis had major rib depths varying from 0.625 to 1 in., the bottom of major rib widths varying from 1 to 2.5 in., and the top width of major ribs from 0.25 to 0.75 in. Leeway was provided to include sheets that may have slightly smaller or larger dimensions than those panels that were surveyed. The simplified geometry of corrugated sheets and dimension ranges for major ribs included in the analysis are shown in Figure 3.4.

Common spacing of major ribs is 9 in. and 12 in. on center. For profiles with 9 in. major rib spacing 30 (0.120 in.), 29 (0.135 in.), and 28 (0.149 in.) gauge steel sheets were considered in the analysis. For 12 in. major rib spacing 29 (0.135 in.), 28 (0.149 in.), and 26 (.0179 in.) gauge steel sheets were considered in the analysis. Steel sheets ranging from 30-28 gauge are typically formed from full-hard galvanized steel with a minimum yield strength of 80 ksi and ultimate strength of 82 ksi. The thicker 26 gauge steel panels typically have lower strength properties, and were evaluated at a yield strength of 50 ksi and ultimate strength of 52 ksi. Only sheets with a 36 in. coverage width were considered in analysis, which is typical of most panel products used in post-frame applications.

Purlins

The analysis considered purlin spacing of 2 ft. on center, which is typical for most tested diaphragms. This allows use of the repetitive member factor, C_r, for bending strength in accordance with NDS provisions (AF&PA, 2008). With regards to purlin specific gravity, the approach in post-frame literature and diaphragm tests has been to use lumber with a low specific gravity allowing lumber species with a higher specific gravity to be conservatively substituted. The same approach will be used with design tables in which all unit shear strength values will be computed assuming a purlin specific gravity of 0.42.

Fastener Sizes

While a variety of structural screws can be used to fasten sheets to the wood frame, only a select few were included in the analysis. Test reports and literature revealed common use of No. 9-1”, 10-1”, and 10-1.5” field screws. No. 12-1.5” screws were also included to allow slightly higher shear design values.

Screws through the overlapping seams are generally classified as one of the following: elevated sidelap structural fasteners or stitch screws. Elevated sidelap structural fasteners are used in the overlapping seams at purlin locations only, penetrating both sheets and the purlin below. Stitch screws only penetrate both sheets at the overlap. For design tables, elevated sidelap structural fasteners were not considered because they can only be placed at a spacing equal to that of the purlin spacing. They also have a larger thread pitch to allow easy penetration into the purlin, however the large thread pitch does not function well in tightly fastening the two sheets together. Stitch screws may be placed at any spacing, and the smaller thread pitch serves well in providing a tight steel-to-steel connection. No. 10 and 12 screws through the overlap are common in tested diaphragms and were included in the analysis. Spacing of stitch screws is analyzed at 24, 12, and 8 in. on center.

Fastener Locations

While structural fasteners can be placed anywhere in the field where a framing member lies beneath, a standard fastening pattern had to be established for use with design tables. For diaphragms utilizing stitch screws, structural field screws must be placed on one side of all major ribs, and stitch screws must be placed through the ridge at sheet overlaps as shown in Figure 3.5. For diaphragms without stitch screws, structural field screws must be placed on one side of each major rib, except at panel overlaps, where screws must be placed on both sides of major ribs as shown in Figure 3.6. This fastening pattern was typical in tested diaphragms.

At end purlins, screws are typically placed adjacent to and on both sides of every major rib. The structural purpose behind this practice is to limit end panel warping. When the diaphragm is loaded, major ribs warp and roll to one side. The screws on both sides of the major rib at panel ends aid in limiting end panel warping and provides a stiffer diaphragm. The use of only a single screw next to major ribs may cause the flat region between major ribs to lift up on the side without a screw. For use of design tables, screws must be placed adjacent to the major rib on both sides at end purlins as shown in Figure 3.7.

MODEL DEVELOPMENT

Since screws are placed adjacent to major ribs, the fastener pattern is dictated by major rib spacing. Consequently, separate design tables are presented for cladding profiles with 12 inch and 9 inch major rib spacing. Additionally, separate tables had to be developed for strength governed by screw failure and strength governed by panel buckling. As will be discussed later in this section, the design unit shear strength governed by screw failure decreases with increases in cladding length parallel to the corrugations. The length effect will be accounted for by tabulating design unit shear strengths governed by screw failure for a 10 ft and 50 ft long cladding length (eave to ridge distance). A non-linear adjustment formula is provided to calculate design unit shear strength for diaphragms of intermediate lengths.

The procedure used to develop unit shear strength governed by screw failure and in-plane stiffness is shown in the flowchart in Figure 3.8. The flowchart is for constructions utilizing a cladding profile with 9 in. major rib spacing. Cladding profiles with a 12 in. major rib spacing utilized several different cladding properties, and a footnote at the bottom of Figure 3.8 indicates replacement input values. The process as shown in the flowchart will be explained.

Input 1 values depicted in Figure 3.8 are those variables held as constants for all constructions in the initial analysis. Diaphragm length, purlin spacing, and purlin specific gravity are taken as 10 ft, 2 ft, and 0.42, respectively. A combination of construction materials from Input 2 values were then selected. Holding Input 1 and 2 values as constants, the modified MCA procedure was run using all combinations of the major rib dimensions listed under Input 3 with 0.125 in. incremental changes in dimensions. Thus, a total of 140 different cladding configurations were considered. The construction yielding the lowest design unit shear strength governed by screw failure (minimum value for the two limit states of field and corner screw failure) is the value presented in design tables for a 10 ft long diaphragm. The governing construction for shear strength governed by screw failure was then evaluated for diaphragm lengths ranging from 10 ft to 50 ft on one foot increments. The design unit shear strength for a 50 ft diaphragm is presented in design tables. A new combination of Input 2 values was chosen and the process was repeated. The process was completed a total of 84 times for each of the 9 and 12 in. major rib spacing profiles. Thus, a total of 168 different constructions are presented in the two tables combined, allowing use of any cladding profile within the specified range of major rib dimensions evaluated. The in-plane stiffness of the diaphragm was computed using the same procedure, however, only one in-plane stiffness value is presented in the design tables. This is the lowest in-plane stiffness obtained from the analysis of all 140 cladding configurations and changes in diaphragm length from 10 to 50 ft.

The third limit-state of panel buckling is not dependent on fasteners and had to be treated separately. Strength governed by panel buckling is a function of the sheet profile, base metal thickness, moment of inertia, and purlin spacing. The 9 and 12 in. major rib spacing profiles each required the development of three separate design unit shear strength tables; one for each of the three steel thicknesses. Panel buckling design values are presented such that a designer can chose the dimensions of the major rib and determine the buckling design strength of the panel. For intermediate rib dimensions, interpolation can be used. The lowest design unit shear strength between panel buckling and screw failure governs.

The modified MCA procedure predicts diaphragm length can have an impact on strength and stiffness. Unit shear strength decreases as diaphragm length increases. This effect is largely due to the different screw patterns being used for end purlins and interior purlins. The screws at an end purlin contain twice as many screws as an interior purlin, thus contributing roughly twice the strength as the screw pattern for interior purlins. For diaphragms of smaller length, the contribution from end purlin screws is more pronounced. However, as the diaphragm length increases their contribution is diminished because diaphragm strength becomes largely dictated by the interior purlin screw pattern.

As mentioned previously, the modified MCA procedure was used to predict design unit shear strength governed by screw failure for diaphragm lengths ranging from 10 ft to 50 ft on one foot increments for all 168 diaphragm constructions. The design unit shear strength for all constructions and diaphragm lengths was normalized using Equation 3.1.

$$N = (V_x - V_{50})/(V_{10} - V_{50}) \tag{3.1}$$

where:

N = normalized value

V_x = design unit shear for diaphragm of length x, between 10 and 50 ft.

V_{10} = design unit shear for 10 ft long diaphragm

V_{50} = design unit shear for 50 ft long diaphragm

Normalized curves for all 168 diaphragm constructions were nearly identical, all falling in the envelope between the two curves shown in Figure 3.9. A curve was fit to the minimum normalized values curve and is given by Equation 3.2 which will serve as a reduction factor.

$$R = 12.5/L - 0.25 \tag{3.2}$$

where:

R = reduction factor for length

L = diaphragm length

By rearranging Equation 3.1, and substituting R for N, the unit shear strength of a diaphragm for any length between 10 and 50 ft can be calculated by Equation 3.3.

$$V_x = R \cdot (V_{10} - V_{50}) + V_{50} \quad (3.3)$$

RESULTS AND DISCUSSION

The design tables for diaphragms governed by screw failure for a cladding profile with 9 in. and 12 in. major rib spacing are presented in Table 3.2 and Table 3.3, respectively. These two tables also include the in-plane diaphragm stiffness which the designer will use to determine the effective shear modulus of the diaphragm assembly as will be explained later in this section. The reduction factor for diaphragm length has been tabulated in Table 3.4.

The use of a reduction factor for diaphragm length was validated by comparing the tests conducted by Lukens (1988) to predictions by the modified MCA procedure. Lukens conducted cantilever tests on 6, 8, 12, 16, and 20 ft diaphragm lengths with a 9 ft rafter spacing. The Grandrib 3 profile was used with a single #10 x 1" screw placed on one side of major ribs at interior purlins and #10 x 1" screws placed on both sides of the major rib at end purlins. Test results showed that the design unit shear strength decreased as diaphragm length increased as does the modified MCA procedure. A comparison of tested and predicted design unit shear strength using the modified MCA procedure is shown in Table 3.5. Although the modified MCA procedure slightly underestimated the design unit shear strength, it is important to note that the ratio of predicted to tested strength is similar for each diaphragm. Table 3.6 shows the decrease in tested and predicted values expressed as a ratio of the design strength of a 6-foot panel to the design strength of larger panel lengths. The percent decrease predicted by the modified MCA method is consistent with the percent decrease in test values. The decrease in unit shear strength from a 6-foot panel to a 20-foot panel is 35 percent, and the modified MCA method predicts a 34 percent decrease.

The shear strength values obtained from testing small-scale panels (typically 12 feet in length) may overestimate design unit shear strengths for diaphragms of longer length. There have not been any studies evaluating the effects of length on diaphragm strength when different screw patterns are used at end purlins and interior purlins. This issue needs to be resolved to determine at what length the additional end screws are considered negligible to the overall strength of the diaphragm. As a conservative measure, the design unit shear strength values for 10 and 50 ft diaphragms with an adjustment factor for diaphragm length are presented in tables for each construction.

Table 3.7, 3.8, and 3.9 provide the design unit shear strength governed by panel buckling for 9 in. major rib spacing profiles with 28, 29, and 30 gauge steel, respectively. Table 3.10, 3.11, and 3.12 provide the design unit shear strength governed by panel buckling for 12 in. major rib spacing profiles with 26, 28, and 29 gauge steel, respectively. The designer must compare design unit shear strength governed by screw failure and panel buckling and use the lower of the two.

To facilitate shear load transfer out of the diaphragm, sufficient screws must be placed at locations where load is transferred out of the diaphragm and into the shear walls. The required spacing of fasteners was determined by considering the contribution of screws on one side of the panel centerline. The panels without stitch screws do not require additional screws. Diaphragms with stitch screws spaced 24, 12, and 8 in. on center require structural screws spaced 12, 12, and 8 in. on center, respectively, at the location where load transfer to shear walls occur. Blocking is required between purlins to facilitate placement of these screws and prevent purlins from rolling.

Note, if cut panels are to be used, they should not be located in high shear zones or next to shear walls because the sheet capacity is reduced with smaller widths. If cut panels must be used they should be placed in low shear zones.

The stiffness values presented in Table 3.2 and Table 3.3 account for deflections resulting from shear strain of the steel, end panel warping, and fastener slip. The blocking can have significant effect on the effective shear modulus of the diaphragm. It is often placed at the location of shear walls to facilitate load transfer out of the diaphragm and into the shear wall. The designer may choose to use fully, partially, or unblocked diaphragms. For this reason, the equation that combines purlin and blocking connections with the in-plane stiffness of the diaphragm was left for the designer to compute. The equation to compute the effect shear modulus of the entire diaphragm assembly is given by Equation 3.4 as determined by Leflar (2008).

$$G_a = \frac{a/b}{\frac{a}{G'b} + \frac{2}{K_R}} \tag{3.4}$$

Where:

G_a = the effective shear modulus of the SCWF diaphragm assembly

G’ = the in-plane stiffness modulus tabulated in design tables

a = the frame spacing (width between rafters)

b = the diaphragm length (eave to ridge distance)

K_R = the total stiffness of all the rafter-purlin and rafter-shear block connections on a single rafter

The stiffness values of all of the rafter-purlin and rafter-shear block connections on a single rafter are computed by Equation 3.5 as determined by Leflar (2008).

$$K_R = N_p K_p + N_{sb} K_{sb} \tag{3.5}$$

where:

N_p = the number of purlins attached to a single rafter

K_p = the stiffness of one rafter-purlin connection

N_{sb} = the number of shear blocks attached to a single rafter

K_{sb} = the stiffness of one shear block connection

Table 3.13 provides stiffness values for purlin and shear block connections for use with Equation 3.5. Only the case with purlin and blocking oriented on edge and nailed to the top of the rafter had been experimentally determined by Leflar (2008). If purlins or shear blocks of different size, connection type, or significantly different specific gravity are used, the connection stiffness can be determined through testing methods established by Leflar (2008). Appendix C provides an example diaphragm design example to illustrate the use of design tables and accompanying formulas.

While the in-plane stiffness was shown to vary depending on diaphragm length and the configuration of the panel profile, only the minimum in-plane stiffness value was chosen from all constructions. Tabulating the smallest in-plane stiffness instead of the largest in-plane stiffness only had a small impact on the maximum forces distributed to the diaphragm, and it caused conservative over predictions of forces applied to critical frames.

In the design tables, the in-plane stiffness is constant for a diaphragm with a given steel gauge thickness and stitch screw spacing regardless of the size of field fasteners or size of stitch screws. This is because the flexibility of field fasteners are taken as a constant 0.2 in/kip, and the flexibility of No. 10 and 12 screws are identical since stitch screw flexibility is only dependent on the thickness of the steel. Table 3.14 shows the minimum and maximum in-plane stiffness calculated for the various combinations of major rib spacing, panel gauge, and spacing of stitch screws. The difference between maximum and minimum in-plane stiffness are minor. The largest difference between minimum and maximum in-plane stiffness for each stitch screw spacing is produced for a profile with 9 in. major spacing and 30 gauge steel. To assess the level of conservatism in critical frame forces and non-conservatism of maximum diaphragm shear force, a sensitivity analysis was conducted on 40 ft x 40 ft x 16 ft building and an 80 ft x 40 ft x 16 ft building. The building specifications and applied wind loads are listed below.

Sensitivity Analysis:

Building Properties:

Post/Wall Height	= 16 ft
Post Spacing	= 8 ft
Frame Base Fixity Factor	= 3/8
Roof Pitch	= 4:12
Roof Angle	= 18.43°
Effective Shear Modulus of Endwall	= 5.5 kips/in
Post-frame Stiffness	= 163 lb/in

Building Wind Pressures (no interior pressure assumed)

q_{ww}	= 5.5 psf
q_{wr}	= 0 psf
q_{lr}	=0 psf

q_{lw} = -4.4 psf

Eave load = 498 lbs

DAFI software (Bohnhoff, 1992) was used to determine load sharing between the frames and diaphragm. Table 3.15 tabulates the ratio of the effective shear modulus, G_a, using the minimum in-plane stiffness value versus the maximum in-plane stiffness for a building 40 ft x 40 ft x 16 ft building with a fully blocked diaphragm. The effective shear modulus is reduced 18% when the lowest in-plane stiffness is used instead of the highest in-plane stiffness for an unstitched diaphragm. However, the ratio of maximum shear in the diaphragm when using the minimum in-plane stiffness versus maximum in-plane diaphragm stiffness is relatively unchanged. The ratio of load carried by the critical frame when using the minimum in-plane stiffness versus maximum in-plane diaphragm stiffness is conservative. For an unstitched diaphragm, the load carried by the critical frame is 11% over predicted with use of the minimum in-plane stiffness.

The same sensitivity analysis was completed for fully blocked and unblocked diaphragms for the two different building geometries. Ratios of effective shear modulus, maximum load in the diaphragm, and load carried by the critical frame are presented in Table 3.16, 3.17, and 3.18. These comparisons show the use of the lowest in-plane stiffness instead of the highest in-plane stiffness will provide conservative estimates of load in the critical frame, while causing only a very small underestimation of maximum shear loads in the diaphragm. It should be noted that the tabulated in-plane stiffness is a theoretical value that has not been validated with tested diaphragms since the stiffness of concern in tested diaphragms usually includes the purlin and blocking connections.

SUMMARY AND CONCLUSIONS

The modified MCA procedure allows design values to be determined analytically, placing less reliance on expensive small-scale diaphragm tests; however, the model is too complex for use in the post-frame design industry. A simpler means to analytically derive design values was judged to be necessary. This study utilized the modified MCA procedure to develop design tables and accompanying formulas to determine strength and stiffness values for common constructions of SCWF used in the post-frame industry.

The modified MCA procedure provided insight into how changes in geometry, material properties and placement of screws can affect diaphragm performance. The model shows a decrease in unit shear strength as the diaphragm length (eave-to-ridge distance) increases. This effect is largely due to different screw configurations being used for end purlins and interior purlins. This effect was taken into account in the development of design tables as a conservative measure.

The modified MCA procedure also shows a significant increase in effective shear modulus when blocking is used. When small-scale diaphragms are tested in a laboratory, blocking is used, with additional screws placed through the cladding to ensure that applied loads are getting into and out of the diaphragm. However, it is common practice in the post-frame industry to use blocking only at end/shear wall locations, while leaving interior purlins unblocked. Effective shear modulus values derived from blocked diaphragms can yield effective shear modulus values higher than those from actual diaphragms being constructed without the use of blocking. Since post design is a function of eave deflections, overestimating diaphragm stiffness could lead to non-conservative estimates of critical post loads. The design tables developed in this study allow for the designer to use either an unblocked, partially blocked or fully blocked diaphragm and compute a more accurate effective shear modulus.

REFERENCES

American Forest & Paper Association, Inc., National Design Specification for Wood Construction, 2008 edition, Washington, DC.

American Forest & Paper Association, Inc., Wind & Seismic, Special Design Provisions for Wind and Seismic, 2008 edition, Washington, DC.

Anderson, G. A. 2011. Modification of the MCA Procedure for Strength and Stiffness of Diaphragms used in Post-Frame Construction. Frame Building News (June): 22-25.

ASAE. 2012. ANSI/ASAE Standard EP484.2. Diaphragm Design of Metal-Clad, Wood-Frame Rectangular Buildings. ASABE Standards, Engineering Practices, and Data. St. Joseph, Mich.: ASABE.

ASAE. 2014. ANSI/ASAE Standard EP558 Load Tests for Metal-Clad Wood-Frame Diaphragms. ASABE Standards, Engineering Practices, and Data, American Society of Agricultural and Biological Engineers, St. Joseph, MI.

Bohnhoff, D. R. 1992. Expanding diaphragm analysis for post-frame buildings. Applied Engineering in Agriculture 35(4):509-517

Bohnhoff, D.R. and G.D. Williams. 1999. Evaluation of metal-clad wood-frame diaphragm assembly tests. ASAE Paper No. 994205. ASAE, St. Joseph, Michigan.

Bohnhoff, D.R. P.A. Boor and G.A. Anderson. 1999. Thoughts on metal-clad wood-frame diaphragm action and full-scale building test. Presented at the 1999 ASAE/CSAE-SCGR Annual International Meeting, Toronto, Ontario, Canada. ASAE Paper No. 994204. ASAE, St Joseph, MI.

Lukens, A.D. 1988. Shear strength and stiffnesses of steel clad, timber-framed diaphragm panels, and a procedure for applying diaphragm theory to post-frame building design. M.S. thesis, Iowa State University, Ames, Iowa.

Luttrell, L. D. 1992. Diaphragm Design Manual. Steel Deck Institute, Second Edition, Fox River Grove, IL.

TABLES

Table 3.1: Properties of common corrugated steel panels

Panel Name	Major Rib Pitch (in.)	Cladding Depth (in.)	Top Corrugation Width (in.)	Bottom Corrugation Width (in.)	Gauge or Thickness	Yield/Ultimate Strength (ksi)
Pro Panel II	9	0.625	0.6875	1.25-2.5	30 or 26 gauge	----
StrongClad	9	0.625	----	----	29 gauge	----
ProClad	9	0.625	----	----	29 gauge	----
MP Panel	9	0.75	----	----	29 gauge	----
Grandrib 3	9	0.75	0.375	1.75	0.014 in.	80/82
Classic Rib	9	0.75	----	1.75	----	----
CenturyDrain	9	0.75	----	----	0.014 in.	111/116
EPS Corrugations	----	----	----	----	0.0142 in.	80/105
Channel Drain 2000	9	0.75	----	----	29 gauge	87/92.1
Midwest Manufacturing/ Distributers	12	0.969	0.5	2.5	0.015 in.	80/82
Wick panel	12	0.75	0.5	1.34	0.015 in.	80/82

Table 3.2: Design unit shear strength governed by screws for 9 in. major rib spacing

Purlin Spacing = 2 ft.						Minimum Purlin S.G. = 0.42						Major Rib Bottom Width Range = 1 - 2.5 in.			
Major Rib Spacing = 9 in.						Major Rib Height Range = 0.625 - 1.0 in.						Major Rib Top Width Range = 0.25 - 0.75 in.			
Min. Yield/ Ultimate Strength	**Gauge**	**Field Screws into Flats**	**Screws into Panel Overlaps**	**NO Screws into Panel Overlaps**			**Spacing of screws into the panel overlaps (in.)**								
							24			**12**			**8**		
				V_{10}	**V_{50}**	**G'**	**V_{10}**	**V_{50}**	**G'**	**V_{10}**	**V_{50}**	**G'**	**V_{10}**	**V_{50}**	**G'**
(ksi)				**(plf)**	**(plf)**	**(kips/in)**	**(plf)**	**(plf)**	**(kips/in)**	**(plf)**	**(plf)**	**(kips/in)**	**(plf)**	**(plf)**	**(kips/in)**
80/82	28 (.0149")	#9-1"	None	102	85	2.8	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	130	102	19	167	142	27	199	178	32
			#12 Stitch	----	----	----	136	107	19	176	152	27	211	190	32
		#10-1"	None	108	89	2.8	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	134	105	19	172	146	27	205	182	32
			#12 Stitch	----	----	----	140	111	19	182	156	27	217	195	32
		#10-1.5"	None	129	107	2.8	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	151	117	19	190	158	27	225	197	32
			#12 Stitch	----	----	----	157	123	19	200	169	27	238	211	32
		#12-1.5"	None	165	136	2.8	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	179	137	19	219	179	27	256	219	32
			#12 Stitch	----	----	----	185	142	19	230	190	27	271	234	32
	29 (.0135")	#9-1"	None	102	85	2.8	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	123	96	18	156	131	25	186	163	29
			#12 Stitch	----	----	----	128	101	18	165	140	25	197	175	29
		#10-1"	None	108	89	2.8	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	128	99	18	161	135	25	191	167	29
			#12 Stitch	----	----	----	133	104	18	170	143	25	202	179	29
		#10-1.5"	None	127	105	2.8	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	142	109	18	176	145	25	208	179	29
			#12 Stitch	----	----	----	148	114	18	185	155	25	220	192	29
		#12-1.5"	None	155	128	2.8	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	164	125	18	199	162	25	232	197	29
			#12 Stitch	----	----	----	170	130	18	209	171	25	245	210	29
	30 (0.0120")	#9-1"	None	102	85	2.8	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	116	89	16	145	120	22	171	148	26
			#12 Stitch	----	----	----	121	94	16	152	127	22	181	158	26
		#10-1"	None	108	89	2.8	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	121	93	16	149	123	22	176	152	26
			#12 Stitch	----	----	----	125	97	16	157	131	22	186	162	26
		#10-1.5"	None	112	93	2.8	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	124	95	16	153	126	22	180	154	26
			#12 Stitch	----	----	----	129	99	16	161	133	22	190	165	26
		#12-1.5"	None	130	107	2.8	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	138	105	16	167	136	22	194	165	26
			#12 Stitch	----	----	----	142	109	16	175	143	22	205	176	26

* Safety factor of 2.5 has been applied in all cases

Table 3.3: Design unit shear strength governed by screws for 12 in. major rib spacing

Purlin Spacing = 2 ft.				Minimum Purlin S.G. = 0.42						Major Rib Bottom Width Range = 1 - 2.5 in.					
Major Rib Spacing = 12 in.				Major Rib Height Range = 0.625 - 1.0 in.						Major Rib Top Width Range = 0.25 - 0.75 in.					
Min. Yield/ Ultimate Strength	**Gauge**	**Field Screws into Flats**	**Screws into Panel Overlaps**	**NO Screws into Panel Overlaps**			**Spacing of screws into the panel overlaps (in.)**								
							24			**12**			**8**		
				V_{10}	**V_{50}**	**G'**	**V_{10}**	**V_{50}**	**G'**	**V_{10}**	**V_{50}**	**G'**	**V_{10}**	**V_{50}**	**G'**
(ksi)				**(plf)**	**(plf)**	**(kips/in)**	**(plf)**	**(plf)**	**(kips/in)**	**(plf)**	**(plf)**	**(kips/in)**	**(plf)**	**(plf)**	**(kips/in)**
50/52	26 (.0179")	#9-1"	None	84	71	2.4	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	124	101	21	164	147	32	194	181	38
			#12 Stitch	----	----	----	131	108	21	174	157	32	204	192	38
		#10-1"	None	89	75	2.4	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	128	104	21	169	150	32	201	186	38
			#12 Stitch	----	----	----	135	111	21	179	161	32	211	198	38
		#10-1.5"	None	89	75	2.4	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	128	104	21	169	150	32	201	186	38
			#12 Stitch	----	----	----	135	111	21	179	161	32	211	198	38
		#12-1.5"	None	137	117	2.4	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	166	131	21	213	183	32	254	228	38
			#12 Stitch	----	----	----	174	138	21	226	195	32	269	244	38
80/82	28 (.0149")	#9-1"	None	85	73	2.4	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	111	89	19	146	127	27	173	158	32
			#12 Stitch	----	----	----	117	95	19	154	136	27	183	169	32
		#10-1"	None	90	77	2.4	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	115	92	19	150	130	27	179	162	32
			#12 Stitch	----	----	----	121	97	19	159	139	27	189	173	32
		#10-1.5"	None	108	92	2.4	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	129	101	19	165	141	27	197	176	32
			#12 Stitch	----	----	----	135	107	19	174	151	27	208	188	32
		#12-1.5"	None	137	117	2.4	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	151	117	19	189	158	27	223	196	32
			#12 Stitch	----	----	----	157	123	19	199	168	27	236	209	32
	29 (.0135")	#9-1"	None	85	73	2.4	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	105	83	18	136	117	25	162	146	29
			#12 Stitch	----	----	----	110	88	18	144	125	25	171	156	29
		#10-1"	None	90	77	2.4	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	109	86	18	140	120	25	167	149	29
			#12 Stitch	----	----	----	114	91	18	148	128	25	176	160	29
		#10-1.5"	None	105	90	2.4	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	120	94	18	153	129	25	181	160	29
			#12 Stitch	----	----	----	126	99	18	161	137	25	192	171	29
		#12-1.5"	None	129	110	2.4	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	138	107	18	172	142	25	202	175	29
			#12 Stitch	----	----	----	144	112	18	180	151	25	213	187	29

* Safety factor of 2.5 has been applied in all cases

Table 3.4: Reduction factor for diaphragm length

Length (ft)	R[a]	Length (ft)	R[a]
10	1.00	31	0.15
11	0.89	32	0.14
12	0.79	33	0.13
13	0.71	34	0.12
14	0.64	35	0.11
15	0.58	36	0.10
16	0.53	37	0.09
17	0.49	38	0.08
18	0.44	39	0.07
19	0.41	40	0.06
20	0.38	41	0.05
21	0.35	42	0.05
22	0.32	43	0.04
23	0.29	44	0.03
24	0.27	45	0.03
25	0.25	46	0.02
26	0.23	47	0.02
27	0.21	48	0.01
28	0.20	49	0.01
29	0.18	50	0.00
30	0.17		

[a] Reduction factor equation: $R = 12.5/L - 0.25$

Table 3.5: Effective of length on diaphragm strength

Diaphragm Length (ft)	Sample Size	Average Test Value (lb/ft)	Predicted per Modified MCA Procedure (lb/ft)	Ratio: Predicted/Test
6	1	143	124	0.86
8	2	120	109	0.91
12	2	110	94	0.86
16	2	99	86	0.88
20	2	93	82	0.88
			Average =	0.88
			COV =	2%

Table 3.6: Normalized design unit shear strength

Diaphragm Length (ft)	Normalized Strength for Tested Panel	Normalized Strength for Modified MCA Prediction
6	1.00	1.00
8	0.84	0.88
12	0.77	0.76
16	0.69	0.70
20	0.65	0.66

Table 3.7: Buckling design unit shear strength (lb/ft) for 28 gauge, 9 in. major rib spacing

Rib Height (in.)	Top Width of Rib (in.)	Bottom Width (in)						
		1	1.25	1.5	1.75	2	2.25	2.5
0.625	0.25	147	152	157	163	169	175	182
0.625	0.75	185	187	191	196	201	206	211
0.75	0.25	201	206	213	220	227	235	243
0.75	0.75	250	253	258	263	269	275	282
0.875	0.25	263	269	276	284	293	302	312
0.875	0.75	324	328	333	339	346	353	362
1	0.25	333	340	348	357	367	377	388
1	0.75	407	411	417	424	432	440	449

* Safety factor of 2.5 has been applied to all cases

Table 3.8: Buckling design unit shear strength (lb/ft) for 29 gauge, 9 in. major rib spacing

Rib Height (in.)	Top Width of Rib (in.)	Bottom Width of Rib (in)						
		1	1.25	1.5	1.75	2	2.25	2.5
0.625	0.25	124	128	132	137	143	148	154
0.625	0.75	155	157	161	165	169	174	178
0.75	0.25	169	174	180	186	192	199	205
0.75	0.75	210	213	217	222	227	232	238
0.875	0.25	222	227	233	240	248	256	264
0.875	0.75	273	276	281	286	292	299	306
1	0.25	281	287	294	302	310	319	329
1	0.75	343	347	352	358	364	372	380

* Safety factor of 2.5 has been applied to all cases

Table 3.9: Buckling design unit shear strength (lb/ft) for 30 gauge, 9 in. major rib spacing.

Rib Height (in.)	Top Width of Rib (in.)	Bottom Width of Rib (in)						
		1	1.25	1.5	1.75	2	2.25	2.5
0.625	0.25	101	104	108	112	117	121	126
0.625	0.75	126	128	131	134	138	142	146
0.75	0.25	138	142	147	152	157	163	168
0.75	0.75	171	174	177	181	185	190	195
0.875	0.25	181	186	191	197	203	209	216
0.875	0.75	223	225	229	234	239	244	250
1	0.25	230	235	241	247	254	262	270
1	0.75	280	284	288	293	298	305	311

* Safety factor of 2.5 has been applied to all cases

Table 3.10: Buckling design unit shear strength (lb/ft) for 26 gauge, 12 in. major rib spacing

Rib Height (in.)	Top Width of Rib (in.)	Bottom Width of Rib (in.)						
		1	1.25	1.5	1.75	2	2.25	2.5
0.625	0.25	174	180	186	192	199	205	212
0.625	0.75	222	225	229	233	238	244	249
0.75	0.25	238	244	251	259	267	275	284
0.75	0.75	300	303	308	313	319	325	332
0.875	0.25	311	318	326	334	344	353	364
0.875	0.75	387	391	396	402	409	417	425
1	0.25	394	401	410	419	430	441	452
1	0.75	485	489	495	501	509	518	527

* Safety factor of 2.5 has been applied to all cases

Table 3.11: Buckling design unit shear strength (lb/ft) for 28 gauge, 12 in. major rib spacing

Rib Height (in.)	Top Width of Rib (in.)	Bottom Width of Rib (in.)						
		1	1.25	1.5	1.75	2	2.25	2.5
0.625	0.25	119	123	127	132	137	142	147
0.625	0.75	150	152	155	159	162	167	171
0.75	0.25	163	168	173	178	184	190	197
0.75	0.75	203	206	209	213	218	223	229
0.875	0.25	214	219	224	231	238	245	253
0.875	0.75	264	267	271	276	281	287	293
1	0.25	271	277	283	290	298	306	315
1	0.75	332	335	340	345	351	358	365

* Safety factor of 2.5 has been applied to all cases

Table 3.12: Buckling design unit shear strength (lb/ft) for 29 gauge, 12 in. major rib spacing

Rib Height (in.)	Top Width of Rib (in.)	Bottom Width of Rib (in.)						
		1	1.25	1.5	1.75	2	2.25	2.5
0.625	0.25	100	104	107	111	116	120	124
0.625	0.75	126	128	130	133	137	141	144
0.75	0.25	137	141	146	150	156	161	166
0.75	0.75	171	173	176	180	184	188	193
0.875	0.25	180	185	190	195	201	207	214
0.875	0.75	222	225	228	232	237	242	248
1	0.25	229	234	239	245	252	259	267
1	0.75	280	283	286	291	296	302	309

* Safety factor of 2.5 has been applied to all cases

Table 3.13: Strength and stiffness of rafter-purlin and blocking connections

Member	Connection	Size	Orientation	Location	Specific Gravity	Stiffness (kips/in)
Purlin	1-60d hardened steel ring shank nail	2x4	on-edge	on top of rafter	0.42	1.0
Blocking	2-60d hardened steel ring shank nails	2x4	on-edge	on top of rafter	0.42	10.0

Table 3.14: Comparison of minimum and maximum calculated in-plane stiffness

Major Rib Spacing (in.)	Gauge	Stitch Screw Spacing (in.)	Minimum In-Plane stiffness (kips/in)	Maximum In-Plane Stiffness (kips/in)	Difference (max-min)
9	28 (0.0149 in.)	none	2.8	3.5	0.7
		24	19.6	22.0	2.4
		12	27.0	31.8	4.8
		8	31.8	38.7	6.9
	29 (0.0135 in.)	none	2.8	3.5	0.7
		24	18.1	20.7	2.6
		12	24.7	29.7	5
		8	28.8	36.0	7.2
	30 (0.0120 in.)	none	2.8	3.5	0.7
		24	16.4	19.2	2.8
		12	22.0	27.3	5.3
		8	25.5	32.8	7.3
12	26 (0.0179 in.)	none	2.4	3.1	0.7
		24	22.4	24.3	1.9
		12	32.0	36.0	4
		8	38.4	44.5	6.1
	28 (0.0149 in.)	none	2.4	3.1	0.7
		24	19.6	21.7	2.1
		12	27.3	31.7	4.4
		8	32.4	38.8	6.4
	29 (0.0135 in.)	none	2.4	3.1	0.7
		24	18.1	20.4	2.3
		12	24.9	29.6	4.7
		8	29.3	36.0	6.7

Table 3.15: 40'W x 40'L x 16'H building – blocked diaphragm

	On Center Spacing of Stitch Screws			
	none	24 in.	12 in.	8 in.
G_a Ratio	0.82	0.92	0.90	0.90
Max Shear Load in Diaphragm Ratio	1.00	1.00	1.00	1.00
Critical Frame Load Ratio	1.11	1.02	1.02	1.02

Table 3.16: 40’W x 40’L x 16’H building – unblocked diaphragm

	On Center Spacing of Stitch Screws			
	none	24 in.	12 in.	8 in.
G_a Ratio	0.91	0.98	0.98	0.98
Max Shear Load in Diaphragm Ratio	1.00	1.00	1.00	1.00
Critical Frame Load Ratio	1.06	1.01	1.01	1.01

Table 3.17: 80’W x 40’L x 16’H building – blocked diaphragm

	On Center Spacing of Stitch Screws			
	none	24 in.	12 in.	8 in.
$80G_a$ Ratio	0.82	0.92	0.90	0.90
Max Shear Load in Diaphragm Ratio	0.98	1.00	1.00	1.00
Critical Frame Load Ratio	1.13	1.03	1.04	1.04

Table 3.18: 80’W x 40’L x 16’H building – unblocked diaphragm

	On Center Spacing of Stitch Screws			
	none	24 in.	12 in.	8 in.
G_a Ratio	0.91	0.98	0.98	0.98
Max Shear Load in Diaphragm Ratio	0.99	1.00	1.00	1.00
Critical Frame Load Ratio	1.06	1.01	1.01	1.01

FIGURES

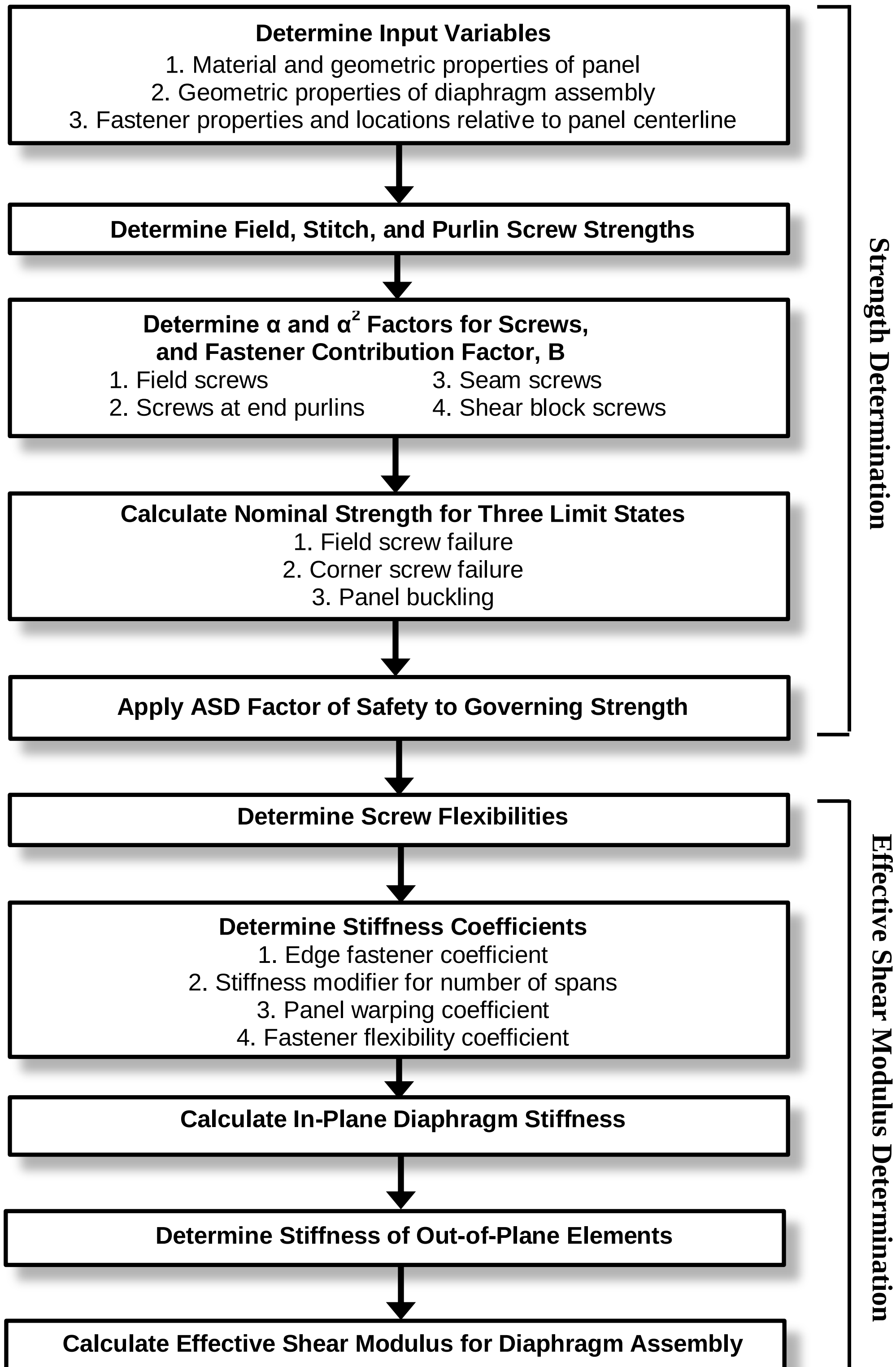


Figure 3.1: Flowchart for modified MCA procedure

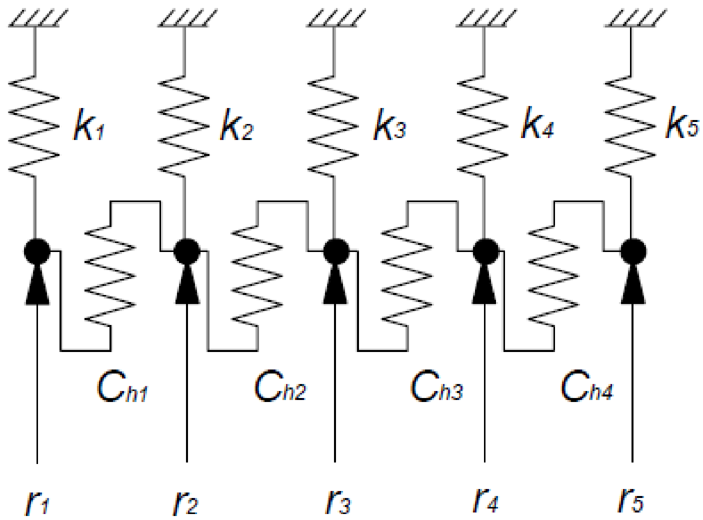


Figure 3.2: Two-element spring analog used for interaction between frames and diaphragms

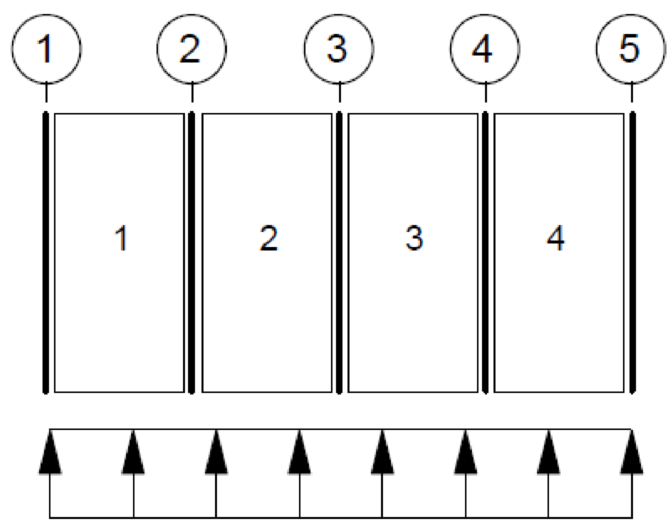


Figure 3.3: Lateral loads applied to building

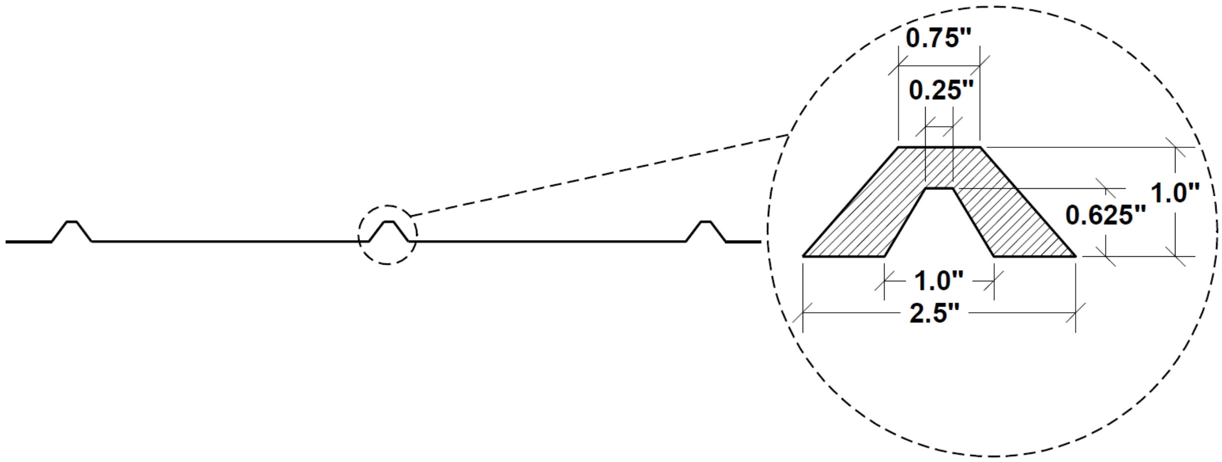


Figure 3.4: Range of major rib dimensions considered in the analysis

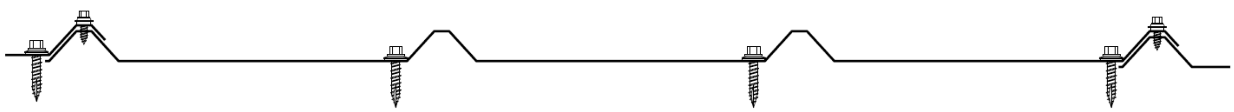


Figure 3.5: Fastener locations at interior purlins for diaphragms with stitch screws

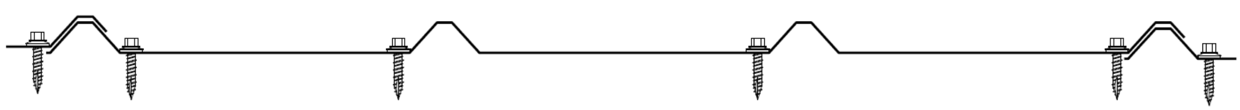


Figure 3.6: Fastener location at interior purlins for diaphragm without stitch screws

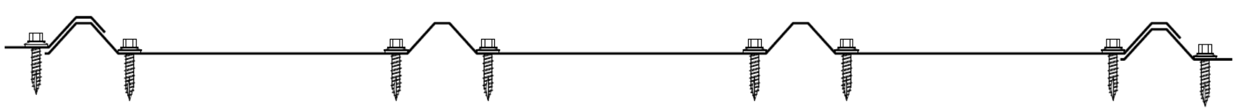
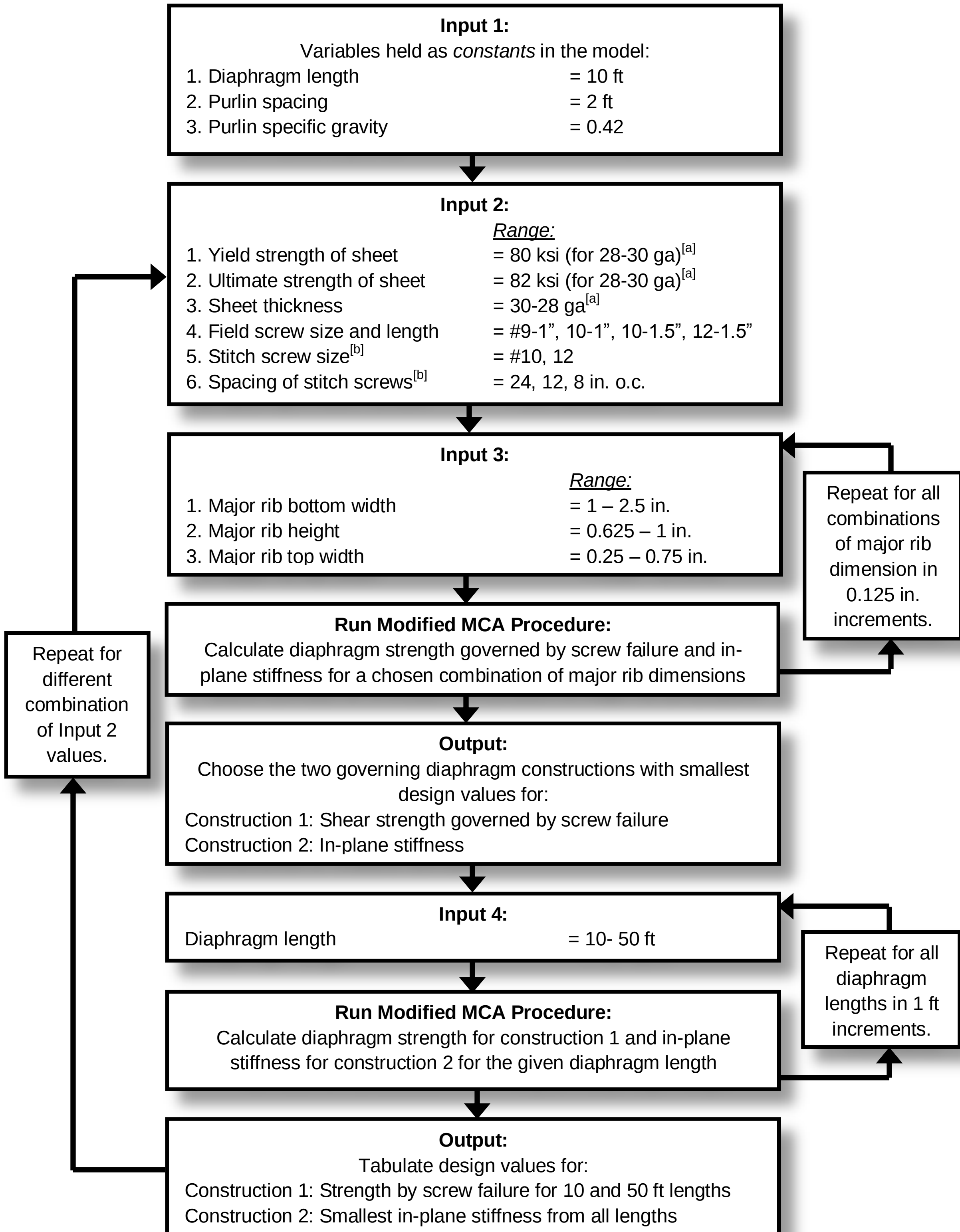


Figure 3.7: Fastener location at end purlins for all diaphragms



[a] For 12 in. major rib spacing the following values were used: yield strength = 50ksi (for 26 ga), ultimate strength = 52 ksi (for 26 ga), and steel gauge thickness = 29, 28, and 26 ga.
[b] Not applicable to non-stitched diaphragms.

Figure 3.8: Flowchart for development of tabulated design values

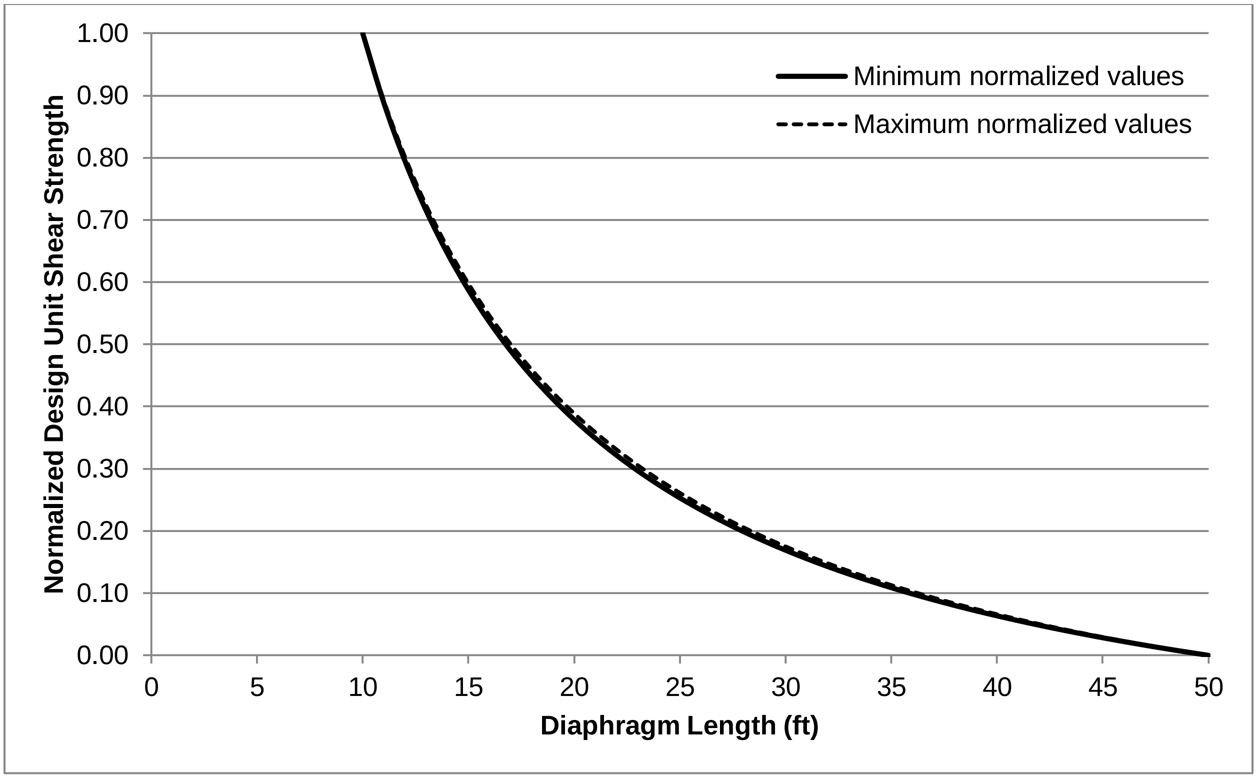


Figure 3.9: Normalized design unit shear strength

CHAPTER 4

SUMMARY AND CONCLUSIONS

The results of preceding chapters demonstrated that the modified MCA procedure provides an accurate means of determining strength and stiffness of SCWF diaphragms used in post-frame buildings. Minor changes were made to the model, producing better results between predicted and tested diaphragms. Predictions of failure due to panel buckling were overestimated using the moment of inertia from panel manufacturers. The moment of inertia was reduced by considering the major rib as a stiffener and computing an effective width of the flats between major ribs using Equation 2.20. The use of Equation 2.3 is recommended when using elevated sidelap structural fasteners as use of Equation 2.4 can significantly over predicted design unit shear strength governed by screw failure.

With the amendments discussed, the study herein provided independent validation of the modified MCA procedure by comparing predicted unit shear strength and effective shear modulus values to those obtained from diaphragm tests. The ratio of predicted to tested design unit shear strength averaged 0.97 and the ratio of predicted to tested effective shear modulus averaged 1.07. The predicted design unit shear strengths are in good agreement with average test values, with only slight conservatism in most cases. While the effective shear modulus had a larger coefficient of variation, predicted values are within reason when one considers that some diaphragm types had only one replication, and large variability of stiffness results is typical from testing. For shear walls, the ratio of predicted to tested design unit shear strength averaged 0.81 and the ratio of predicted to tested effective shear modulus also averaged 0.81. The modified MCA procedure does not appear to be as accurate for shear walls compared to diaphragms, however the predictions are conservative.

The modified MCA procedure accurately predicted the percent decrease in design unit shear strength when compared to cantilever tests conducted on 6, 8, 12, 16, and 20-foot diaphragm lengths. As most diaphragm tests are usually 12 ft in length, this highlights a potential issue with using design values obtained from test panels. This length effect has not been the subject of research, so as a conservative measure adjustment formulas were provided to determine reduced design unit shears strength for diaphragms between 10 and 50 ft long.

Design tables were provided to simplify the process in calculating strength and stiffness of common SCWF diaphragm constructions used in the post-frame industry. With 168 different constructions tabulated, designers will have a more flexible approach to diaphragm design. A variety field screws, lumber species, stitch screws, and cladding profiles can be used when designing diaphragms, provided that the minimum requirements specified in the tables are met. The database for diaphragm design values is limited, and the addition of these design tables will provide engineers with a variety of diaphragm construction to meet a broad range of load demands. Design tables will allow safer, more economic post-frame building designs with simplicity in design procedure.

RECOMMENDATIONS FOR FURTHER RESEARCH

Additional testing needs to be conducted to determine the stiffness of common connections to expand Table 3.13. Testing should be complete for the most common types of purlin connections such as recessed purlin without steel straps, recessed purlins with steel straps, and purlins oriented on-flat. Additionally, the girt-to-post connection stiffness should be determined for use with the modified MCA procedure, as it may provide more accurate effective shear modulus predictions for shear walls. However, further research may be required to determine the effect of loads applied perpendicular to the corrugations and the influence of embedded posts on end/shear wall strength and stiffness. Also, the decrease in unit shear strength with increases in diaphragm length should be further investigated to determine at what length the additional screws at the end purlins become negligible.

APPENDIX A: DERIVATION OF STRENGTH FORMULAS

The derivation of the three limit state equations define by the MCA is explained to provide the reader with a better understanding of formulas used in the modified MCA procedure, their application, and limitations.

Limit State 1: Field Screws

Consider a single panel from the interior of the diaphragm as shown in Figure A.1. The panel consists of field screws into the purlins and seam screws through the panel sidelaps. The contribution from each screw in transferring shear depends on its distance from the centerline of the panel. The shear force of each screw is conservatively assumed to vary linearly from zero at the panel centerline to its capacity at the panel edge. The fastener forces at purlin locations can be replaced by equivalent moment couples M_e and M_p for the two end purlins and interior purlins, respectively, as shown in Figure A.1. M_p is computed by multiplying the forces F_p and Q_f, by their respective lever arms, x_p. The same is done for the two end purlins to compute M_e. Summing the moments about the lower left corner of the panel in Figure A.1 produces the equilibrium equation:

$$\left(\frac{P_u w}{L}\right)L = 2M_e + n_p M_p + n_s Q_s w \qquad \text{(A.1)}$$

where:

P_u = ultimate shear load

w = panel width

L = panel length

M_e = end purlin couple

M_p = interior purlin couple

n_p = number of interior purlins (2 shown)

n_s = number of seam screws at one overlap (3 shown)

Q_s = shear strength of stitch screw

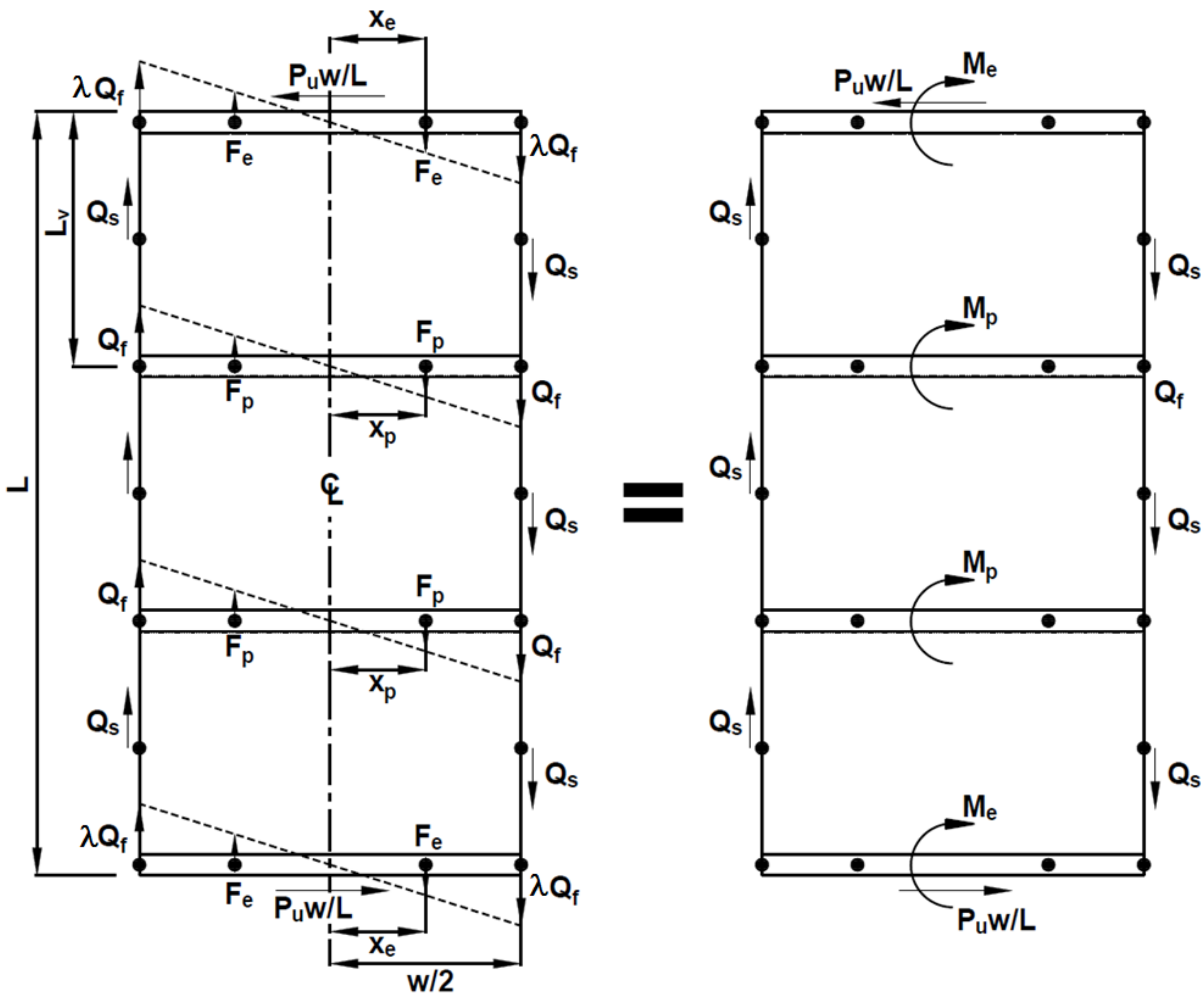


Figure A.1: Shear forces at fastener locations and equivalent moment couples

The field screw capacity, Q_f limits the development of S_u . Considering a linear variation in shear forces, the shear force of each screw, F_p, at interior purlins can be computed as

$$F_p = Q_f \left(\frac{x_p}{w/2}\right) \tag{A.2}$$

Summing the moments produced by each fastener, and substituting Equation A.2 into the following expression results in Equation A.3

$$M_p = \Sigma F_p x_p = \frac{2}{w} Q_f \, \Sigma x_p^2 \tag{A.3}$$

The same process applies to M_e and is given by Equation A.4

$$F_e = Q_f \left(\frac{x_e}{w/2}\right)$$

$$M_e = \Sigma F_e x_e = \frac{2}{w} Q_f \, \Sigma x_e^2 \tag{A.4}$$

where:

Q_f = ultimate shear strength of field screw fastener

x_p = interior purlin field screw location relative to panel centerline

x_e = end purlin field screw location relative to panel centerline

The panel corners are subject to eccentric forces from the resultant of shear forces acting perpendicular and parallel to the corrugations at the panel edges as shown in Figure A.2.

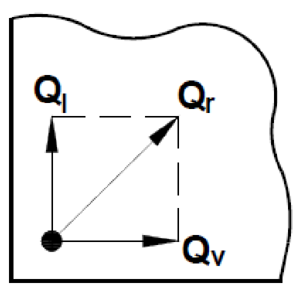


Figure A.2: Corner screw resultant force

Two of the panel corners are subject to compression. The edge-most corrugation at compression corners may not be able to resist the full eccentric compression force Q_r, thus the structural fastener at that location is reduced to λQ_f where the reduction factor, λ, is given by Equation A.5

$\lambda \quad = 1 - \frac{d_d L_v}{240\sqrt{t}} \geq 0.7$ (for steel) (A.5)

where:

d_d = panel depth (in.)

L_v = purlin spacing (ft.)

t = panel thickness (in.)

The reduction factor, λ, is conservatively applied to all four corners screws, although only the two compression corners are effected. Assuming there is only one structural screw at each corner, Equation A.4 can be rewritten as Equation A.6 shown below.

$$M_e = \frac{2}{w} Q_f \, \Sigma x_e^2 - 2(Q_f - \lambda Q_f)w/2$$

$$= \frac{2}{w} Q_f \, \Sigma x_e^2 - Q_f(1-\lambda)w \qquad \text{(A.6)}$$

Substituting Equations A.3 and A.6 into Equation A.1, yields

$$\left(\frac{P_u w}{L}\right)L = 2\left(\frac{2}{w} Q_f \, \Sigma x_e^2 - Q_f(1-\lambda)w\right) + n_p\left(\frac{2}{w} Q_f \, \Sigma x_p^2\right) + n_s Q_s w$$

Rearranging in terms of average shear per unit length gives Equation A.7 which is the equation for the limit state of field screw failure.

$$S_u = \frac{P_u}{L} = \left[2\left(2\Sigma\frac{x_e^2}{w^2} - (1-\lambda)\right) + n_p\left(2\Sigma\frac{x_p^2}{w^2}\right) + n_s\frac{Q_s}{Q_f}\right]\frac{Q_f}{L}$$

$$S_u = [2(\lambda - 1) + B]\frac{Q_f}{L} \qquad \text{(A.7)}$$

where:

$B = n_s\alpha_s + 2n_p\alpha_p^2 + 4\Sigma\alpha_e^2$

And:

$\alpha_e^2 = \Sigma \frac{x_e^2}{w^2}$

$\alpha_p^2 = \Sigma \frac{x_p^2}{w^2}$

$\alpha_s = Q_s/Q_f$

Limit State 2: Corner Fasteners

Figure A.2 shows a corner fastener which is subject to shear forces in orthogonal directions. The resultant force on the fastener is given by Equation A.8

$$Q_r = \sqrt{Q_l^2 + Q_v^2} \text{ which also equivalent to } Q_r^2 = Q_l^2 + Q_v^2 \tag{A.8}$$

The shear force along the end purlins is shared equally by the screws. Thus the corner screw experiences a shear force in the direction parallel to the end purlins computed as

$$Q_v = \frac{S_u}{N}$$

Where N = number of fasteners per unit length along the width of one panel, w

The reduction factor, λ, is not applied to the screw for the force parallel to corrugations. Thus Q_l is computed as

$$Q_l = \frac{S_u L}{B}$$

By letting Q_r approach the fastener capacity, Q_f, Equation A.8 becomes

$$Q_r^2 = S_u^2 \left(\frac{L^2}{B^2} + \frac{1}{N^2}\right)$$

By rearranging this expression, Equation A.9 is developed which gives the diaphragm capacity limited by the corner fasteners

$$S_u = Q_f \sqrt{\frac{N^2 B^2}{L^2 N^2 + B^2}} \tag{A.9}$$

Limit State 3: Panel buckling

Buckling of the cladding can limit the capacity of the diaphragm, controlling over screw failure.

The equation for buckling capacity is given by Equation A.10.

$$S_u = \frac{3250}{L_v^2} \cdot \left(\frac{I_x^3 t^3 p}{s}\right)^{1/4} \tag{A.10}$$

APPENDIX B: MODIFIED MCA PROCEDURE EXAMPLE

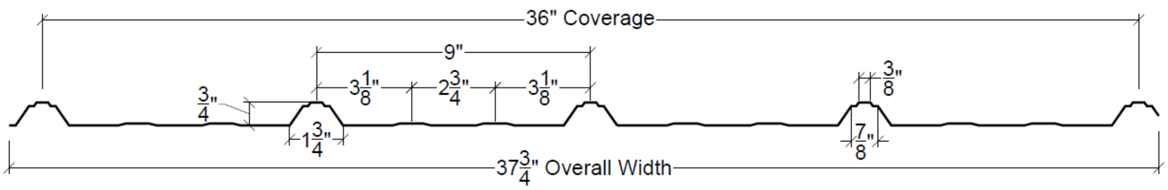
Example calculations used in validating the modified MCA procedure are provided below. This example is for Diaphragm Type 2.

Diaphragm Strength Prediction

Determine input variables for strength prediction

Corrugated panel geometric and material properties:

Panel Profile



Panel width		$w := 36$	in
Major rib corrugation pitch		$p := 9$	in
Height of major rib		$d_d := .75$	in
Top width of major rib		$w_t := .375$	in
Bottom width of major rib		$w_c := 1.75$	in
Base metal thickness		$t := .0142$	in
Yield strength		$F_y := 80$	ksi
Ultimate strength		$F_u := 82$	ksi
Elastic modulus		$E := 29500$	ksi
Moment of inertia (using reduced section per AISI equation)		$I_x := 0.00412$	$\frac{in^4}{ft}$
Number of corrugations per cladding width	$n := \frac{w}{p}$	$n = 4$	in
Corrugation flat width	$w_b := p - w_c$	$w_b = 7.25$	in

Web width $\quad w_w := \sqrt{d_d^{\,2} + \left(\frac{w_c - w_t}{2}\right)^2} \quad w_w = 1.017 \quad$ in

Flat width to form one pitch $\quad s := w_b + w_t + 2 \cdot w_w + \frac{1}{8} \quad s = 9.785 \quad$ in

Diaphragm properties:

Rafter spacing $\quad a := 7.89 \quad$ ft

Diaphragm panel length $\quad b := 12 \quad$ ft

Sloped roof length parallel to corrugations $\quad L := b = 12 \quad$ ft

Purlin spacing $\quad L_v := 2 \quad$ ft

Purling specific gravity $\quad G := 0.44$

Number of shear blocks per rafter $\quad n_{rsb} := 6$

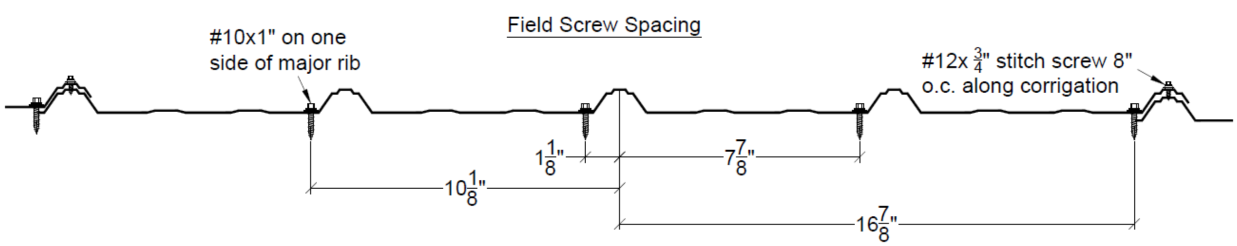
Number of spans $\quad n_{spans} := \frac{L}{L_v} \quad n_{spans} = 6$

Number of interior purlins $\quad n_{ip} := n_{spans} - 1 \quad n_{ip} = 5$

Number of cladding sheets between rafters

$$n_{shts} := \frac{a \cdot 12}{w} \qquad n_{shts} = 2.63$$

Screw properties and locations relative to panel centerline:

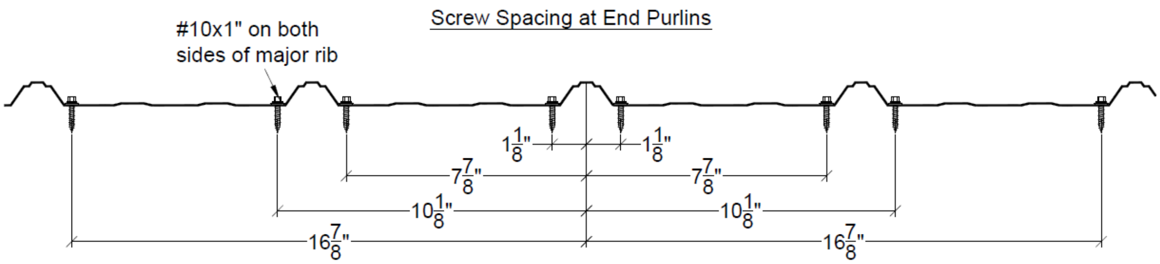


Interior purlin field screw location relative to panel centerline

$xpa_1 := 10.125 \quad xpa_2 := 1.125 \quad xpa_3 := 7.875 \quad xpa_4 := 16.875 \quad$ in

Number of screws per shear block $\quad n_{ssb} := 2$

Shear block screw location relative to panel centerline $\quad xpb_1 := 16.875 \quad$ in



Number of end purlin field screws across sheet width $\quad N_e := 8$

End purlin screw location relative to panel centerline

$xe_1 := 16.875 \quad xe_2 := 10.125 \quad xe_3 := 7.875 \quad xe_4 := 1.125$

$xe_5 := 1.125 \quad xe_6 := 7.875 \quad xe_7 := 10.125 \quad xe_8 := 16.875 \quad$ in

Number of elevated sidelap structural srews in one overlap $\quad n_{purlin} := 0$

Number of stitch screws in one overlap $\quad n_{stitch} := 17$

Total seam screws in one overlap $\quad n_s := n_{purlin} + n_{stitch} \quad n_s = 17$

Number of phantom intrerior purlins per average sheet between rafters $\quad n_{pip} := \dfrac{n_{rsb} \cdot n_{ssb}}{n_{shts}} \quad n_{pip} = 4.563$

Number of screws per unit length at end of panel $\quad N := \dfrac{N_e}{\frac{w}{12}} \quad N = 2.667 \quad \dfrac{\text{screws}}{\text{foot}}$

Diameter of field screw

$$\text{diameter(size)} := \begin{cases} 0.165 & \text{if size} = 8 \\ 0.177 & \text{if size} = 9 \\ 0.187 & \text{if size} = 10 \\ 0.211 & \text{if size} = 12 \\ 0.248 & \text{if size} = 14 \\ 0 & \text{otherwise} \end{cases}$$

Field screw diameter $\quad size_{fs} := 10 \quad d_{fs} := \text{diameter}(size_{fs}) \quad d_{fs} = 0.187 \quad$ in

Length of field screw $\quad L_f := 1.0 \quad$ in

Stitch screw diameter $\quad size_{ss} := 12 \quad d_{ss} := \text{diameter}(size_{ss}) \quad d_{ss} = 0.211 \quad$ in

Determine the strength of field, stitch, and elevated sidelap structural screws

Field screw strength:

Nominal penetration expressed as number of screw diameters

$$num\_dia := \frac{L_f}{d_{fs}} \qquad num\_dia = 5.35$$

Screw strength reduction factor

$$R := \begin{cases} \min\left(\frac{num\_dia}{6.4}, 1.0\right) & \text{if } num\_dia \geq 4 \\ 0 & \text{otherwise} \end{cases} \qquad R = 0.836$$

Field screws strength if ***steel*** controls

$$Q_{f\_stl} := \begin{cases} (2.22 \cdot d_{fs} \cdot F_u \cdot t) & \text{if } d_{fs} \leq 0.187 \\ \text{if } d_{fs} > 0.187 & \\ \quad \begin{cases} 1.25 \cdot F_y \cdot t \cdot (1.0 - 0.005 \cdot F_y) \cdot \sqrt{\frac{t}{0.028}} & \text{if } t < 0.028 \\ [1.25 F_y \cdot t \cdot (1.0 - 0.005 \cdot F_y)] & \text{otherwise} \end{cases} & \\ 0 & \text{otherwise} \end{cases} \qquad Q_{f\_stl} = 0.483 \quad \text{kips}$$

Field screw strength if ***wood*** controls

$$Q_{f\_wood} := 32.0 \cdot d_{fs}{}^2 \cdot G \qquad Q_{f\_wood} = 0.492 \quad \text{kips}$$

Reduce the field screw fastener strength to account for penetration

$$Q_{f\_wood} := Q_{f\_wood} \cdot R \qquad Q_{f\_wood} = 0.41 \quad \text{kips}$$

The strength of the field screw is the lover of these two values

$$Q_f := \min(Q_{f\_stl}, Q_{f\_wood}) \qquad Q_f = 0.41 \quad \text{kips}$$

Seam screw strength:

$$Q_s := \begin{cases} 115 \cdot d_{ss} \cdot t \cdot \sqrt{\frac{t}{0.028}} & \text{if } t < 0.028 \\ (115 \cdot d_{ss} \cdot t) & \text{otherwise} \end{cases} \qquad Q_s = 0.25 \quad \text{kips}$$

Determine diaphragm factors and fastener contribution factor

Diaphragm factors for field screws:

$$\alpha a_p := \frac{\sum xpa}{w} \qquad \alpha a_p = 1$$

$$\alpha a_{p\_sq} := \frac{\sum (xpa)^2}{w^2} \qquad \alpha a_{p\_sq} = 0.348$$

Diaphragm factors for screws into blocking:

$$\alpha b_p := \frac{xpb_1}{w} \qquad \alpha b_p = 0.469$$

$$\alpha b_{p\_sq} := \frac{\left(xpb_1\right)^2}{w^2} \qquad \alpha b_{p\_sq} = 0.22$$

Weighted average diaphragm factors for field and blocking screws:

$$\alpha_p := \frac{n_{ip} \cdot \alpha a_p + n_{pip} \cdot \alpha b_p}{n_{ip} + n_{pip}} \qquad \alpha_p = 0.75$$

$$\alpha_{p\_sq} := \frac{n_{ip} \cdot \alpha a_{p\_sq} + n_{pip} \cdot \alpha b_{p\_sq}}{n_{ip} + n_{pip}} \qquad \alpha_{p\_sq} = 0.287$$

Diaphragm factors for end screws:

$$\alpha_e := \frac{\sum xe}{w} \qquad \alpha_e = 2$$

$$\alpha_{e\_sq} := \frac{\sum xe^2}{w^2} \qquad \alpha_{e\_sq} = 0.695$$

Diaphragm factors for seam screws:

$$\alpha_s := \frac{Q_s}{Q_f} \qquad \alpha_s = 0.6$$

Fastener contribution factor:

$$n_p := n_{ip} + n_{pip} \qquad n_p = 9.563$$

$$B := n_s \cdot \alpha_s + 2 \cdot n_p \cdot \alpha_{p\_sq} + 4 \cdot \alpha_{e\_sq} \qquad B = 18.402$$

Calculate the nominal strength for the three limit-states

Limit-state 1: Unit shear strength controlled by field fasteners

Corner fastener strength reduction factor $\lambda := \max\left(0.7, 1 - \frac{d_d \cdot L_v}{240 \cdot \sqrt{t}}\right)$ $\qquad \lambda = 0.948$

$$S_{u\_f} := \frac{Q_f}{L} \cdot [B + 2 \cdot (\lambda - 1)] \qquad S_{u\_f} = 0.627 \ \frac{kips}{ft}$$

Limit-state 2: Unit shear strength controlled by corner fasteners

$$S_{u\_c} := Q_f \cdot \sqrt{\frac{N^2 \cdot B^2}{L^2 \cdot N^2 + B^2}} \qquad S_{u\_c} = 0.547 \ \frac{kips}{ft}$$

Limit-state 3: Unit shear strength controlled by panel buckling

$$S_{u\_b} := \frac{3250}{L_v^2} \cdot \left(\frac{I_x^3 \cdot t^3 \cdot p}{s}\right)^{\frac{1}{4}} \qquad S_{u\_b} = 0.532 \ \frac{kips}{ft}$$

Controlling nominal unit shear strength:

$$S_u := \min(S_{u\_c}, S_{u\_f}, S_{u\_b}) \qquad S_u = 0.532 \ \frac{kips}{ft}$$

Apply ASD factor of safety

$$Des\_Unit\_Strength := \frac{S_u}{2.5} \qquad Des\_Unit\_Strength = 0.213 \ \frac{kips}{ft}$$

Diaphragm Stiffness Prediction

Determine screw flexibilities

Field fastener flexibility $S_f := 0.20 \ \frac{in}{kips}$

Elevated sidelap structural fastener flexibility $S_{purlin} := \frac{3}{1000\sqrt{t}}$ $\quad S_{purlin} = 0.025 \ \frac{in}{kips}$

Stitch screw flexibility (determined from connection tests) $S_{stitch} := .054 \ \frac{in}{kips}$

Average seam/stitch screw flexibility

$$S_s := \begin{cases} S_{purlin} & \text{if } n_{stitch} = 0 \wedge n_{purlin} = 0 \\ \dfrac{n_{purlin} \cdot S_{purlin} + n_{stitch} \cdot S_{stitch}}{n_{purlin} + n_{stitch}} & \text{otherwise} \end{cases} \qquad S_s = 0.054 \ \frac{in}{kips}$$

Determine stiffness coefficients

Edge fastener stiffness coefficient $K := .5$

Coefficient for accounting for the number of spans

$n_{spans} = 6$

$$\phi := \begin{cases} 1.0 & \text{if } n_{spans} = 1 \\ 1.0 & \text{if } n_{spans} = 2 \\ 0.9 & \text{if } n_{spans} = 3 \\ 0.8 & \text{if } n_{spans} = 4 \\ 0.71 & \text{if } n_{spans} = 5 \\ 0.64 & \text{if } n_{spans} = 6 \\ 0.58 & \text{otherwise} \end{cases} \qquad \phi = 0.64$$

Panel warping coefficient

Number of corrugations per cladding sheet $n = 4$

Number of corrugations per sheet with 2 screws at each end corrugation $n_2 := 4$

Number of corrugations per sheet with 1 screw at each end corrugation $n_1 := n - n_2$ $n_1 = 0$

End Warping Coefficient Constants

For 1 screw per corrugation $V_1 := 1.6$

For 2 screws per corrugation $V_2 := 1.0$

D.n for a given V value

$$D_n(V) := \begin{cases} \dfrac{d_d \cdot w_t^{\,2}}{25 \cdot L} \cdot \left(\dfrac{1}{t}\right)^{1.5} & \text{if } n_2 = n \\ \left[\left(0.94 \cdot \dfrac{p}{w_t}\right) \cdot V^2 \cdot \left(\dfrac{d_d \cdot w_t^{\,2}}{25 \cdot L}\right) \cdot \left(\dfrac{1}{t}\right)\right] & \text{otherwise} \end{cases}$$

$D_n(V_1) = 0.208$

$D_n(V_2) = 0.208$

$$D_n := \frac{n_1 \cdot D_n(V_1) + n_2 \cdot D_n(V_2)}{n}$$ $D_n = 0.208$

Fastener flexibility coefficient, C

$$C := \frac{E \cdot t \cdot S_f}{w} \cdot \left[\frac{24 \cdot L}{2 \cdot \alpha_e + n_p \cdot \alpha_p + 2.n_s \cdot \left(\dfrac{S_f}{S_s}\right)}\right]$$ $C = 4.89$

In-Plane shear stiffness modulus

$$G' := \frac{E \cdot t \cdot K}{2.6 \cdot \left(\dfrac{s}{p}\right) + \phi \cdot D_n + C}$$ $G' = 26.7 \ \dfrac{\text{kips}}{\text{in}}$

Effective shear modulus for the diaphragm assembly

Number of purlins per rafter $n_{pur} := n_{spans} + 1$ $n_{pur} = 7$

Number of shear blocks per rafter $n_{rsb} = 6$

Stiffness of a single rafter-purlin connection $K_{rp} := 0.969 \frac{kips}{in}$

Stiffness of a single shear block $K_{sb} := 9.77 \frac{kips}{in}$

Total Stiffness of all connections on a single rafter

$$K_R := n_{pur} \cdot K_{rp} + n_{rsb} \cdot K_{sb}$$ $K_R = 65.403 \frac{kips}{in}$

Effective shear modulus for the diaphragm assembly

$$G'_{net} := \frac{\frac{a}{b}}{\frac{a}{G' \cdot b} + \frac{2}{K_R}}$$ $G'_{net} = 11.91 \frac{kips}{in}$

APPENDIX C: DIAPHRAGM DESIGN EXAMPLE

Given:

A roof diaphragm is 20 feet long (eave to ridge distance), and trusses/rafters are spaced 8 feet on center. The roof will be constructed with 2x4 purlins oriented on-edge and connected to the top of the rafter with a 60d ring shank nail. Based on preliminary calculations, the diaphragm unit shear demand is estimated to be 180 plf.

Find:

Design the diaphragm and determine the effective shear modulus of the chosen diaphragm construction.

Solution:

The roof diaphragm will be constructed using Fabral Grandrib 3 cladding. The cladding properties of interest are:

- Major rib spacing: 9 in. o.c.
- Gauge: 29
- Yield/Ultimate strength: 80/82 ksi
- Top width of major corrugation: 0.375 in.
- Bottom width of major corrugation: 1.75 in.
- Height of major corrugation: 0.75 in.

Step 1: Determine design unit shear strength governed by screws:

The Grandrib 3 meets all the required criteria for use of design tables. No. 12-1.5” field screws and No. 12 stitch screws spaced 12” on center will be used. The shear strength and in-plane stiffness values are boxed and shaded in the applicable design table shown below.

Purlin Spacing = 2 ft. Minimum Purlin S.G. = 0.42 Major Rib Bottom Width Range = 1 - 2.5 in.
Major Rib Spacing = 9 in. Major Rib Height Range = 0.625 - 1.0 in. Major Rib Top Width Range = 0.25 - 0.75 in.

Min. Yield/ Ultimate Strength	Gauge	Field Screws into Flats	Screws into Panel Overlaps	NO Screws into Panel Overlaps			Spacing of screws into the panel overlaps (in.)								
							24			12			8		
				V_{10}	V_{50}	G'	V_{10}	V_{50}	G'	V_{10}	V_{50}	G'	V_{10}	V_{50}	G'
(ksi)				(plf)	(plf)	(kips/in)	(plf)	(plf)	(kips/in)	(plf)	(plf)	(kips/in)	(plf)	(plf)	(kips/in)
80/82	28 (.0149")	#9-1"	None	102	85	2.8	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	130	102	19	167	142	27	199	178	32
			#12 Stitch	----	----	----	136	107	19	176	152	27	211	190	32
		#10-1"	None	108	89	2.8	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	134	105	19	172	146	27	205	182	32
			#12 Stitch	----	----	----	140	111	19	182	156	27	217	195	32
		#10-1.5"	None	129	107	2.8	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	151	117	19	190	158	27	225	197	32
			#12 Stitch	----	----	----	157	123	19	200	169	27	238	211	32
		#12-1.5"	None	165	136	2.8	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	179	137	19	219	179	27	256	219	32
			#12 Stitch	----	----	----	185	142	19	230	190	27	271	234	32
	29 (.0135")	#9-1"	None	102	85	2.8	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	123	96	18	156	131	25	186	163	29
			#12 Stitch	----	----	----	128	101	18	165	140	25	197	175	29
		#10-1"	None	108	89	2.8	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	128	99	18	161	135	25	191	167	29
			#12 Stitch	----	----	----	133	104	18	170	143	25	202	179	29
		#10-1.5"	None	127	105	2.8	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	142	109	18	176	145	25	208	179	29
			#12 Stitch	----	----	----	148	114	18	185	155	25	220	192	29
		#12-1.5"	None	155	128	2.8	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	164	125	18	199	162	25	232	197	29
			#12 Stitch	----	----	----	170	130	18	209	171	25	245	210	29
	30 (0.0120")	#9-1"	None	102	85	2.8	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	116	89	16	145	120	22	171	148	26
			#12 Stitch	----	----	----	121	94	16	152	127	22	181	158	26
		#10-1"	None	108	89	2.8	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	121	93	16	149	123	22	176	152	26
			#12 Stitch	----	----	----	125	97	16	157	131	22	186	162	26
		#10-1.5"	None	112	93	2.8	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	124	95	16	153	126	22	180	154	26
			#12 Stitch	----	----	----	129	99	16	161	133	22	190	165	26
		#12-1.5"	None	130	107	2.8	----	----	----	----	----	----	----	----	----
			#10 Stitch	----	----	----	138	105	16	167	136	22	194	165	26
			#12 Stitch	----	----	----	142	109	16	175	143	22	205	176	26

* Safety factor of 2.5 has been applied in all cases

Determine reduction factor for a 20 ft diaphragm length

R $= 12.5/L - 0.25$

$= 12.5/20 - 0.25$

$= 0.38$

Determine shear strength governed by screws

$V_{screw} = R(V_{10} – V_{50}) + V_{50}$

$= 0.38(209 \text{ plf} – 171 \text{ plf}) + 171$

$= 185 \text{ plf}$

Step 2: Determine design unit shear strength governed by panel buckling

The design table corresponding to 29 gauge steel and 9 in. major rib spacing must be used. Linear interpolation between the two values boxed and shaded in the design table below is required to determine the design unit shear strength governed by panel buckling for a major rib top width of 0.375 in.

Rib Height (in.)	Top Width of Rib (in.)	Bottom Width of Rib (in)						
		1	1.25	1.5	1.75	2	2.25	2.5
0.625	0.25	124	128	132	137	143	148	154
0.625	0.75	155	157	161	165	169	174	178
0.75	0.25	169	174	180	186	192	199	205
0.75	0.75	210	213	217	222	227	232	238
0.875	0.25	222	227	233	240	248	256	264
0.875	0.75	273	276	281	286	292	299	306
1	0.25	281	287	294	302	310	319	329
1	0.75	343	347	352	358	364	372	380

* Safety factor of 2.5 has been applied to all cases

Using linear interpolation,

$$V_{buck} = y_1 + (x - x_1)\frac{y_2 - y_1}{x_2 - x_1}$$

$$= 186 \text{ plf} + (0.375 \text{ in} - 0.25 \text{ in})\frac{222 \text{ plf} - 186 \text{ plf}}{0.75 \text{ in} - 0.25 \text{ in}}$$

$$= 195 \text{ plf} > [V_{screw} = 185 \text{ plf}]$$

Therefore, the allowable design unit shear strength is 185 plf.

$$V_{allow} = 185 \text{ plf} > 180 \text{ plf} \therefore \text{ OK}$$

Step 3: Determine the effective shear modulus

For a diaphragm length of 20 feet with purlins spaced 2 feet on center, a total of 11 purlins are used. The stiffness of all connections on one rafter is determined to be:

$$K_R = N_p K_p + N_{sb} K_{sb}$$

$$= 11 \text{ purlins} \cdot (1.0 \text{ kips/in})$$

$$= 11 \text{ kips/in}$$

Member	Connection	Size	Orientation	Location	Specific Gravity	Stiffness (kips/in)
Purlin	1-60d hardened steel ring shank nail	2x4	on-edge	on top of rafter	0.42	1.0
Blocking	2-60d hardened steel ring shank nails	2x4	on-edge	on top of rafter	0.42	10.0

The effective shear modulus is then calculated as:

$$G_a = \frac{a/b}{\frac{a}{G'b} + \frac{2}{K_R}}$$

$$= \frac{8\text{ ft}/20\text{ ft}}{\frac{8\text{ ft}}{25\text{ kips/in} \cdot 20\text{ ft}} + \frac{2}{11\text{ kips/in}}}$$

$$= 2.0\text{ kips/in}$$

<u>Diaphragm Design:</u> The Fabral Grandrib 3 will be used with No. 12 x 1.5” field screws and No. 12 stitch screws spaced 12 in. on center. At the location of shear walls, structural screws are required at 12 in. on center with blocking placed between purlins. 2x4 purlins will be spaced 2 feet on center using lumber with a minimum specific gravity of 0.42. The effective shear modulus is 2.0 kips/in.

Note: The actual load in the diaphragm can be determined now that the effective shear modulus is known for load distribution calculations. The design can then be refined if needed.