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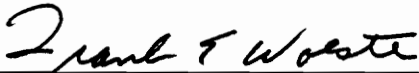
Bracing Requirements for Post-Frame Endwalls

by

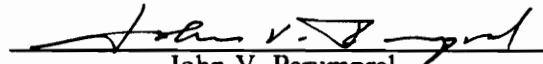
Donald Lee Wirt, Jr.

Thesis submitted to the Faculty of the
Virginia Polytechnic Institute and State University
in partial fulfillment of the requirements for the degree of
Master of Science
in
Agricultural Engineering

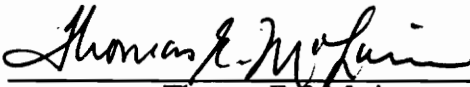
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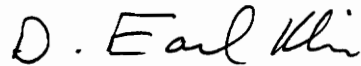
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by

Donald Lee Wirt, Jr.

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Agricultural Engineering

(ABSTRACT)

When diaphragm action is considered in the design of post-frame buildings, the endwalls may require supplemental reinforcement to replace lost strength and stiffness, due to placement of large doors and openings in the endwalls. The diaphragm design standard, ASAE EP484, does not provide guidelines concerning such endwall reinforcement. Flat steel strapping is proposed for use as supplemental reinforcement to replace this lost strength and stiffness.

An analysis procedure was developed, with the intention of serving as input to endwall diaphragm design. Using PPSA 3.00 and calibrated endwall data, a computer structural analog was developed that "mimics" the deflection of an actual solid endwall. Sections of the structural analog were removed to represent a centered door opening. Fictitious members representing steel-strap bracing were added to the remaining endwall diaphragm to restore the endwall deflection to that of the same endwall with no door openings.

Large corner post uplift values were observed for tall endwalls. For purposes of endwall analysis, it is necessary to distribute corner post uplift forces to the other posts along the adjacent endwall and sidewall.

Along with the tabulated results in this research, this analysis procedure provides building designers a tool that will enable them to analyze virtually any endwall-door combination. All that is required is PPSA 3.00 software (or equivalent), a personal computer, and endwall stiffness data.

Acknowledgements

I would like to express my appreciation to my major professor, Dr. Frank Woeste, for his guidance and teaching me how to work and have fun at the same time. Appreciation is also extended to the other members of my advisory committee, to which I consider it an honor to have their name on my thesis.

A special thanks is extended to Mr. Matthew Carroll for giving his time freely to produce the illustrations. Thanks Matt.

My deepest appreciation is expressed to my parents, Mr. and Mrs. Donald Wirt, and especially my wife, Denise, for all of their support during my college career. Denise, perhaps I will now be able to support you.

And most of all, I could not have done any of this without the strength and guidance of the Lord. It is written, "I can do all things through Him who strengthens me".

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1. Introduction

The potential for diaphragm design in post-frame buildings is widely recognized. Diaphragm design utilizes the ability of the framing and metal-cladding to carry in-plane shear forces that are produced by wind loads to the ground. When diaphragm design is considered, adequate endwall strength and stiffness is critical in limiting horizontal deflection at the endwall.

ASAE (1989) EP484 provides standardized methods for metal-clad, timber-framed diaphragm testing and outlines engineering procedures for analysis and design of metal-clad, post-frame buildings. Stated in the purpose of this practice is that the endwalls must be sufficiently rigid to transfer shear loads from the roof, through the endwall, and down to the ground. Also stated is that the presence of large openings in the endwall may require supplemental reinforcement to replace the lost strength and stiffness. Lacking, however, are the guidelines necessary to assist building designers in reinforcing endwalls having large doors and openings.

The standard is vague about endwalls having doors because known solutions were just not available at the time of publication. Research literature on endwall studies is limited. However, several recent studies on diaphragms and endwalls provide the foundation for a study that will provide practical solutions for reinforcement of endwalls with openings.

A material often used for reinforcement of endwalls with large doors is flat, coil type, steel strapping. It is usually nailed to the posts. This form of bracing provides a practical and economical means of supplemental reinforcement. However, this method of reinforcement may not live up to the design expectations.

On occasion, racking will occur in buildings having braced endwalls with large doors, at well below design wind loading conditions. If, for example, the interior walls of the building were covered with gypsum wallboard, cracks may ensue. Problems such as this may result in dissatisfied customers, degradation of a builder's reputation, and possible litigation suits. There is strong potential for steel strap bracing in endwall reinforcement, but an analysis procedure is essential for quantifying the required bracing.

Buildings having large sidewall surface areas, resulting in high wind loads acting upon the building, often have high uplift forces at windward foundation corners. When a large door is placed in an endwall, there are fewer posts to carry endwall shear to the ground. Consequently, forces on the remaining posts are magnified. Historically, corner post uplift failures do not appear to be a problem, but design analyses frequently indicate otherwise. This study provides an opportunity to investigate the discrepancy between the actual cases and their design analyses.

The overall scope of this research was to develop an endwall analysis procedure, to aid in determining the required wall and foundation reinforcement, for post-frame buildings having large doors centered in the endwall.

By means of steel strap bracing, the design approach to be used in this study is to restore the final deflection of an endwall having a large opening to that of the same endwall that is solid. Predicting diaphragm behavior requires extensive knowledge in the background of the mechanics of all variables involved and their interactions. The predictive models are not available. We propose to develop an analysis procedure, calibrated with test data, that can supply information for required bracing when doors are installed in endwalls.

Specifically, the objectives of this study were to:

1. Determine the required supplemental reinforcement to replace the lost strength and stiffness of endwalls due to placement of doors and large openings.
2. Investigate the characteristics of strap performance and nail connection stiffness.
3. Investigate corner post uplift for endwalls with doors and determine how high uplift forces may be reconciled.
4. Provide a simple tool that will enable building designers to analyze endwall-strap bracing schemes outside of the scope of this study.
5. Determine the applicability of the actual endwall data used in this research to the many variations common to the post-frame industry.

2. Literature Review

2.1 *Background*

Figure 1 is an excellent illustration of how loads produced by wind are transferred from the building to the ground using diaphragm action (Canada Plan Service 1985). The wind produces a force on the sidewall and roof cladding, which is transferred to the post-frames by means of the wall girts and purlins. Some of the shear developed will be transferred directly to the ground by the posts, with the magnitude increasing as the stiffness of the frame increases. The remaining shear is then transferred by the post-frames into the roof cladding. With the roof acting as a deep beam, the maximum roof shear is at the building ends. Near the endwall, the roof shear goes from the roof cladding back into the purlins, and into the top chord of the gable-end truss. Once into the endwall, some of the shear is carried by the gable-end truss cladding above the door opening. Desiring to be conservative, the shear resistance of the truss cladding above doors is neglected here. Thus it is assumed that all of the shear from the roof goes into the endwall posts by the attachment of the truss, then through the endwall girts to the endwall cladding, back through the lower endwall girts to the posts, and finally is transferred to the ground.

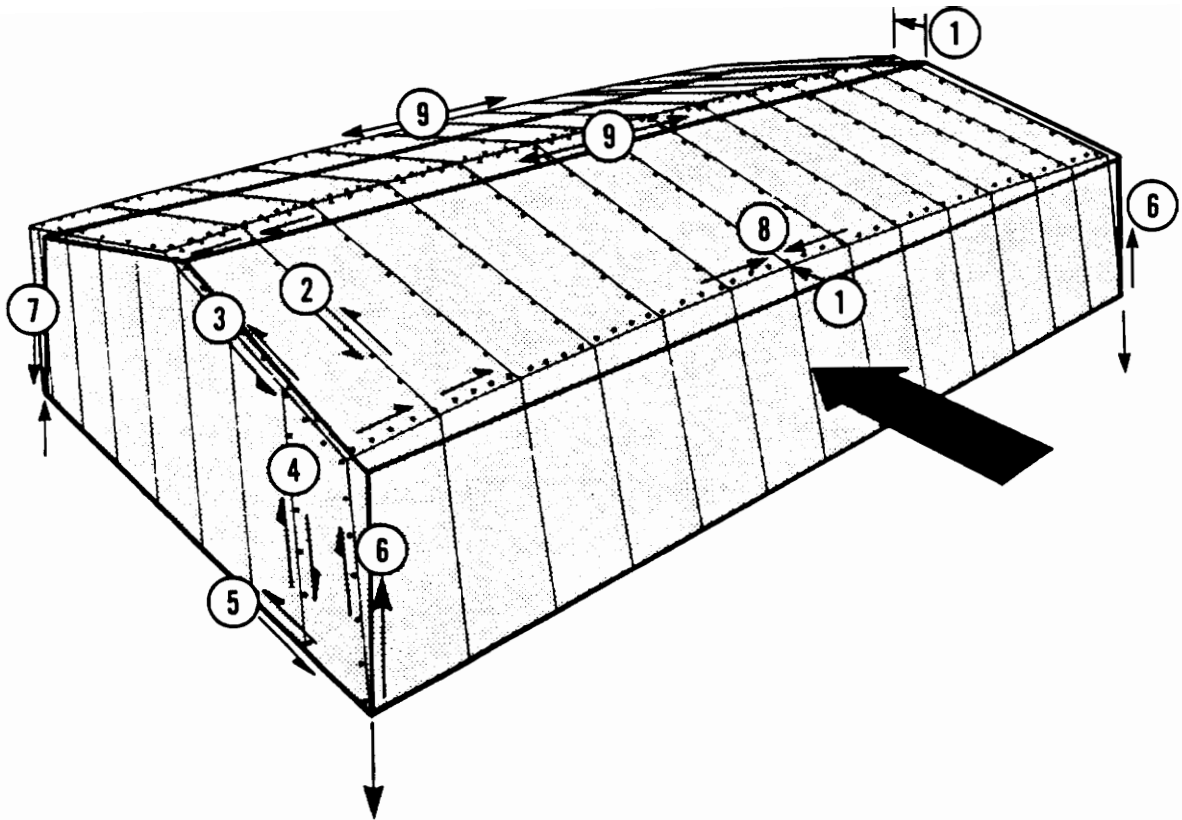


Figure 1. Diaphragm action of a metal-clad post-frame building: (Canada Plan Service, 1985).
 1 sideways in assumed critical wind direction
 2 stitch screws at the roofing edge-laps carry the wind shear forces to the ends of the roof
 3 roofing screws and blocking carry the forces into the gable truss
 4 endwall cladding carries the forces from the gable trusses down the endwalls
 5 cladding-to-foundation shear connection
 6 uplift forces at windward foundation corners
 7 leeward corner forces may be up or down, and are less critical than windward corner
 8 roof bending causes compression at the windward edges, and
 9 tension at the leeward edges of the roof planes.

There is an extensive data base and a wealth of research available for diaphragm design. However, the vast majority of this research is based on roof and ceiling diaphragms. The following literature does lend to the study of endwall diaphragms. While reviewing this literature, it may be beneficial to refer to Tables 1 and 2 for summaries of the diaphragm tests. Specific details such as lumber, hardware, or sizes, are sometimes not specified, as they were not specified in the respective literature.

2.2 Early Studies on Diaphragms

Hausmann and Esmay (1975) addressed the role the building skin plays in the design of pole buildings, for wind resistance. The need for utilizing diaphragm action in design was realized, but the data available for metal-clad pole buildings was insufficient. The 26 diaphragms tested were referred to as cantilever tests, but an illustration and test description depict the use of a simple beam test assembly. The lumber used was typical to common practice but grade descriptions were not reported. The load-deflection relationships for diaphragms were studied by varying the size of frame assembly, type and number of fasteners, type of metal, and the inclusion of rigid insulation. The shear load versus deflection curves were reported. The two major conclusions drawn were that aluminum and steel sheathing make good diaphragm systems, and to provide the best diaphragm action, the fasteners need to be placed in the flats of the sheathing.

Table 1. Summary of diaphragm constructions in cited literature.

Investigator(s)	Test Assembly	Number of Tests	Diaphragm Size ^{3 4}	Truss Chord Grade Size ¹
Anderson 1987	Cantilever	16	9 ft x 12 ft	#2 SPF 2x8
Turnbull et al. 1982	Simple beam	3	63 ft x 16 ft	N/R 2x6
Hausmann & Esmay 1975	Cantilever	26	16 ft x 8 ft	N/R 2x6
Lukens & Bundy 1987	Cantilever	19	9 ft x 6 ft to 9 ft x 20 ft	#2 + ⁵ SPF 2x8 or #2 + DFL 2x8
Hoagland 1981	Cantilever	12	8 ft x 12 ft	#2 + 2x8
Anderson 1989	Cantilever	28	9 ft x 12 ft 9 ft x 18 ft	#1 SYP 2x8 #1 SYP 2x8
Anderson & Bundy 1990	Cantilever	15	7-2/3 ft x 12 ft	#2 SPF 2x8
Gebremedhin & Irish 1986	Simple beam	16	20 ft x 8 ft 16 ft x 8 ft	#2 + DF 2x8 #2 + DF 2x8
White 1978	Cantilever	40	9 ft x 12-2/3 ft 9 ft x 18-2/3 ft	#1 or #2 DSYP 2x8 #1 or #2 DSYP 2x8

¹ All dimensions are nominal.

² N/R denotes information was not reported.

³ First dimension gives the width, a, for a cantilever test, or 2a, for a simple beam test. The second dimension is the diaphragm length, b.

⁴ 1 ft. = 0.3048 m.

⁵ #2 + refers to #2 and Better material grade.

Table 2. Summary of diaphragm constructions in cited literature.

Investigator(s)	Purlin Grade	Purlin Size ¹	Purlin Spacing (OC)	Sheeting Fasteners ²	Sheeting Type(s)
Anderson 1987	Std-Btr	SPF 2x4	20-24 in.	#10 (2 in. lg. in ribs)	29 ga Grandrib 3
Turnbull et al. 1982	N/R	2x2	18 in.	#8	30 ga W-R-L Diamond Rib
Hausmann & Esmay 1975	N/R N/R	2x4 2x4	24 in. 24 in.	N/R N/R	29 ga steel 0.021 in. Aluminum
Lukens & Bundy 1987	Std-Btr or Std-Btr	SPF 2x4 DFL 2x4	24 in.	#10	29 ga Grandrib 3
Hoagland 1981	Const Const	2x4 2x4	24 in. 24 in.	"Woodgrip" (2 in. lg. in ribs) "Woodgrip" (2 in. lg. in ribs)	Weatherstrong 29 ga Strongpanel
Anderson 1989	Varies	2x4	24 in.	#10	28 or 29 ga Wick Agri Panel
Anderson & Bundy 1990	Std-Btr	SPF 2x4	20-24 in.	#10 (1.5 in. lg. in ribs)	29 ga Strongpanel
Gebremedhin & Irish 1986	Const	DF 2x4	Varied	#11 edge x 2 in. #8 frame #12 stitch x 5/8 in.	0.018 ga Aluminum 29 ga steel by Fabral
White 1978	#1 or #2 #1 or #2	DSYP 2x4 DSYP 2x6	Varied Varied	#10 #10	0.015-0.016 in. Super Temper Rib by ALCOA

¹ All dimensions are nominal.

² All screws are self-tapping, with neoprene/steel washer and 1 in. long unless otherwise specified.

³ Descriptions are for the principal diaphragm configurations.

Hurst and Mason (1961) studied the effects of increased rigidity attributable to endwall framing and metal sheathing. Two full-size pole-type buildings, 10.9 m x 13.7 m (36 ft. x 45 ft.) clear span having knee-braces, were used. The poles were set in 0.3 m (1 ft.) of concrete at the base and there was a concrete slab at ground level to permit repeated loadings with a minimum of permanent set. All lumber in both buildings was No. 2 Southern Pine, dressed except for what was used in the experimental trussed rafters, and hardened screw nails were used for fasteners. The poles used in both buildings were pressure treated southern pine having 127 to 152 mm (5 to 6 in.) tops. The moisture content was not reported. The buildings were subjected to increasing increments of simulated wind loading. Deflections were measured at 18 points along each pole at the corners of the endwalls. The first part of the test involved loading the sidewall and roof framing, without the sheathing and endwalls. Next, one endwall was erected and the structure was retested. The third step was to apply sheathing to the sidewalls and endwalls, and again retest. Their study indicated that deflection at the ends of the buildings was reduced by two-thirds with the addition of endwall framing. Further, the endwall deflection was reduced approximately 95 percent with the addition of diamond rib aluminum sheathing.

2.3 Diaphragm Tests

White (1978) conducted an array of experiments on 40 diaphragms to assess the performance for varied truss spacing, purlin spacing and size, connector patterns and type, diaphragm length, and type of loading. All sheeting was 0.38-0.40 mm (0.015-0.016 in.) aluminum panels, having 19 mm (3/4 in.) deep ribs. In all cases, the shear stiffness of fully constructed frames without sheeting was negligible. An important conclusion of this paper is that the purlin spacing does not appear to be a major variable in strength, given that the sidelap seam strength is maintained according to 0.61 m (2 ft.) spacing, by means of either adhesives or stitching. This may be extended to endwall girt

spacing in that as long as the comparable design strength of 0.61 m (2 ft.) on-center roof purlins is maintained, endwall girt spacing can be increased if stitching is used, 0.61 m (2 ft.) or less on-center.

Hoagland (1981) reported the in-plane strengths and stiffnesses of some typical roof framing systems. Included in this study was the assessment of the effect of typical framing methods and the application of construction glues on the strength and stiffness. Twelve full-scale roof diaphragms were tested. Panel construction included recessed purlins, purlins on-edge, and purlins on-edge with shear connector blocking. Panel stitching was used on all diaphragms. Hoagland found all diaphragm panels to have a linear behavior up to 40 percent of the ultimate shear strength.

Turnbull et al. (1982) studied the effects of simulated wind loads on three different types of farm building ceilings, testing one diaphragm for each type. Using the simple beam test, they tested a conventional plywood diaphragm, an improved plywood diaphragm where a 1.2 m by 1.2 m (4 ft. by 4 ft.) grid of 2 x 4 strapping was nailed to the truss bottom chord for an improved plywood nail joint, and also a screwed, metal-clad diaphragm. They found that a ceiling with steel sheathing was 2.4 times stiffer than the conventional plywood diaphragm and 1.6 times stiffer than the improved plywood diaphragm. The plywood diaphragms showed some creep during a hold period, indicating that including an adjustment for load duration is an appropriate measure. The steel sheathing showed no signs of creep, indicating that a load duration factor need not be applied.

Turnbull (1981) presents a summary of the Canada Plan Service methods of diaphragm design for wind bracing of agricultural buildings. Turnbull et al. (1982) is reviewed and noted that the steel sheathing diaphragms are much stiffer than the other diaphragms tested, and that steel sheathing diaphragms are now much easier to construct. They indicated that there is great potential for use of metal-clad diaphragms in farm buildings. A general overview of the diaphragm design practice is discussed along with considerations of various elements of Canadian diaphragm design.

Lukens and Bundy (1987) reported a summary of the shear strengths and stiffnesses on nineteen roof diaphragms tested using the cantilever type test. Variables included methods of defining shear

stiffness, reverse loading, type of wood used for purlins, purlin to chord connection, and length. It was concluded that diaphragm stiffness increased with the increase of diaphragm length, and that diaphragm strength is independent of diaphragm length because of failure at the purlin-to-chord connection.

Anderson and Bundy (1990) studied the stiffness of roof diaphragms having openings in the metal cladding. Classifications of the 2.34 m x 3.66 m (7-2/3 ft. x 12 ft.) diaphragms included: control diaphragm, 11 percent of steel missing, 33 percent of steel missing, and purlins recessed. Each of the fifteen diaphragms were constructed using nine steel panels (each representing a grid of 11 percent of total area) and by removing one or several panels, the effects of the size of opening as well as location of opening could be observed. Their results, from both testing and computer analysis, showed that diaphragms behave nonlinearly. With computer analysis, they also found that 80 to 90 percent of the diaphragm shear displacement is attributed to fastener slip. Removing small amounts of metal cladding should not significantly affect diaphragm stiffness, all else remaining the same. When one-third of the metal cladding is removed along the full length of the diaphragm, whether it be parallel or perpendicular to loading, it becomes ineffective as a diaphragm. Recessing the purlins resulted in a small decrease in stiffness.

Anderson (1987) studied the behavior of light gage metal diaphragms on wood frames as well as diaphragm interaction with the post frames. Thirty-one diaphragms were tested of which the first 15 are discussed in Anderson and Bundy (1990). The diaphragm tests of Anderson and Bundy (1990) were intended to evaluate the effect of openings and the effect of recessing the purlins in from the edge of the steel on diaphragm stiffness. The diaphragm tests of Anderson (1987) were intended to look at the effects of multiple-sheet diaphragms, alternating the internal sheet to purlin pattern, placing purlins flatwise, and sheet and edge purlin strains on diaphragm stiffness. The information reported was the diaphragm stiffnesses, the predicted diaphragm stiffnesses by computer analysis, and the failure types that occurred. At low loads, the diaphragms were very stiff, and the computer model better predicted the stiffnesses in the linear portion of the load-deflection curve.

Anderson (1990) presented a summary of approximately 100 roof diaphragm tests, discussing how the components of the diaphragms interact and affect the strength and stiffness. This paper has a detailed discussion of the components of slip and how they contribute to diaphragm shear displacement, as well as and how each major slip component may be controlled. Anderson found that all of the components of roof diaphragms contribute to the strength and stiffness. Because of the complexity of diaphragms, prediction of diaphragm strength and stiffness is likewise complex. A problem in diaphragm design is that when one component is improved or reduced, the desired effect may not occur because one or more other components can become critical.

Anderson (1989) studied 28 roof diaphragms which are a subset of the 100 tests reported in the 1990 report. One of the primary objectives was to provide strength and stiffness data for:

1. different fastener patterns and types,
2. inclusion of rigid insulation,
3. seam fasteners,
4. different purlin lumber species,
5. different types of sheeting,
6. different diaphragm lengths.

This study is well documented, including diaphragm construction, strength, stiffness, and failure mechanisms for each diaphragm tested. Anderson states that current equations for predicting diaphragm stiffness for different diaphragm lengths and thicknesses are not accurate. He also concluded that stiffness can be significantly enhanced by improvements in fastener applications.

2.4 Prediction Models Assessed

Wright and Manbeck (1990) present a review of prediction models used for modeling diaphragm panels. Analytical methods used in timber-clad timber-framed, and metal-clad metal-framed diaphragm panels are summarized. Prediction capabilities for metal-clad, timber-framed diaphragm panels is discussed. The on-going development of a comprehensive finite element model for metal-on-wood diaphragms at the Pennsylvania State University is also discussed. It was concluded that prediction models for wood-on-wood and metal-on-metal diaphragm panels are basically inadequate for predicting metal-on-wood performance, and that a comprehensive finite element model may lead to better performance predictions in metal-on-wood diaphragm research.

Wright and Manbeck (1990b) present a progress report on the finite element modeling of metal-on-wood diaphragms. Some of the component flexibilities were not accurately predicted. However, in comparison of test results to model results, the overall stiffness was quite accurate and the strength was of the same order of magnitude.

2.5 A Key Study for Endwalls

Gebremedhin and Irish (1986) studied the effect of load orientation on strength for ceiling diaphragms. For one test, the orientation of the loading was parallel to the ribs (Figure 2). This test condition is representative of a roof diaphragm with purlins recessed (Figure 3). For another test, the rib orientation was perpendicular to the loading (Figure 4). This test condition is analogous to an endwall diaphragm or shearwall (Figure 5).

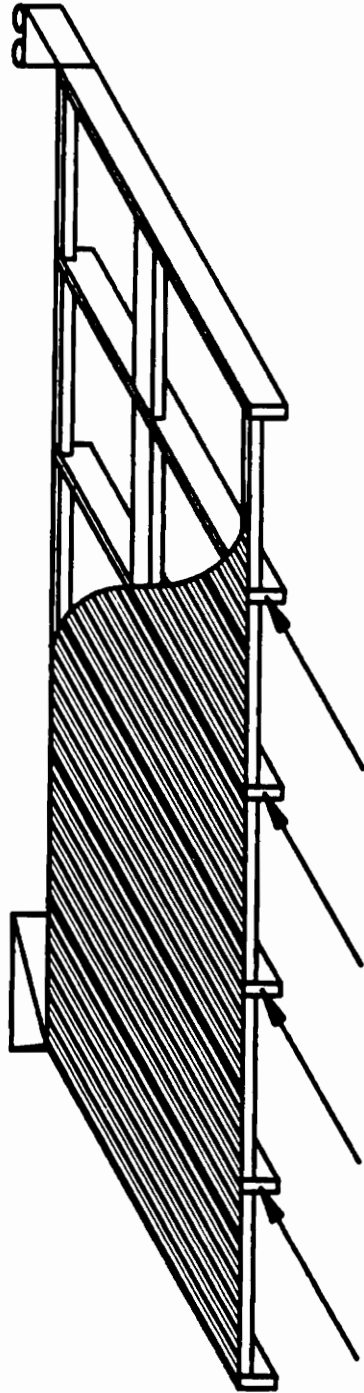


Figure 2. Simple beam diaphragm test by Gebremedhin and Irish (1986): The rib orientation of the metal cladding is parallel to the direction of loading.

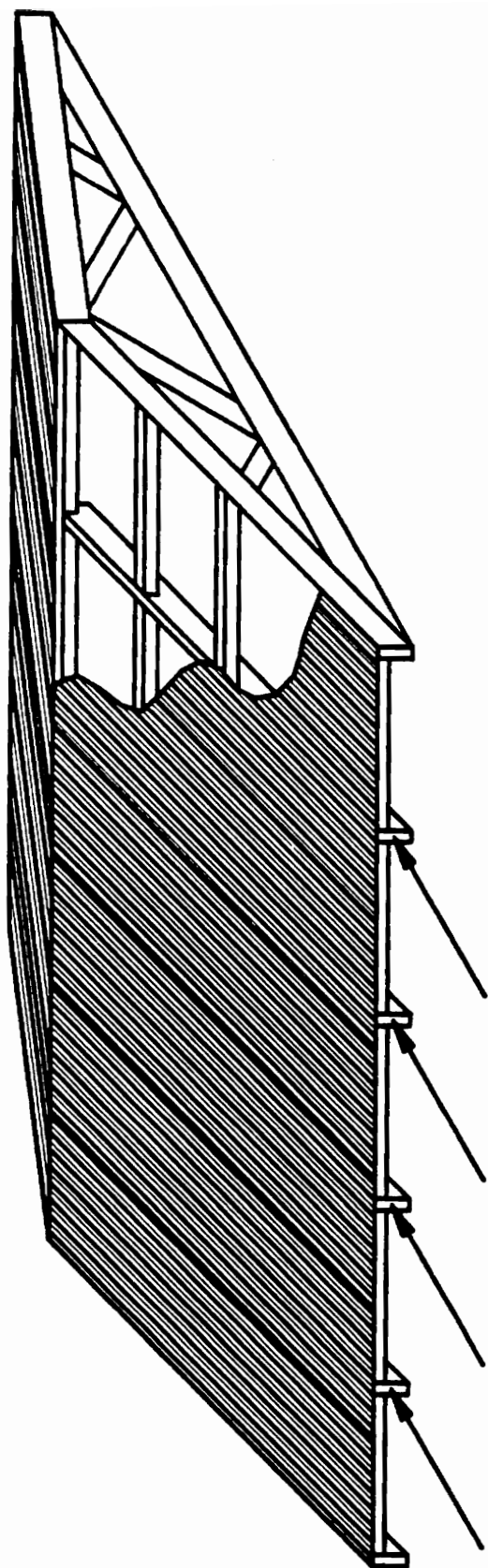


Figure 3. Common roof construction having recessed purlins: Wind pressures on the sidewall result in truss loading as shown.

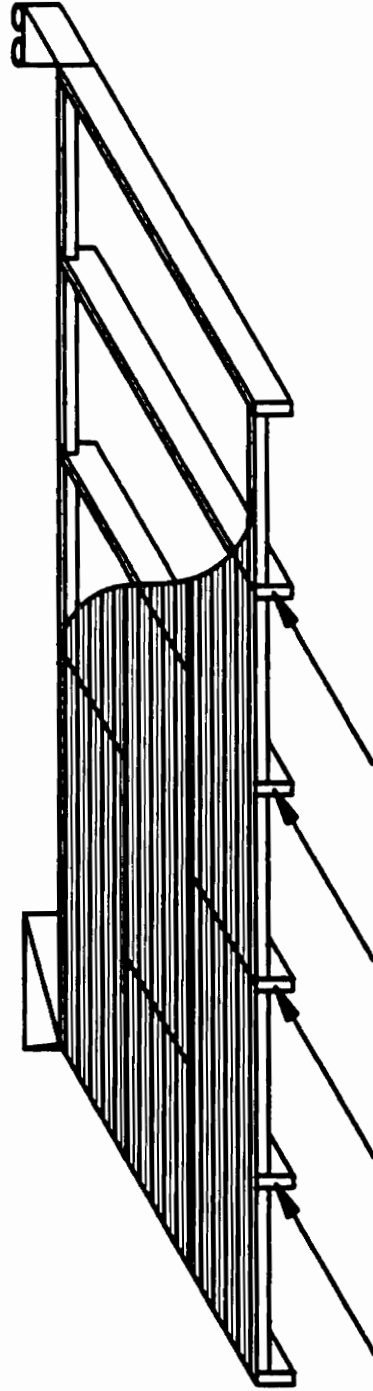


Figure 4. Simple beam diaphragm test by Gebremedhin and Irish (1986): The rib orientation is perpendicular to the direction of loading.

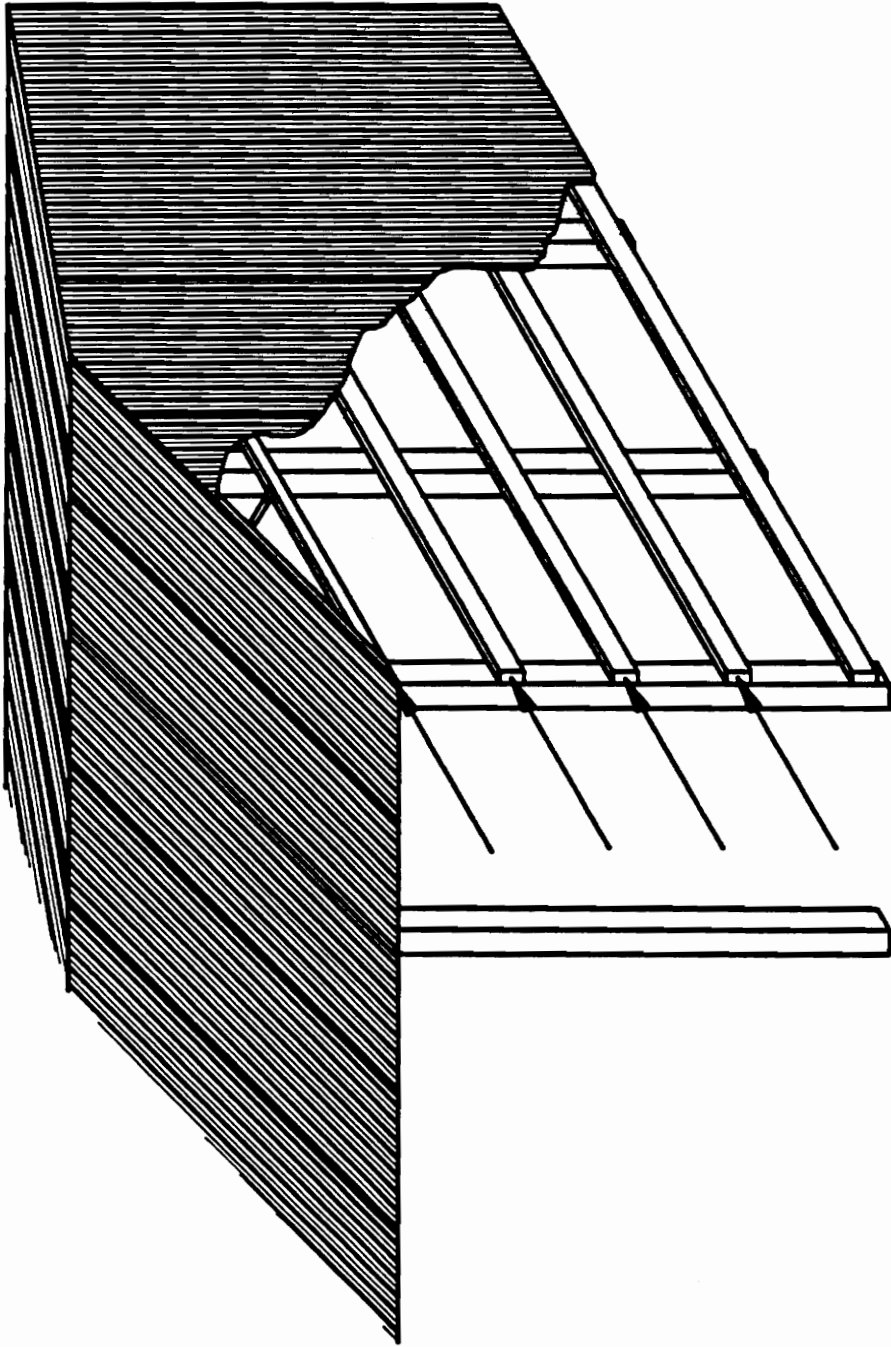


Figure 5. Common endwall construction having flatwise wall girts: The shear loading in an endwall actually enters horizontally, along the gable end truss. The vertical loading along the corner post is for drawing an analogy of the endwall to the diaphragm test in Figure 4.

The 2.44 m x 6.10 m (8 ft. x 20 ft.) diaphragms from Gebremedhin and Irish were tested using the simple beam diaphragm test. The truss bottom chords were simulated by 38 mm x 140 mm (nominal 2 in. x 6 in.) No. 2 and Better Douglas-fir members, placed on-edge and spaced 1.22 m (4 ft.) on-center. The 38 mm x 38 mm (nominal 2 in. x 2 in.) nailers were toe-nailed between chords along the sides, and 38 mm x 89 mm (nominal 2 in. x 4 in.) members were placed flatwise for use as edge beams. When the metal cladding was fastened with the ribs parallel to the truss bottom chords as in Figure 2, additional 38 mm x 89 mm (nominal 2 in. x 4 in.) blocking was nailed flatwise between the chords at 1.22 m (4 ft.) spacing or one-half the diaphragm width. This blocking acted as support members. All members were Construction grade Douglas-fir. Tests were conducted using 0.457 mm (0.018 in.) thick, equally-spaced, diamond-embossed aluminum panels, 0.368 mm (29 gage) galvanized steel having 11 mm (7/16 in.) deep ribs 203 mm (8 in.) on-center, and 0.368 mm (29 gage) galvanized steel having 19 mm (3/4 in.) deep ribs 228 mm (9 in.) on-center.

When the ribs were placed perpendicular to the supporting chords as in Figure 4 (and likewise the loading), the aluminum panel diaphragm had an ultimate shear strength 18 percent greater than when orientation was parallel to loading. In the same manner, the 11 mm (7/16 in.) steel panels had a 24 percent greater shear strength, and the 19 mm (3/4 in.) steel panels had a 39 percent greater shear strength. Comparison of the near linear region of the load-deflection relationships of the two orientations show that the shear stiffness is also increased in the perpendicular arrangement. The slope of the load-deflection curve is nearly twice that of parallel loading. With the results of this portion of the study, one may deduce that a diaphragm having ribs placed perpendicular to loading has greater shear strength and stiffness than when ribs are parallel to loading. These deductions are critical to the development of the analysis procedures for endwall design presented in this thesis.

With the results of the Gebremedhin and Irish (1986) tests, one may make a definitive statement about endwall strength. For a construction practice where purlins are 0.61 m (2 ft.) on-center, on-edge, and endwall girts are no greater than 0.61 m (2 ft.) on-center (Figures 3 and 5), the results of Gebremedhin and Irish can be used to reach the conclusion that the building endwalls are stronger than the roofs of the same configuration. When the girts are placed flatwise, as in many

endwall constructions, the strength and stiffness is at least as great as the on-edge application of framing. Thus, endwalls using either orientation of girts will be adequate, provided the roof has adequate strength.

2.6 Actual Endwall Data

McFadden et al. (1988) laterally loaded a full-scale post-frame building to study the diaphragm action of the roof. The posts, having 0.915 m (3 ft.) embedment depth, were fully set in concrete to minimize deflection due to leaning. This would also permit an analysis of deflections with the assumption of fixed supports. The building was tested in four stages of construction to determine the effect of the metal cladding on the eave deflections. Stage-1 testing included the building frame and framing without the metal sheathing. Stage-2 testing had the sheathing fastened to the endwalls. Stage-3 and stage-4 testing included the addition of the sheathing to each side of the roof. The lumber used was Douglas Fir-Larch grade-marked Standard and Better. (McFadden was unable to identify post grades at the lumber yard but in conversation said they were also Douglas Fir-Larch.) The 7.32 m (24 ft.) endwall included a W-truss having 38 mm x 140 mm (1.5 in. x 5.5 in.) chords and 38 mm x 89 mm (1.5 in. x 3.5 in.) webs. The 140 mm x 140 mm (5.5 in. x 5.5 in.) posts extended to the top chords of the truss and were embedded 0.91 m (3 ft.) deep in concrete. The 2.4 m (8 ft.) and 4.8 m (16 ft.) wall girts were 38 mm x 89 mm (1.5 in. x 3.5 in.) placed flatwise, spaced 0.61 m (2 ft.) on-center, and fastened with 2-16d common nails. The metal cladding was Strongpanel with a color-coated nominal steel thickness of 0.475 mm (0.0187 in.). The cladding was fastened to the endwall with Woodgrip self-tapping screws, using #10 screws in the ribs and #9 screws in the valleys. The fastening pattern alternated with one row of girts having a screw in each rib, and the next row having a screw in each valley as well as one in every third rib. Lateral loads were applied at the eave and the deflection of the other eave was measured using an attached ruler and a transit. The metal sheathing significantly increased the rigidity of the endwall. In stage-1

with framing only, a 7.25 kN (1630 lb.) load produced a 52.2 mm (2.06 in.) deflection for one endwall for a stiffness of 139.4 kN/m (795 lb./in.). For stage-2, with the endwall cladding fastened, the same load produced a deflection of 1 mm (0.0394 in.) for a stiffness of 7,250 kN/m (41,400 lb./in.). The other endwall behaved similarly in stage-1 tests and was even more rigid in stage-2. It was acknowledged that more studies need to be conducted concerning predictions of endwall deflections under lateral loads.

2.7 Shearwall Design

Faherty and Williamson (1989) discusses diaphragm and shearwall design using plywood sheathing. A primary assumption made in diaphragm design is that the shear in the diaphragm is uniformly distributed across the depth of the diaphragm. When openings are introduced in a shearwall, and the proportion of opening size to diaphragm size is increased, secondary effects become increasingly large and change the unit shear distribution around the openings. To offset these secondary forces, collectors are required to redistribute the forces and shears.

A collector, also known as a drag strut, is a member that collects the diaphragm load and drags it back to a shearwall. In the context of shearwalls, the collector collects the concentrated loads imposed by the openings and redistributes them to the shearwall. A simple analytical approach is given to account for openings, but has not been validated by experiment.

A problem encountered in shearwall design is that the shear distribution in the plywood itself is not uniform. This problem is due to the load path in the diaphragm assembly. The load is imposed on the framing and it is then transferred from the framing to the plywood by the fasteners. Therefore, the fastener capacity is almost always critical in the design of diaphragms.

An assumption made in plywood shearwall design is that the areas above and below openings are neglected, and that the available area for shear transfer is comprised of the panels in between the openings. Further, it is assumed that the rigidity of these panels is directly proportional to their length. Often, the available length for shearwall design is significantly smaller than the depth of the diaphragm attached to it. In cases such as this, collectors are required to increase the effective length of the shearwall to that of the diaphragm depth, thus permitting an optimal transfer of shear forces.

In the design of post-frame buildings, the shear forces in the roof diaphragm are conveniently transferred to the endwall by the gable end truss and truss sheathing. Shear forces are then transferred into the endwall posts, and eventually to the ground by means of the endwall diaphragm assembly. When there is a large door in the endwall, the truss acts as a shear collector, allowing for a uniform transfer of the roof shear above the door into the endwall.

There is no direct connection between the roof sheathing and truss sheathing, except for the corner trim. At present, this trim is merely cosmetic. L. D. Luttrell¹ is seeking to develop trim that also acts as a shear connector, but is not currently available. The primary purpose of the truss sheathing in this study will be to act as a shear collector, and in turn, uniformly transfer roof shear to the endwall when a door is present. That is to say that the truss sheathing will, conservatively, be omitted from analysis.

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3. Endwall Analysis

3.1 Introduction

Flat steel strapping placed diagonally is currently being used as an endwall bracing alternative. Typically, the steel strapping is 2.54 cm (1 in.) wide and 0.8 mm (0.031 in.) thick, having punched holes for fastening. Some builders refer to it as "plumber's strapping", though it is a much stronger and heavier gage. It is a practical and economical means of bracing. However, typical design procedures make assumptions that may grossly overestimate the bracing effectiveness of the strapping. Perhaps an example would best illustrate this problem.

Figure 6 is an illustration of a 19.5 m (64 ft.) wide building with a 4.88 m (16 ft.) ceiling height. The building is to have a 4.88 m (16 ft.) door centered in the endwall. If properly designed, the minimum strength of the endwall is known based on the arguments of Section 2.5 and diaphragm testing of a typical roof panel. The design shear strength of the endwall is found to be 1.459 kN/m (100 lb./ft.). The 19.5 m (64 ft.) endwall then has a design shear capacity of 28.5 kN (6,400 lb.). A 4.88 m (16 ft.) door, full height, constitutes 25 percent of the endwall area. Hence 25 percent of 28.5 kN (6,400 lb.) or 7.12 kN (1,600 lb.) of horizontal reinforcement is needed to meet the

strength replacement criterion set forth in Objective 1. Bracing is along the full diagonal on each side of the door; that is, 7.32 m (24 ft.) horizontal and 4.88 m (16 ft.) vertical. Dividing the required reinforcement by the cosine of the bracing angle, 33.7 degrees, determines that a strapping resistance of 8.55 kN (1,923 lb.) is required. It might be concluded that a 2.54 cm (1 in.) strap having a design strength of 9.72 kN (2,190 lb.) is sufficient. Notice only one strap in each direction, on each side of the door, appears necessary. It will be shown that this design is inadequate, since the indeterminate behavior of the endwall components and resulting forces were not considered. This example of steel strap bracing is not atypical of industry practice.

Several questions can be asked about this example. How many nails are needed for the straps to carry full design load? More importantly, will the straps develop full design load? Endwall diaphragms, being indeterminate in nature, do not lend to a simple analysis that can approximate the needed resistance to be supplemented by steel strapping. An analogy may be made to braced endwall diaphragms as is shown in Figure 7 as a schematic of a cantilever beam braced by a steel cable. When rotated such that the cantilever beam is vertical, Figure 7 is representative of an endwall diaphragm, braced diagonally with steel strapping. This is an indeterminate structural assembly. If the components are not comparable in stiffness, the stiffer component will carry most of the load and could be overloaded.

Translating this analogy to the endwall diaphragm, the stiffness of the metal strapping and the corresponding fasteners, relative to the stiffness of the endwall diaphragm, determines the amount of load carried by the bracing and the metal cladding. The strapping configuration could be too stiff, to the degree that rupture may occur, because the metal cladding would not be carrying a significant amount of the load. More than likely, the strapping would be too limber to adequately carry the loads intended. The number of steel straps can be varied and, consequently, the added strength and stiffness can be varied to attain working relationships between the bracing and endwall elements.

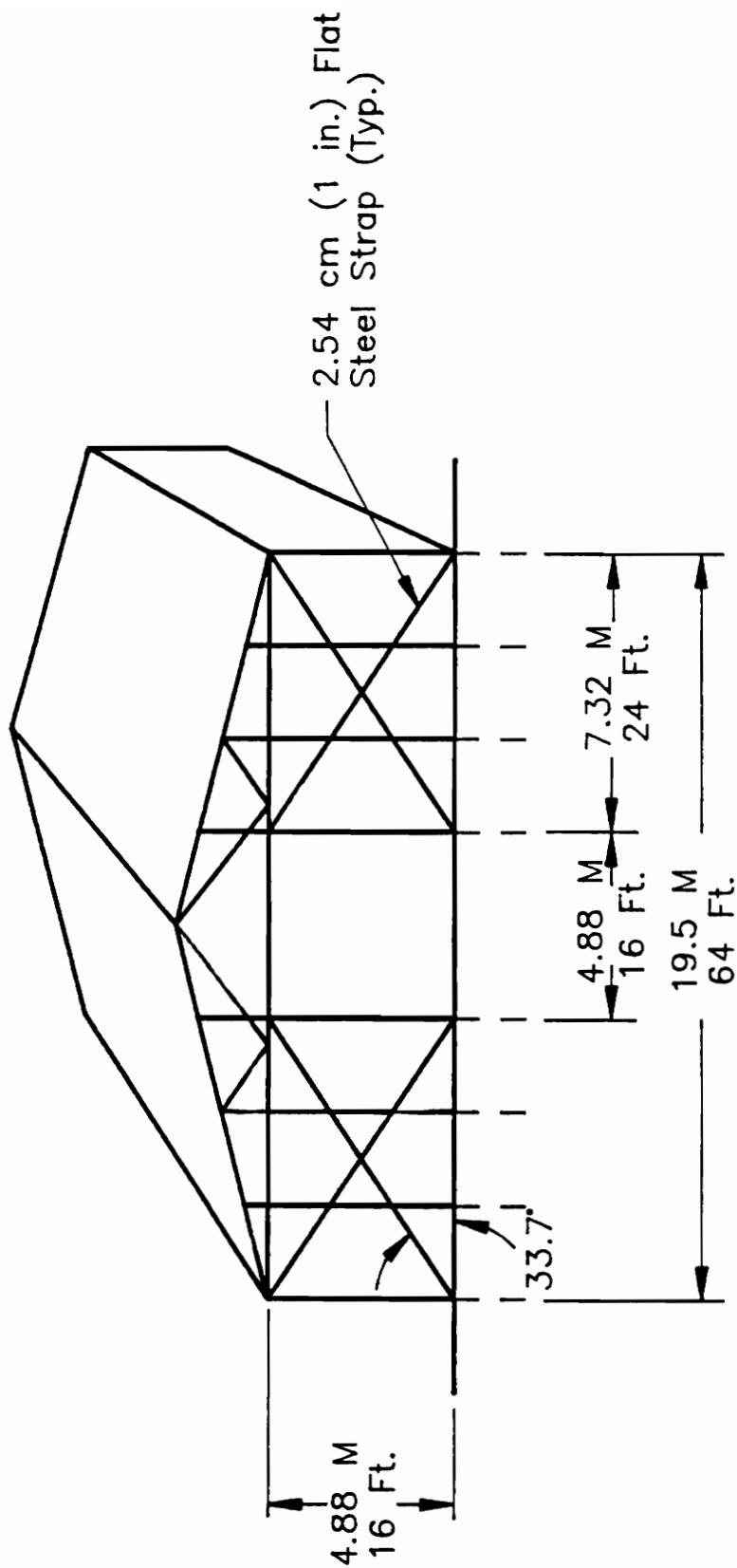


Figure 6. Illustration of a 19.5 m (64 ft.) wide building with a 4.88 m (16 ft.) ceiling height, and 4.88 m (16 ft.) wide door.

E_{rod} psi $\times 10^6$	E_{cable} psi $\times 10^6$	T_{cable} lb.
1.4	15	322
1.4	18	402
1.4	21	437
1.4	24	458
1.4	27	471
1.4	30	481

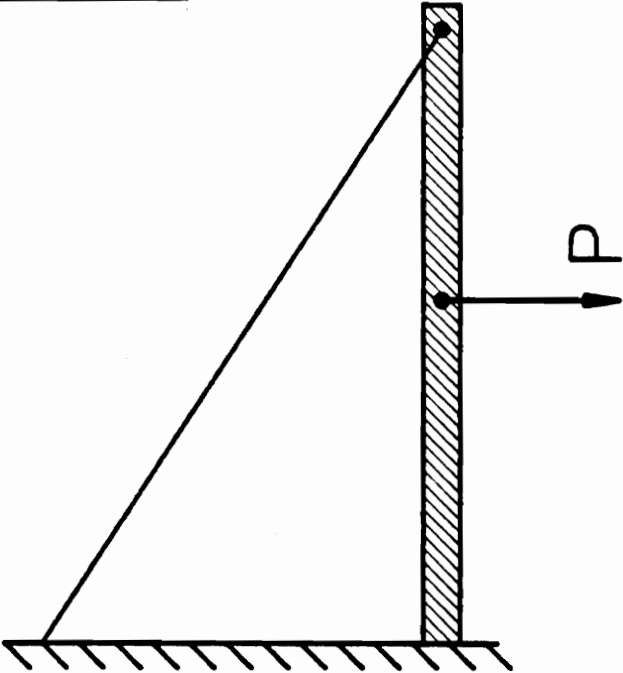


Figure 7. Horizontal analogy of an endwall braced by steel strapping: This is an indeterminate structure. The table above shows how the cable tension, T , for a 0.6 mm (1/4 in.) steel cable varies with varying stiffnesses of the cable and a wooden 14 cm (nominal 6x6) post.

ASAE Engineering Practice EP484 states that lost strength and stiffness due to endwall openings be replaced to the extent that lateral displacement at the eave be negligible under design conditions. We define "replaced strength and stiffness" as the provision of supplemental reinforcement that limits the maximum deflection at the ceiling height to that of the same endwall having no openings. Requiring confirmation, it is presumed that if the stiffness of an endwall with large doors is brought back to that of a solid endwall, using flat steel strapping, then likewise the strength of the endwall is also brought back to that of the solid endwall, provided the steel straps and fasteners are not overstressed.

It is worth noting beforehand that the metal strapping could affect the endwall by concentrating shear loads in the cladding along the lines of the strapping. The strength of the steel cladding will be adequate, as in the case of a solid endwall, if there are only small differential displacements in the endwall caused by the forces of the bracing. In other words, we will assume that the cladding and its corresponding fasteners will be no more stressed than they would be in the solid endwall. This assumption may be violated due to localized stress concentrations. However, only localized yielding would be expected, and the overall capacity of the endwall should not be affected. Because of the potential problems associated with the placing of the strapping, guidelines for strapping installation will be a significant part of this study.

3.2 Analysis Chosen

Diaphragm response to load involves variables such as strength of materials used, type of metal cladding, stitching, and fasteners. The interactions between these variables are complex and require detailed finite element analyses to determine the behavior of the diaphragm. Such an analysis is underway at the Pennsylvania State University, and in the long term, will provide the ideal solution to diaphragm design of endwalls. There is a need in the short term for analysis procedures that

provide for an engineered bracing scheme to aid in designing endwalls having large doors and openings.

The endwall analysis approach presented in this thesis is predicated on two conditions. The first being that a building construction having solid endwalls has adequate stiffness to meet serviceability requirements. Second, the roof was properly designed to resist the shear loading. Then, if the endwall and roof are of the same construction, differing only with the rib orientation of the cladding, the endwall, being at least as strong and stiff as the roof, is likewise adequate. It should be noted that the total shear force in the roof equals the total endwall shear.

This analysis is limited to various endwall sizes having a door centered in the endwall. Realizing that there are numerous endwall-door combinations needed in construction, a second consideration is to provide the designer with a computer-based structural-analysis tool that can be used for cases beyond the scope of this study.

Of the alternatives available, PPSA 3.00 was used for structural analysis. Several reasons making this program desirable include:

1. PPSA 3.00 requires only a personal computer (PC) and is commonly used in the wood construction industry.
2. PPSA 3.00 software is designed to accommodate lumber sizes and stresses.

3.3 Modeling Endwall Deflection of Framed Endwalls

Without Cladding

The first step in the endwall modeling was to calibrate a structural analog using actual test data. The results of McFadden et al. (1988) are the only endwall data available that lends to calibration of an endwall analog. The structural analog was set up matching the same material and configuration of the McFadden et al. (1988) test of a full-scale building. The posts were made rigid at the base to simulate full embedment in concrete. All members representing lumber were pinned at the ends and fixed at the nodal ends where the lumber was continuous through a node. The member ends of the truss were assumed to be rigid connections; all webs were pinned and the peak joint of the truss was pinned. A horizontal load was applied at one eave and the deflection at the other eave was calculated using PPSA 3.00. With a load of 7.27 kN (1634 lb.) applied, the endwall model resulted in an eave deflection of 26.7 mm (1.05 in.), much less than for the actual building, which was 52.2 mm (2.06 in.).

The structural analog was much stiffer than the actual endwall. Likewise, many of the girts were overstressed. The 4.88 m (16 ft.) long girts are continuous over one post location, and at that location they were assumed to be rigidly attached to the post in the analog. The post-to-girt connections at the midspan of the girts are not rigid; a joint having 2-16d nails on a 38 mm (nominal 2 in.) by 89 mm (nominal 4 in.) member will not resist significant moment without rotation. Rather, this connection is similar to a scissor joint. With this, all girts were pinned at each post, hence appearing to be all 2.4 m (8 ft.) members. When pinned, the member bending stresses were relieved and the axial loading of the girts appeared more reasonable. Figure 8 is the resulting input analog for PPSA, showing the endwall members and type of connections used for each of them.

Pinning all girt-to-post connections did little to decrease endwall stiffness, which was still on the order of two times that of the McFadden et al. (1988) data. This excessive stiffness is probably the

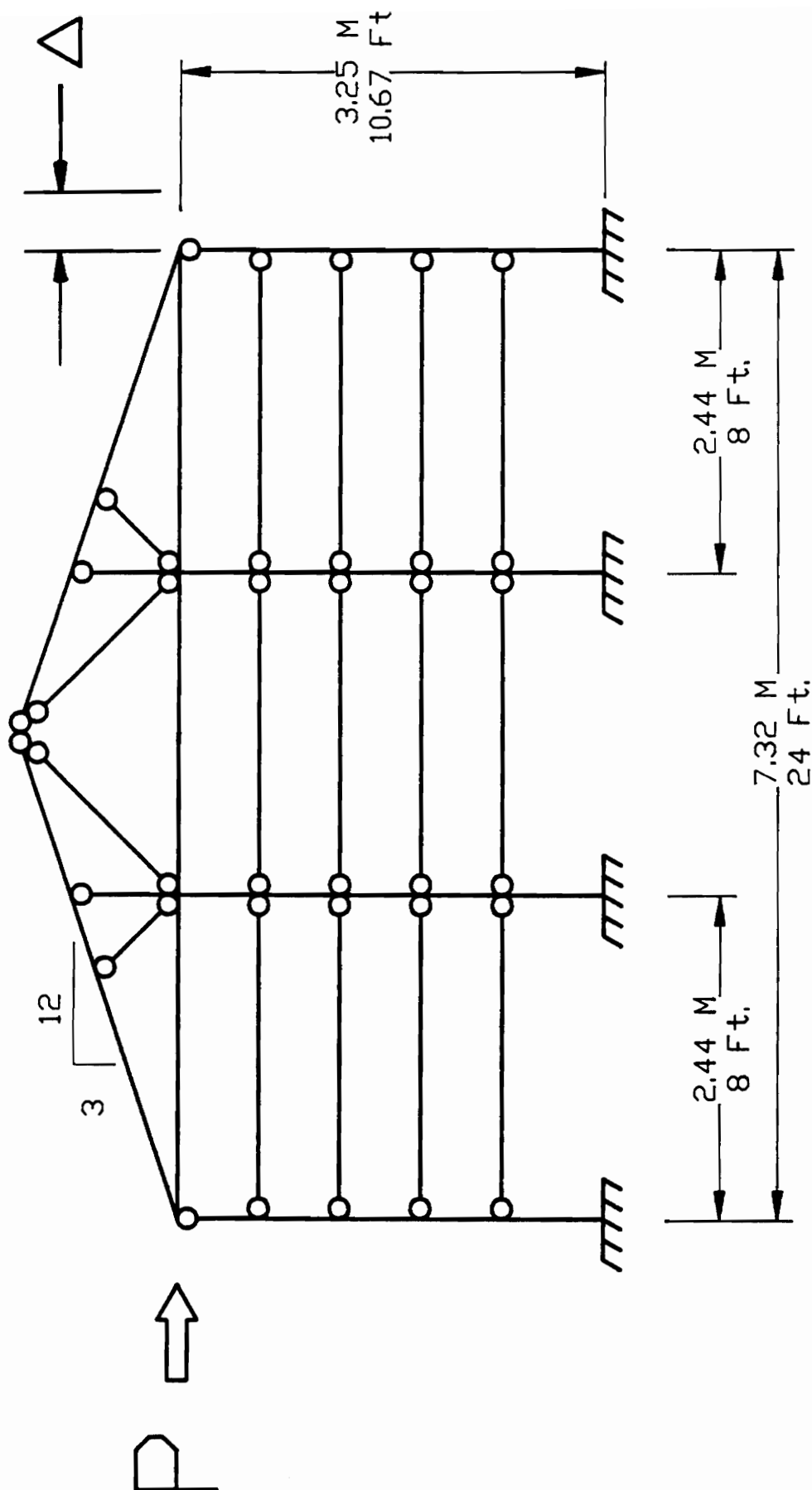


Figure 8. PPSA structural analog of the McFadden et al. (1988) framed endwall test: Load, P , was applied and deflection, Δ , was measured at the eaves. Circles denote pinned connections of various member ends.

result of the analog not allowing slip in the nailed joints as is the real behavior. Incorporating fictitious members into the model, to account for nail slip, would quickly deplete the number of members and nodes available in PPSA 3.00. Modifying PPSA is not a practical alternative when the objective is to provide the designer with a procedure that can be implemented using available software.

By systematically varying the input parameters, it was determined that a significant amount of the stiffness was attributable to the section of the posts extending up to the truss top chords. This finding gives credibility to the practice of many builders of extending the posts to the top of the truss. To simulate the actual building deflection, the top section of each of the two interior posts (from the truss top chord down to the first row of girts) was reduced in width until the eave deflection matched that of McFadden et al. (1988) for the same load. Reducing the post width was based on an arbitrary decision. By reducing the thickness of the stiffest member in the analog, the deflection of the actual endwall could be modeled without having to incorporate numerous nail slip fictitious members. This condition required thinning the posts from 140 mm (nominal 6 in.) to approximately 64 mm (2.5 in.). The thinning significantly overstressed these members, but is satisfactory since they are only acting as fictitious members. Figure 9 shows McFadden's endwall with the above modifications. With this configuration, which includes girts pinned, none of the framing members were overstressed. This procedure effectively accounts for the slip between the truss and endwall posts and the deformations resulting from the loadings in the truss and endwall framing.

3.4 Modeling the Endwall Cladding

Once McFadden's endwall framing was modeled, the next step was to model the endwall with the cladding attached. Using only beam elements in PPSA, the method that was most appealing had a fictitious diagonal brace from groundline to the truss bottom chord, between each pair of posts,

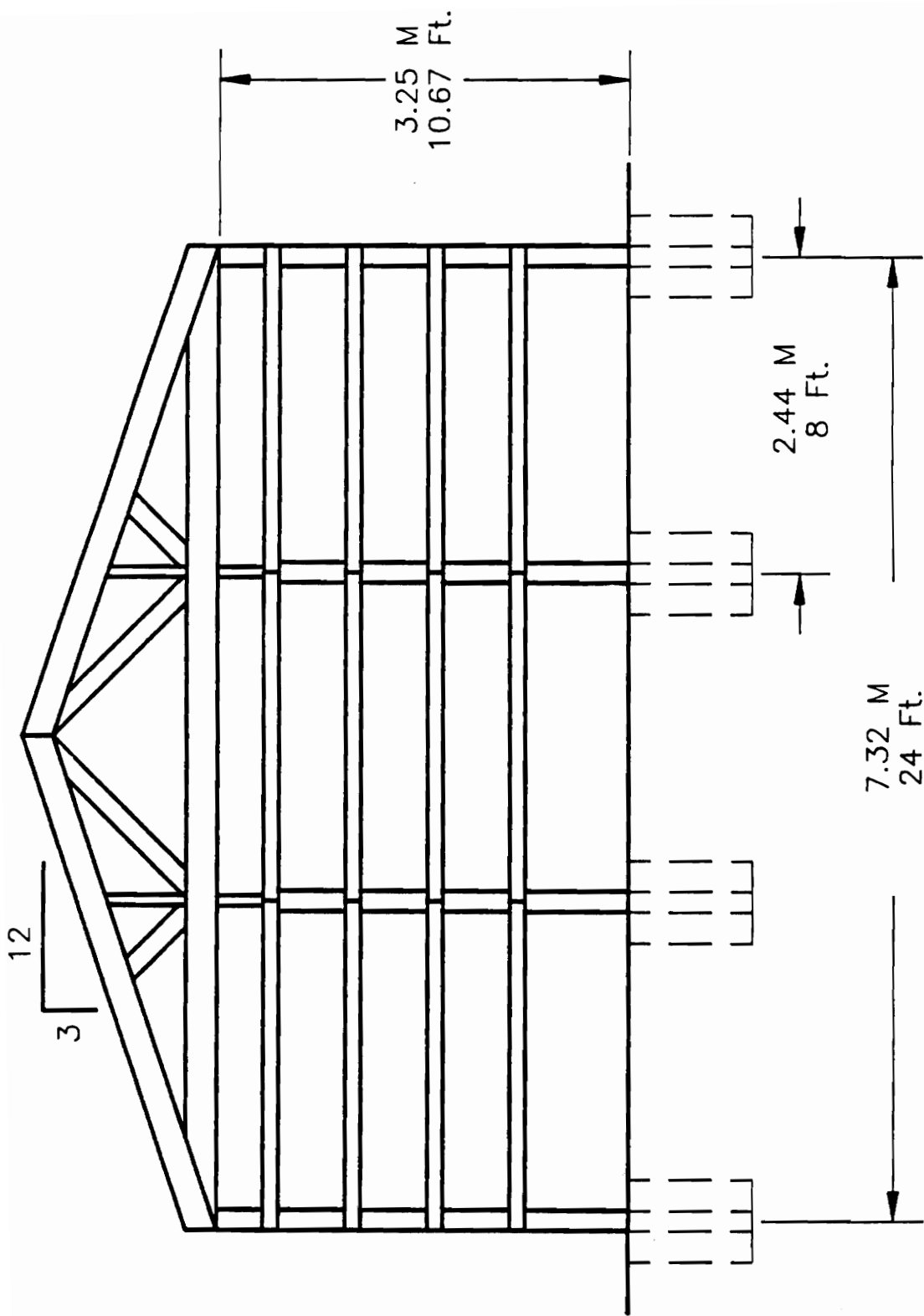


Figure 9. Posts and framing layout of endwall tested by McFadden et al. (1988): Thinning of the upper section of the interior posts was required when modeling the unclad endwall.

to simulate the bracing from diaphragm action. These members were pinned to the posts and would "mimic" a diaphragm panel 3.25 m. (10.67 ft.) tall by 2.44 m. (8 ft.) wide (Figure 10).

"Mimic" is a suitable term for this structural analog. The analog does not attempt to have shear and bending deformations like a diaphragm, rather, to produce the same end product of deflection. PPSA is a linear structural model. Likewise, in the design range of diaphragms, considered to be forty percent of the ultimate strength, diaphragm behavior is approximately linear. Consequently, for design purposes, a linear fictitious brace should sufficiently model or "mimic" the diaphragm action of an actual endwall. Note that sheathing is omitted from the truss (Section 2.7); that is, the diaphragm braces only extend up to the ceiling height.

A 2.54 cm (1 in.) by 2.54 cm (1 in.) fictitious brace was used between each endwall post to mimic McFadden's diaphragm panels. A diaphragm panel was considered to be the section of endwall sheathing between posts. By varying the modulus of elasticity (E) of this member it was found that using an E of 37,530 MPa (5,443,000 psi) calibrated the analog eave deflection to that of the actual endwall.

When comparing McFadden's framed endwall data to the clad endwall data, the cladding produced an endwall stiffness about 50 times greater than the framing only case. This observation led to the conclusion that the stiffness of the wood posts and framing makes only a very small contribution to endwall stiffness. Initially, the analog depicted in Figure 10 included the wall girts and thinned upper sections of the interior posts. When analyzed without the girts and with posts having original thickness, there was no difference in the deflection of the endwall.

In using McFadden's endwall data for other configurations, the diaphragm panel size must remain the same, that is 3.25 m (10.67 ft.) by 2.44 m (8 ft.), and 2.54 cm (1 in.) square fictitious braces having E of 37,530 MPa (5,443,000 psi). It is not known how diaphragm behavior will differ for other panel sizes. Also, to use the calibrated diaphragm fictitious brace data, the building endwall to be analyzed and designed must have a diaphragm construction that results in a panel stiffness

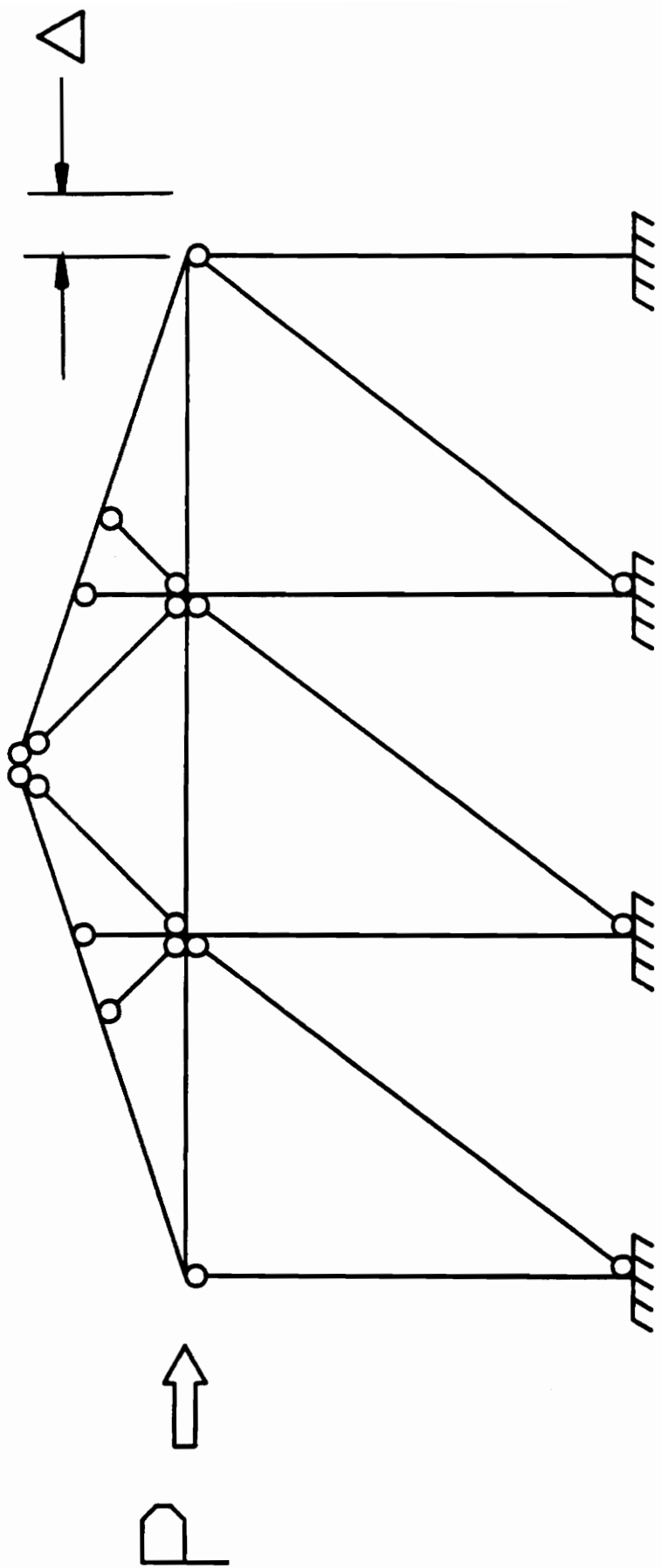


Figure 10. McFadden's endwall with fictitious diagonal members that "mimic" the diaphragm panels: The wall girts were removed and the thinned interior posts were restored to the full size because their contribution to the horizontal restraint were insignificant, relative to the fictitious diaphragm members.

comparable to the stiffness of the McFadden panels. This is a serious constraint but may be managed by employing conservative design procedures and analyzing configurations that are conservative in relation to the results of this study.

3.5 Forty-Foot Endwall

Once a model had been developed that could sufficiently mimic the deflection of an actual endwall, based on the limited data available, it could then be used to develop a working endwall analog. The first choice was to use a 12.19 m (40 ft.) endwall having a centered 7.32 m (24 ft.) door. This is the endwall-door combination having the greatest percentage of opening to be studied in this research. The wall column foundations were comprised of a pin and roller analog to more closely model an actual endwall. The 0.915 m (3 ft.) post embedment depth used in McFadden's test was chosen for analysis. If this extreme case could be handled, it was surmised that larger building widths and deeper post embedments would also work for the same door size.

3.6 Truss Considerations

A few points should be made before proceeding. To permit a comparison of an endwall having a truss to one without, the shear loading must be equivalent. A method judged to be most appropriate, was to apply equal, nodal loads at each post in the endwall, along the ceiling line (Figure 11). It is shown in Jackson et al. (1988) that by the cancelling of the cosines of the roof slope, the shear intensity along the roof slope is the same as the shear intensity horizontally along the ceiling line. Thus, nodal loading along the ceiling line for both cases allows for a comparison that is

keeping with actual loading, regardless of roof slope. Initially, the concept of tributary areas was applied so that the corner posts would receive one-half the shear load of the interior posts. However, the normal heavy nailing and bolting of the trusses at the heel joint would justify an assumption that the corner posts could carry loads equal to the interior posts. In other words, although the interior endwall posts have twice as much tributary shear load as the corner posts, the weaker connection of the interior posts should balance the shear loading among all endwall posts.

The shear intensity used in this study is arbitrary. PPSA, a structural analyzer, solves linear systems. A typical shear intensity value of 2.92 kN/m (200 lb./ft.) was chosen. When interpreting the results, all that is required of the designer is to proportion the shear to 2.92 kN/m (200 lb./ft.). For a solid 12.19 m (40 ft.) endwall, the equivalent loading is $2.92 \text{ kN/m} \times 12.19 \text{ m} / 6 \text{ posts}$ or 5.93 kN (1,333 lb.) at each post along the ceiling line. Note that for McFadden's endwall, a load was applied at the windward eave and the deflection recorded at the leeward eave. Here, the loads are applied at the nodes of the posts along the ceiling and the deflection of interest is at the node having the maximum deflection.

The endwall was analyzed with the truss, and later tested without the truss. The maximum deflection of the trussed endwall was 3.89 mm (0.153 in.), and the endwall deflection without the truss was 4.09 mm (0.161 in.). Maximum deflection for both occurred at the node located 2.44 m (8 ft.) from the windward side; that is, the second post from the left. A difference of 0.203 mm. (0.008 in.) or approximately 5 percent is considered insignificant and so the truss was omitted from subsequent analyses. The resulting analog is shown in Figure 12. Two advantages are gained with this exclusion. First, the many truss types used in construction need not be examined. Second, the quantity of nodes and members of a truss are often more than half of the total in an endwall. If used, the model soon exceeds the maximum number of nodes and members allowed by an unmodified PPSA program.

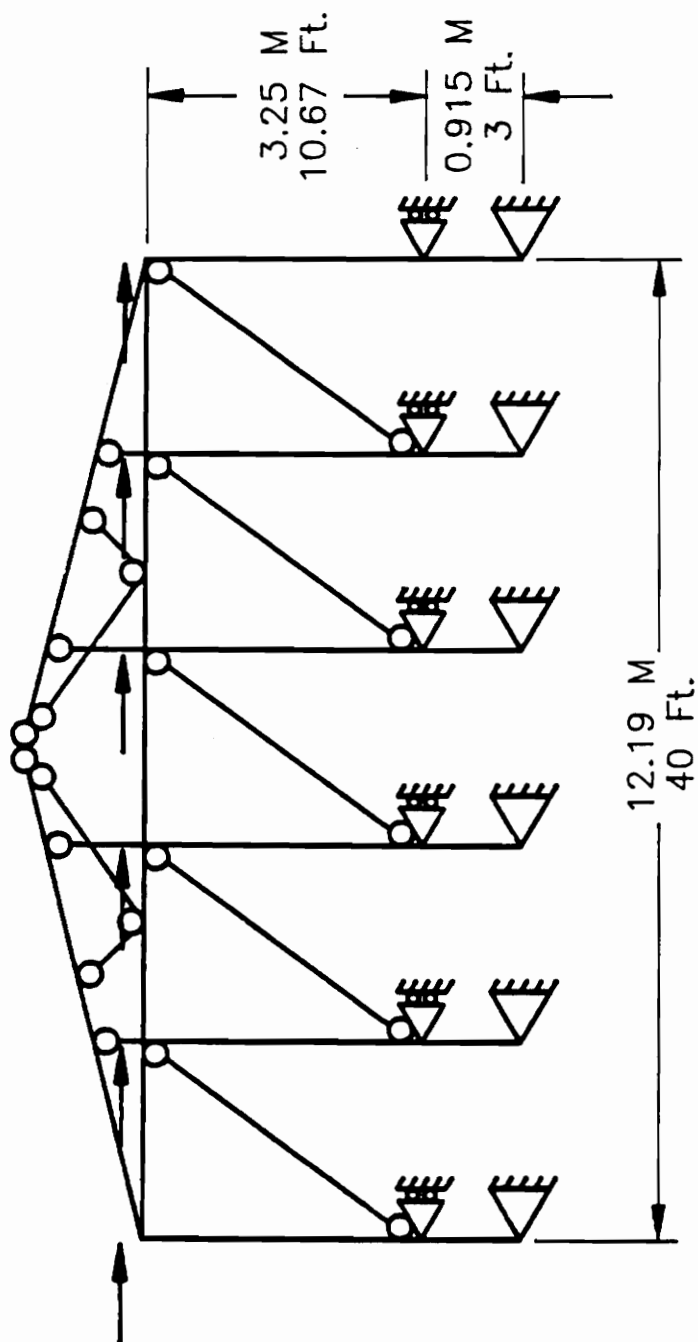


Figure 11. A 12.19 m (40 ft.) wide by 3.25 m (10.67 ft.) tall endwall analog: The roof shear was distributed equally to the endwall post nodes along the ceiling line.

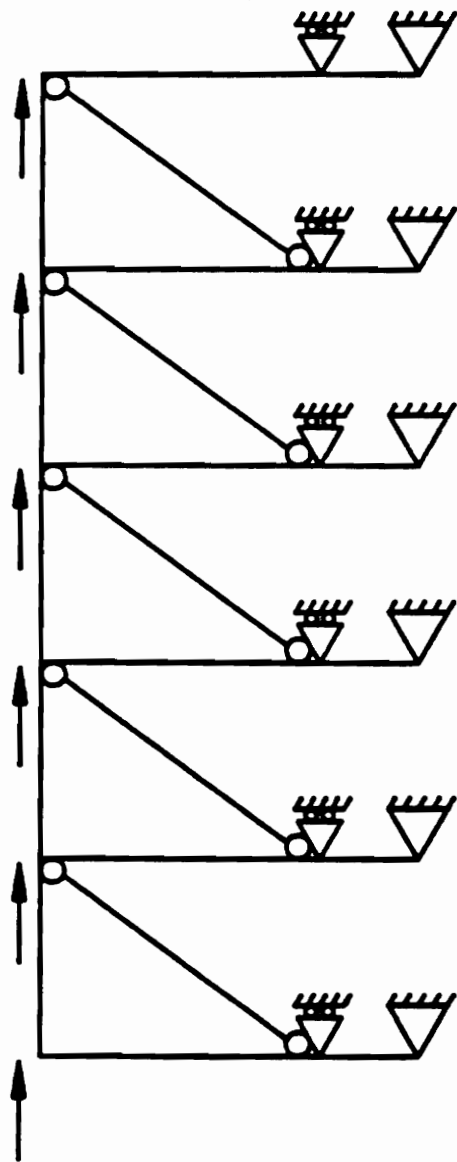


Figure 12. The 12.19 m (40 ft.) by 3.25 m (10.67 ft.) endwall analog with the truss removed: Relative to the diaphragm panel stiffness, the truss stiffness does not play a major role in restraining horizontal deflection.

3.7 Height Considerations

Next, the model would need to be studied for maximum building heights considered in this research. A 6.10 m (20 ft.) maximum ceiling height was chosen. McFadden's endwall height at the eave was 3.25 m (10.67 ft.). Unable to change the parameters used in McFadden's research, the proposed procedure is to stack two rows of panels to obtain a height of 6.50 m (21.3 ft.). The truss may still be omitted since a taller endwall would reduce the relative impact of the truss for resisting horizontal deflection. Figure 13 shows the computer analog used to model this endwall. Notice that there are horizontal members between the two rows of 3.25 m (10.67 ft.) panels. When the model was initially run, the maximum deflection of the 6.50 m (21.3 ft.) endwall was approximately 5 times greater than that of the 3.25 m (10.67 ft.) endwall. Assuming the shear deflection is linear with diaphragm length, as is plywood shear deflection, the tall endwall deflection should be twice that of the 3.25 m (10.67 ft.) endwall. The difference in predicted deflection for the two heights did not appear reasonable. Reviewing the computer results of the strength analysis, the member stresses showed that the diaphragm braces and the posts were under considerably less stress than the same members of the 3.25 m (10.67 ft.) endwall. Also, the truss bottom chords were significantly overstressed, indicating that the shear load was not being evenly distributed into the endwall. Rather, the endwall was quite limber. It appears the diaphragm braces work well to mimic a single panel. But when the second row was added, the line where they were joined did not include a member to act as the 'header' of the bottom row of panels or likewise the 'foundation' of the top row. To account for this, an extremely stiff horizontal fictitious member was placed in between the rows. The member used was a 6x6 having the elasticity of steel; $E = 200,000 \text{ MPa}$ ($29 \times 10^6 \text{ psi}$). The 6.50 m (21.3 ft.) endwall analog was run again, now having a maximum deflection of 7.95 mm (0.313 in.). The deflection for the 3.25 m (10.67 ft.) endwall was 4.09 mm (0.161 in.). Doubling this deflection gives 8.18 mm (0.322 in.) or 3 percent greater deflection than for the 6.50 m (21.3 ft.) endwall, hence near linear which is consistent with the earlier assumption of shear deflection behavior similar to plywood.

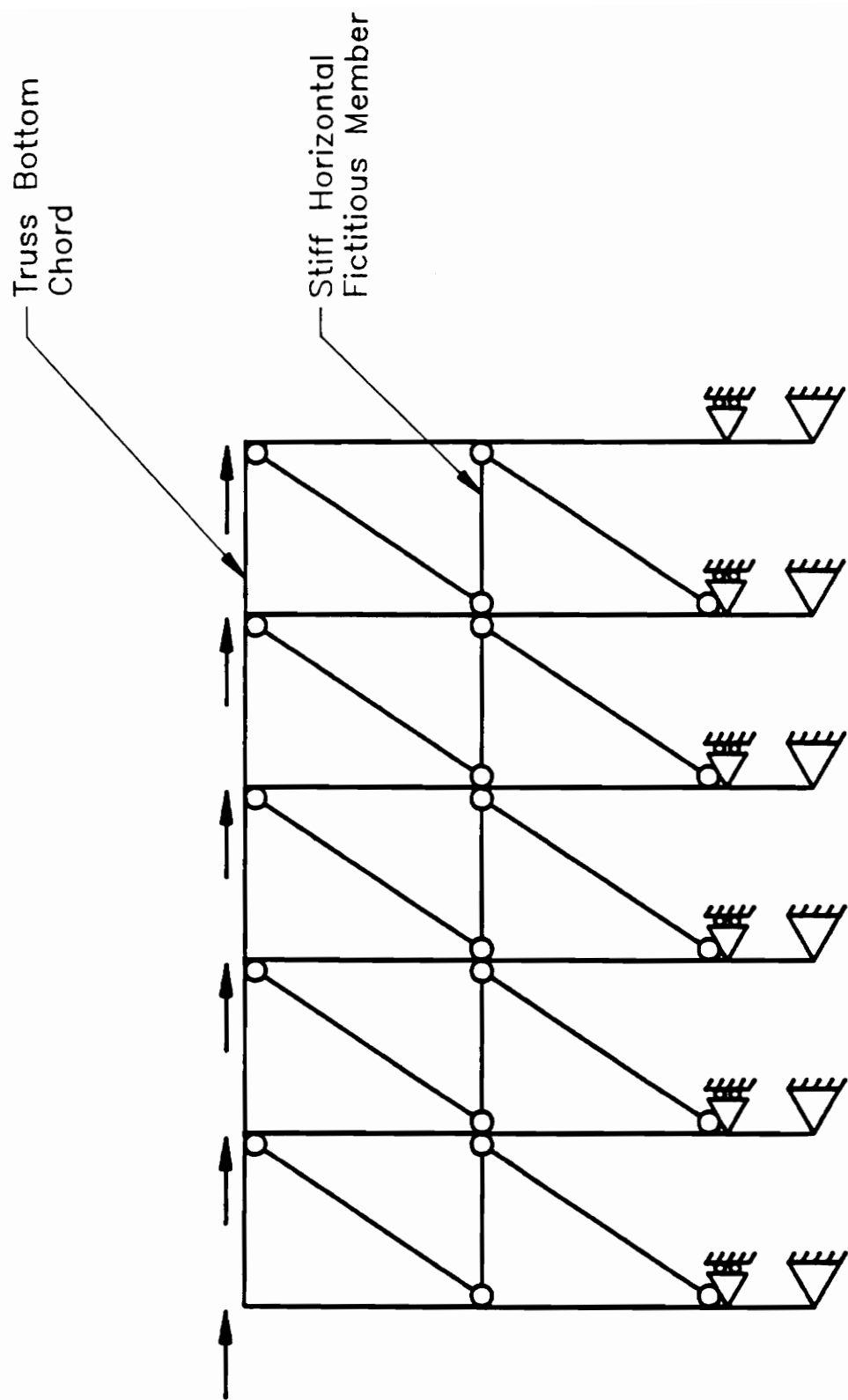


Figure 13. Stacking two rows of 12.19 m (40 ft.) by 3.25 m (10.67 ft.) tall endwalls allow for an analysis of a tall building.

The desired endwall height used for further testing was 6.50 m (21.3 ft.). The 3.25 m (10.67 ft.) endwall yielded member stresses slightly less conservative than the respective counterparts in the 6.50 m (21.3 ft.) endwall. However, it was not known whether all aspects within the study behave linearly. In other words, some aspects of the endwall may behave nonlinearly with respect to height. So overall, the tallest height studied was presumed to provide the most conservative results. Later, this will be verified by checking the 3.25 m (10.67 ft.) tall endwall for various cases to insure that the presumption was correct.

The next step was to take the 12.19 m (40 ft.) wide by 6.50 m (21.3 ft.) tall endwall and model with a door opening. Consider a 7.32 m (24 ft.) wide door. Up to this point, the modeling has been based on calibration using actual endwall data. According to the design rule proposed for when sections are removed for door openings, the subsequent deflection must be restored to that of the solid endwall. When the door height does not extend to the ceiling line, the stiffness contribution of the panels above the door is not known. Conservatively, these panels are neglected, as in Faherty and Williamson (1989), and all doors are assumed to be ceiling height (Figure 14). Recall that the truss is omitted from the design, and that the truss and truss cladding provide the means of a shear collector to evenly distribute the roof shear along the endwall. To make up for this omission, the normal size header above the door was enlarged so that the nodal loading would not localize forces. The header used in the analysis was made up of three 5.08 cm x 30.5 cm (nominal 2 in. x 12 in.) members of the same material as the truss bottom chords. The header construction selected would be common practice when a heavy sliding door is installed.

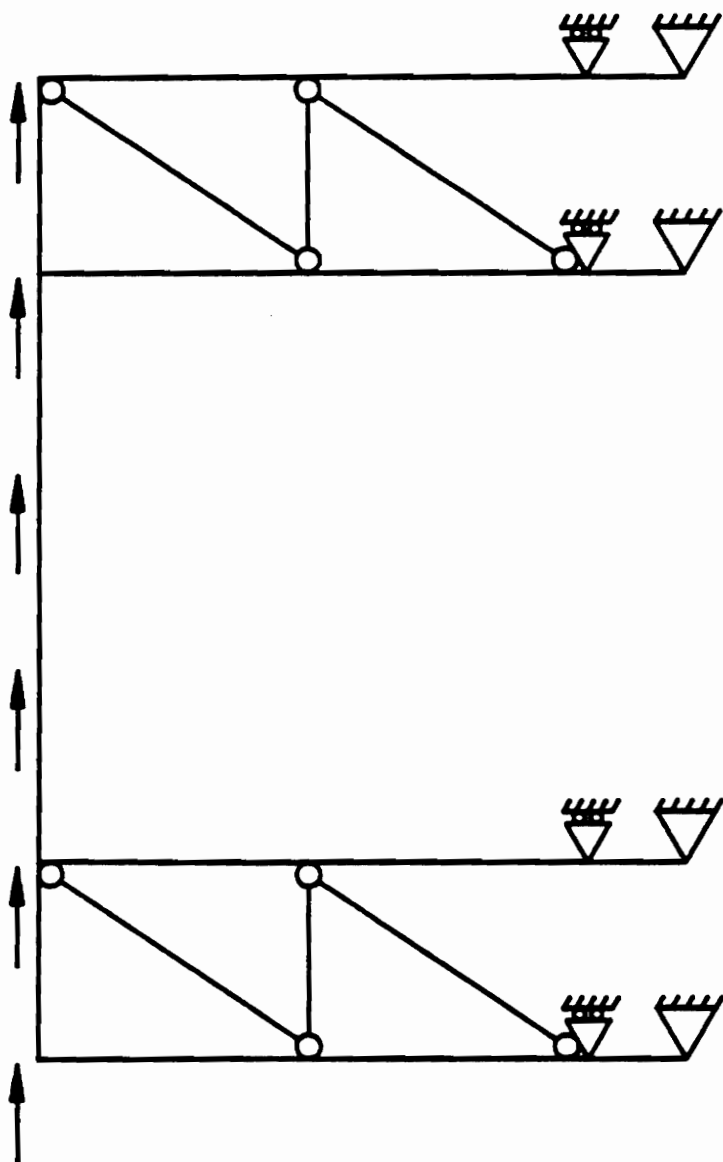


Figure 14. The 12.19 m (40 ft.) by 6.50 m (21.3 ft.) endwall analog having a 7.32 m (24 ft.) wide door opening: The header in the analog used 3-2x12's to evenly distribute the shear throughout the endwall.

3.8 Modeling the Steel Strap Bracing

3.8.1 Bracing Approach

The 3.25 m (10.67 ft.) tall by 12.19 m (40 ft.) wide endwall having a 7.32 m (24 ft.) door was the first case selected for study since it represents the greatest need for endwall reinforcement (Figure 15). There is a 2.44 m (8 ft.) wide section on each side of the door where strapping can be placed. Recall that the fictitious members used to model the endwall cladding, based on actual data, were calibrated to mimic the deflection of the endwall. All member end joints were either pinned or rigid, and nail slip was accounted for by calibrating the fictitious diaphragm members to actual endwall data. To model the steel strapping, a tension member composed of a member having the properties of steel was used. Included with this member was a fictitious element used to simulate the nail connection stiffness on both ends. It was only necessary to use a nail element on one end of the strap member and double the stiffness, to account for both ends. The entire member was then pinned at each end.

The first issue of bracing was where the strapping should be placed in the endwall with respect to the location of the door. It was assumed that placing straps between all posts, regardless of how far away from the opening, would result in uniform bracing of the remaining diaphragm. The strapping members were laid over the diaphragm fictitious members, where the optimum resistance was obtained versus other bracing schemes tested. PPSA allows for the use of multiple members in the same plane.

Two sizes of steel strapping, flat in coil, were used in this research. The properties of these straps are given in Table 3. Allowable stress design, for steel tension members, uses one-half of the tensile strength as the allowable stress or working load. These working loads will serve as a check to insure the strapping is not overloaded.

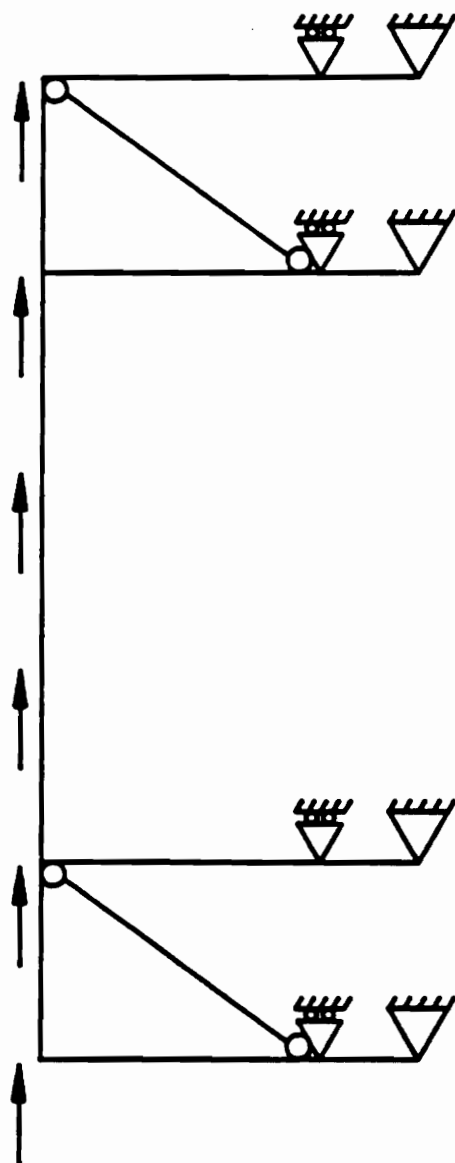


Figure 15. The 12.19 m (40 ft.) by 3.25 m (10.67 ft.) endwall analog having a 7.32 m (24 ft.) wide door opening: The steel strap bracing is placed over the fictitious diaphragm braces. The strap bracing members include a nail connection stiffness element.

Table 3. Selected properties of the flat strapping used for bracing.

Width (in.)	Thickness (in.)	Average Tensile Strength ¹ (lbs.)	Working Load (lbs.)
1.25	0.035	5,470	2,735
2.00	0.050	12,500	6,250

¹Based on recommendation of supplier.

²1 in. = 2.54 cm

³1 lb. = 4.448 kN

3.8.2 Simulating Nail Connection Stiffness

The nail connection of the bracing was next considered. It was assumed that all nails in the end of a strap equally share the strap load. The nails to the inside of the strap probably carry a higher portion of the load. An investigation of the true distribution is beyond the scope of this study. It is accepted practice in design to assume nails are equally loaded, NFPA (1986).

To simulate nail stiffness, the fictitious member included in the brace was based on the additive properties of the total number of nails in the brace. Solid treated posts are generally wet at the time of construction, so threaded hardened nails can be used to circumvent modifying the nail resistance due to high moisture content (NFPA, Table 8.1b, 1986). For convenience, arbitrarily consider the length of the nail stiffness member as 5.08 cm (2 in.). To determine the properties of this member, consider the axial stiffness of a rectangular member, $b \times d$, defined by Hoyle and Woeste (1989) as

$$\frac{P}{\Delta} = \frac{EA}{L} \quad [3.1]$$

where

P = lateral load, based on slip of 0.381 mm (0.015 in.), N (lb.)

Δ = 0.381 mm (0.015 in.)

E = Modulus of Elasticity for wood, MPa (psi)

A = cross section area of member, mm^2 (in.²)

L = member length, cm (in.)

Initially, a stiffness equivalent to ten times that of a 16d nail, on each end of a 3.18 cm (1.25 in.) strap was used to observe the bracing influence. For species Group II (Douglas Fir-Larch), the design lateral load value is given as 480 N (108 lb.) (NFPA Table 8.8c, 1986). A nail slip of 0.381mm (0.015 in.) occurs at approximately 75 percent of the design load for a wood-to-wood connection. In this study, all connections were metal-to-wood, allowing for a stiffness over the NFPA value. Because of this, the nail slip selected for the 480 N (108 lb.) load was 0.381 mm

(0.015 in.). The value used in the analog is 2 times 0.381 or 0.762 mm (0.030 in.) to account for both ends. Under normal load duration, P is calculated as 10 nails $\times$ 1.33 wind $\times$ 480 N or 6.39 kN (1,436 lb.). The corresponding Δ is 0.381 mm (0.015 in.). It was necessary to modify E because a large E resulted in b and d values so small that rounding error occurred in PPSA. The modulus of elasticity, E , used for these nail stiffness elements was 82,750 MPa (12,000 psi). The subsequent cross section area of this member was calculated from the axial stiffness relationship (Equation 3.1) to be $5,130 \text{ mm}^2$ (7.98 in.^2). Hence b and d , or width and depth, to be used was 71.6 mm (2.82 in.).

3.8.3 Optimum Nail Connection Stiffness

The strap-nail fictitious members were added to the analog of the 12.19 m (40 ft.) wide by 3.25 m (10.67 ft.) tall endwall having a 7.32 m (24 ft.) door. When a connection stiffness ten times that of a 16d nail was used for each strap end, the maximum strap load was 5.51 kN (1,239 lb.). Recall that the strap capacity was 12.19 kN (2,740 lb.) and the nail capacity 6.39 kN (1,436 lb.). It would appear that by applying more nails, a greater load carrying capacity of the strap should result. All that is required to change the number of nails in the analog is to change the corresponding lateral load, P , in Equation 3.1, thus giving the corresponding member width and depth. As the number of nails were increased, even up to an unrealistic 50 per end, neither the strap design capacity nor nail capacity could be reached. A plot of strap load versus number of nails was constructed to help understand the strap behavior. Figure 16 shows what appears to be an asymptotic limit on the strap load. Puzzling at first, it soon became apparent that as the number of nails increase, the nail connection increasingly stiffens while the strapping stiffness is unchanged. Hence at a connection stiffness approximately twelve times that for a single 16d nail on each end of the strap, the stiffness of the nailed connection was optimum with respect to the stiffness of the strapping.

From these analyses, a design recommendation can be made -- a connection stiffness equivalent to twelve times that for a single 16d hardened, threaded nail is to be used when a 3.18 cm (1.25 in.) strap is placed in each 2.44 m (8 ft.) section between posts, in the same direction, for a 12.19 m (40 ft.) wide by 3.25 m (10.67 ft.) tall endwall having a 7.32 m (24 ft.) door. Hopefully, after a series of building endwall analyses, a general procedure for endwalls with doors would unfold.

The next step was to double the endwall height. Again using the 12.19 m (40 ft.) endwall-7.32 m (24 ft.) door, the strap bracing was placed in the same manner as the diaphragm members. Two rows of 3.25 m (10.67 ft.) sections were stacked. By varying the number of nails, it is shown by Figure 16 that, as for the 3.25 m (10.67 ft.) building height, a stiffness equivalent to twelve times that of a single nail appears to be optimal per end of strap.

Another step was to investigate the nailing for the 5.08 cm (2 in.) strapping. The 3.25 m (10.67 ft.) tall endwall was used for this analysis. Figure 17 shows that a connection stiffness twelve times that of a single nail again appears to be optimal per end of strap.

Next, the 3.25 m (10.67 ft.) tall endwall was used to investigate the nailing for multiple straps. Two 5.08 cm (2 in.) straps were placed between each section. Putting two straps together is equivalent to doubling the cross section of the strap member as well as doubling the stiffness of the nail connection member. Figure 18 shows once again that a nail connection stiffness twelve times that of one nail appears to be optimal.

An interesting observation was made when more than one strap is used to brace a panel. The resistance developed in each strap was not the same as that developed for one strap per panel. In other words, doubling the number of straps in each panel, for the same load, yields less than twice the resistance of a single strap.

To exhaust the cases for nailing, the 3.25 m (10.67 ft.) tall building was extended in width to 21.9 m (72 ft.) for the case with the 7.32 m (24 ft.) door. As was expected, the curve of strap load versus

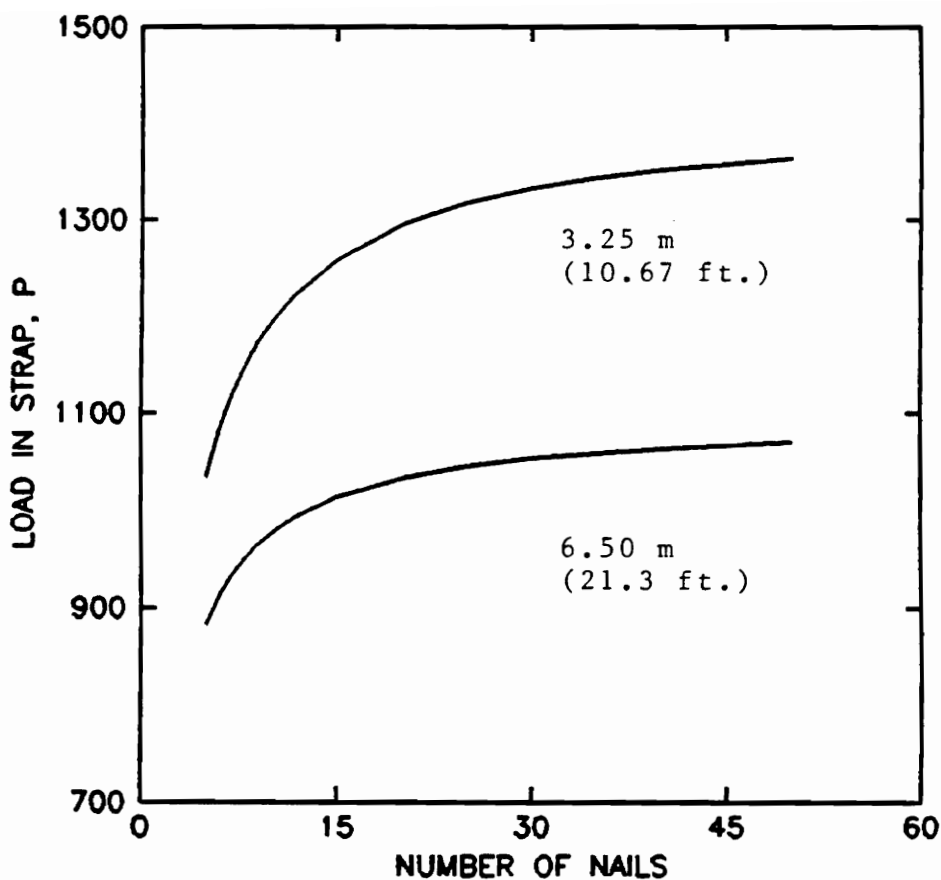


Figure 16. Strap load vs. number of nails for the 3.25 m (10.67 ft.) and 6.50 m (21.3 ft.) ceiling heights: Both cases utilized 3.18 cm (1.25 in.) steel strapping. One strap was used per diaphragm panel.

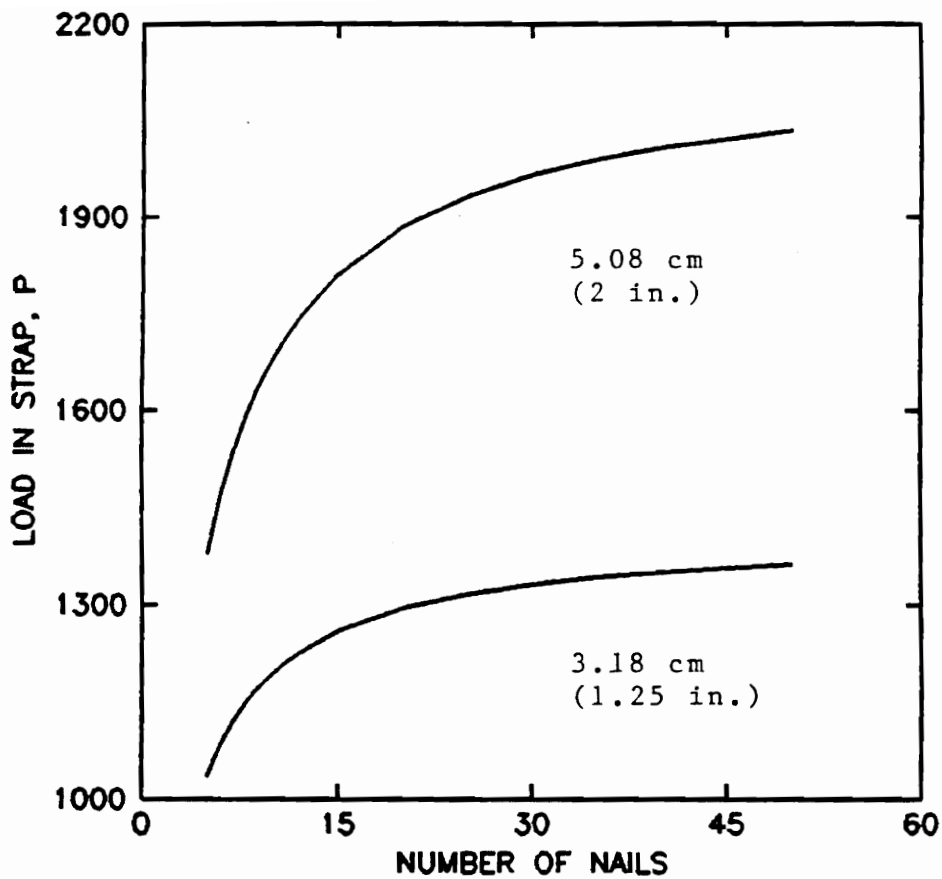


Figure 17. Strap load vs. number of nails for 3.18 cm (1.25 in.) and 5.08 cm (2 in.) straps: Endwall height is 6.50 m (21.3 ft.) and one strap was used per diaphragm panel.

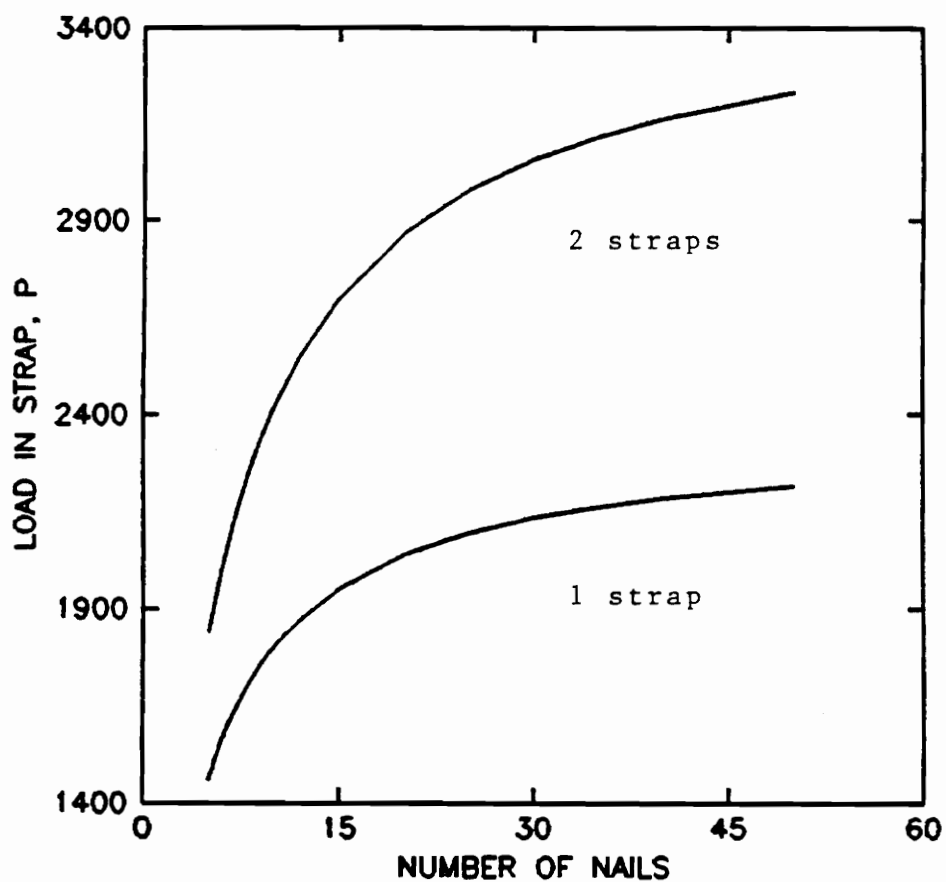


Figure 18. Strap load vs. number of nails for one 5.08 cm (2 in.) and two 5.08 cm (2 in.) straps per diaphragm panel: Ceiling height was 6.50 m (21.3 ft.).

number of nails indicated the use of a connection stiffness equivalent to twelve times that of a 16d nail. For this trial, the single 3.18 cm (1.25 in.) strap was placed in each panel section.

3.8.4 Validity of Nail Connection Stiffness

In reality, the stiffness of connections with multiple nails cannot be predicted without actual testing. Figure 19 shows the sensitivity of strap load to the stiffness of the nail connection. The upper curve is for a linear stiffness, where all nails equally contribute to connection stiffness. The middle curve assumes that each successive nail, starting from the second nail on the inside of the strap, contributes one-half of the stiffness of a single nail connection. The lower curve assumes a reciprocal contribution to connection stiffness; in other words, each successive nail contributes 1, 1/2, 1/3, 1/4, etc., of the stiffness of a single nail connection. Figure 19 gives evidence that indeed the load resisted by the steel straps is sensitive to nail connection stiffness.

The initial estimate, which assumes an equal contribution to stiffness by each nail, was used in this study. This was based primarily on a rationalization. Until there is further testing, the tie between nail connection stiffness and the optimum number of nails is obscure. Based on the results of Figure 19, it is assumed that the optimum nail connection stiffness is twelve times that for a single 16d nail connection. In no way does this research advocate 12-16d nails as an optimum nailing schedule.

At this point of the study, the ground work has been laid to begin analyzing each endwall case to determine the required bracing.

From this study, it has been learned that interactions with an endwall are complicated; common assumptions of the bracing resistance of the straps may not be valid, and structural analysis of the endwall as a whole should be performed for determining strap performance. This analysis indicates

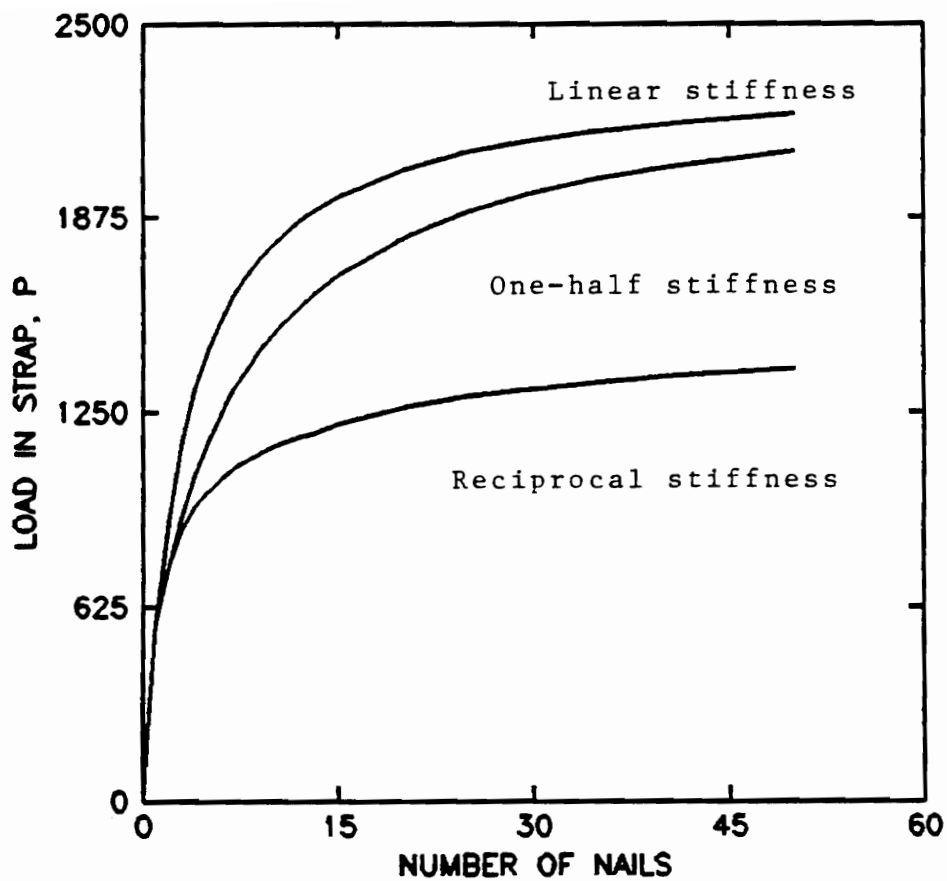


Figure 19. Strap load vs. number of nails for varied distributions of nail stiffness.

that straps never develop full strength when nailed to posts, nor do they develop a predictable loading.

The final check for performance is to determine if any components in the endwall are overstressed. Indeed, for cases of differing heights, widths, number of straps, and size of straps, the posts and diaphragm fictitious members of the braced open endwalls are always lower than their counterparts in the solid endwall.

Strap bracing likely produces concentrated loadings where they are fastened. Consequently, due to bracing, as well as door openings, there is non-uniform shear movement through the endwall. Some local deformation could result in an endwall. It is not known, but the possibility of occurrence should be brought to attention in this study.

4. Results and Discussion

4.1 Strapping Schemes

Once the groundwork for the endwall analysis procedure was established, a remaining question was what endwall width and door width combinations should be studied. The scope of this study was to determine the required bracing for endwalls having single openings centered in the endwall. Several constraints dictated the combinations chosen for study. Complying to McFadden's endwall specifications, the post spacing was 2.44 m (8 ft.). Typical large door sizes for 2.44 m (8 ft.) post spacing are 4.88 m (16 ft.) or 7.32 m (24 ft.) widths. A 9.75 m (32 ft.) wide endwall was the minimum width that would allow a 4.88 m (16 ft.) door and still have a diaphragm on each side. At the other end, 24.4 m (80 ft.) was considered to be the maximum economical span in a post-frame building. The only constraint remaining was the height, which was limited to either 3.25 m (10.67 ft.) or 6.50 m (21.3 ft.).

Table 4 gives the required bracing for the endwalls considered in this study. The 14.63 m (48 ft.) endwall having a 4.88 m (16 ft.) door in Figure 20 illustrates the optimal bracing scheme for building heights not greater than 3.25 m (10.67 ft.). The number of straps designated in Table 4

refer to the number of straps between each post, in both directions, per row of bracing. One row refers to one 3.25 m (10.67 ft.) tall section of an endwall. For ease of discussion, the number of straps in each bracing location is illustrated by Figure 20, in the ballooned portion. Table 4 indicates that 2-5.08 cm (2 in.) straps are required at each bracing location, as shown by Figure 20. Figure 21 shows that for building heights taller than 3.25 m (10.67 ft.), a second identical row of strap bracing is required. Again, 2-5.08 cm (2 in.) straps are required at each bracing location. The endwall deflections and diaphragm loads are included in Table 4 to verify that the loads and deflections are equal to or less than that of the respective solid endwalls (see presumption in Section 3.1).

It is interesting to contrast the typical industry bracing practice with the results of this study. An example was given in Chapter 3 where a 19.5 m (64 ft.) endwall having a 4.88 m (16 ft.) door had 2.54 cm (1 in.) straps, one in each direction, on each side of the door; totalling four straps for the entire endwall. If the ceiling height was in the 3.05 m (10 ft.) to 3.66 m (12 ft.) range, then Table 4 indicates that the required bracing should consist of one 5.08 cm (2 in.) strap between each post, in both directions, on both sides of the door, for a total of 12 straps. By one example of typical practice, three times the number and twice the strap size is needed to brace the endwall.

This study indicates that many more straps are required than are currently used in construction. The critical importance of diaphragm performance is illustrated by the endwall examples studied. Removing even modest sections from the endwall diaphragm result in a severe loss in stiffness of the endwall.

An important consideration in determining the required bracing scheme is what number of straps is a reasonable maximum. Typically, braced endwalls have one strap in each direction, on each side of the door. Preliminary runs indicated that one or two straps in each direction was often insufficient bracing for endwalls with doors, based on the proposed rule in this thesis. That is, the deflection of an endwall with an opening must be restored to that of the same endwall that is solid. After interviewing several builders, it was decided that no more than three straps between each post,

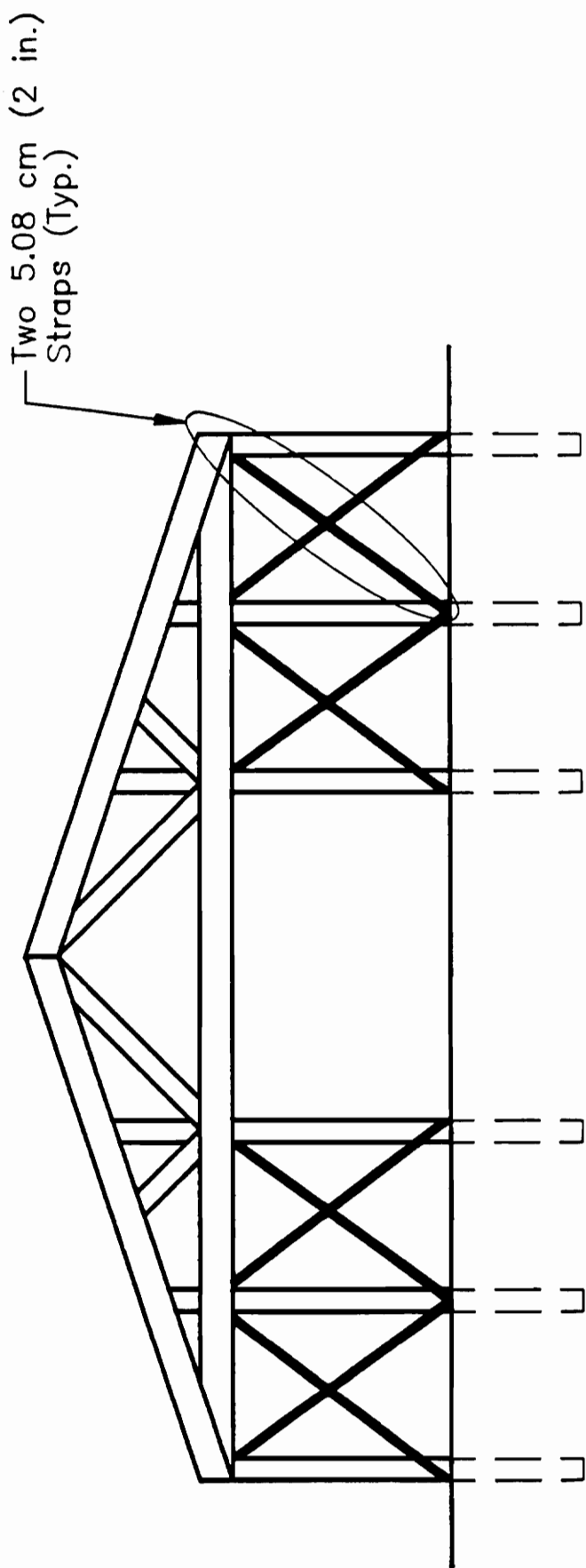


Figure 20. Optimal bracing pattern for a 3.25 m (10.67 ft.) building height.: Two 5.08 cm (2 in.) straps are required at each bracing location, shown in the ballooned portion.

Two 5.08 cm (2 in.)
Straps (Typ.)

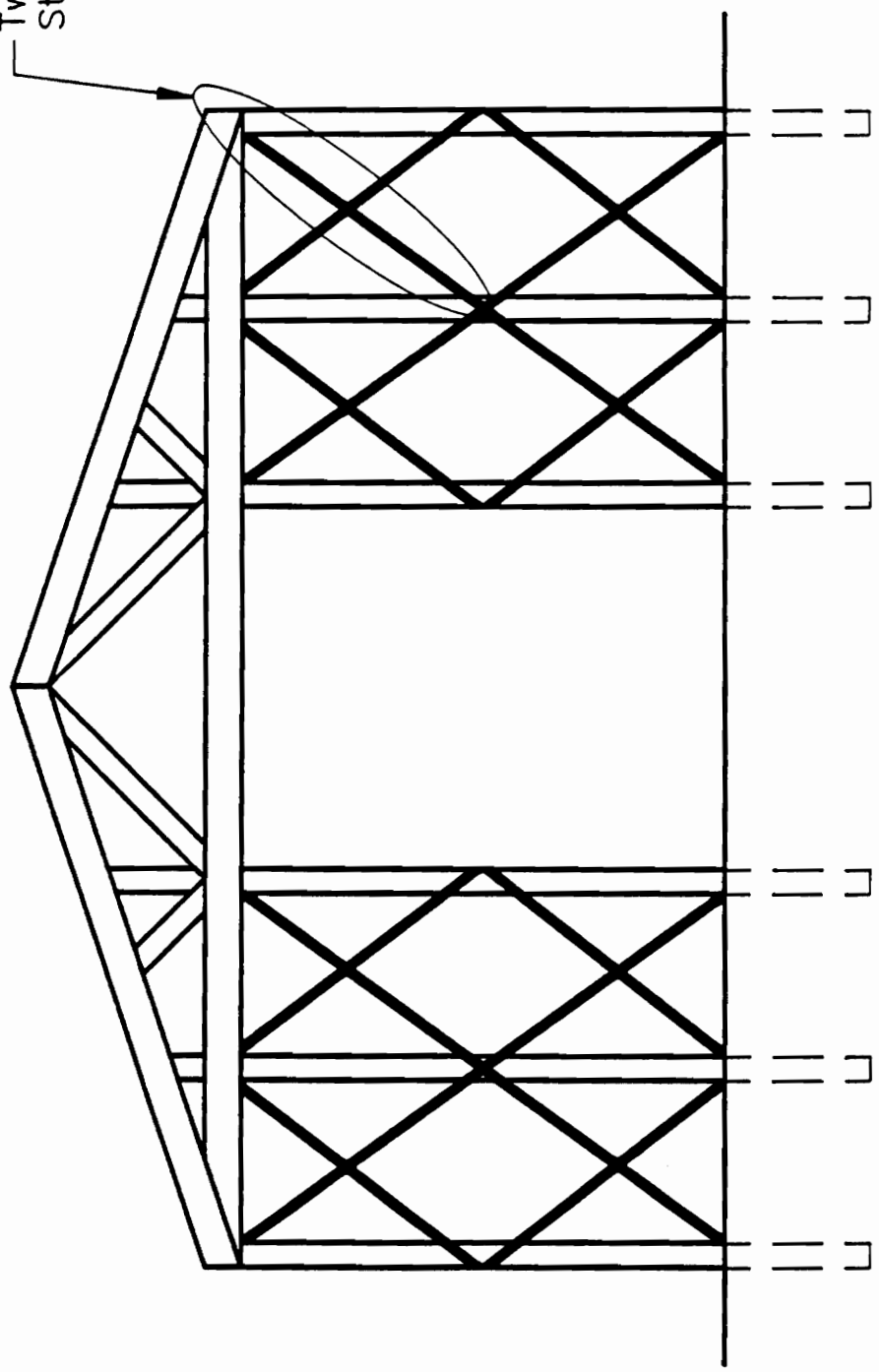


Figure 21. Optimal bracing pattern for a 6.50 m (21.3 ft.) building height.

in each direction, on an endwall with openings would be a viable solution. Placing more than three straps in this manner may be encounter space limitations. Also, the total number of straps, especially in wider endwalls, could reach a ridiculous magnitude.

TABLE 4. Results of PPSA analysis on bracing requirements for endwalls having openings:
Building heights above 10.67 ft. require a second identical row of strapping.

Endwall Width (ft)	Door Width (ft)	Number of Straps	Strap Size ² (in)	Maximum Uplift (kips)	Maximum Target Deflection (in)	Maximum Braced Deflection (in)	Maximum Target Diaphragm Load (kips)	Maximum Braced Diaphragm Load (kips)	Maximum Load Per Strap (kips)
32	16	6	2	8.67	0.308	0.308	2.79	1.573	0.644
40	24	12	2	10.96	0.310	0.307	2.81	1.162	0.475
48	16	2	2	6.66	0.311	0.295	2.83	2.26	1.065
56	24	3	2	7.85	0.313	0.297	2.86	2.19	1.032
64	16	2	1.25	5.79	0.315	0.314	2.89	2.74	0.590
		1	2	5.79	0.315	0.312	2.89	2.71	1.230
72	24	3	1.25	6.52	0.316	0.311	2.91	2.67	0.600
		2	2	6.53	0.316	0.284	2.91	2.37	1.082
80	16	2	1.25	5.39	0.318	0.294	2.93	2.54	0.553
		1	2	5.39	0.318	0.292	2.93	2.51	1.142

¹ Number of Straps denotes the number between each post, in each direction (each bracing location).

² Strap thicknesses for the 2 and 1.25 in straps are 0.05 and 0.035 in, respectively.

³ Loads and deflections based on an endwall shear of 200 lb./ft.

⁴ 1 ft. = 0.3048 m

⁵ 1 kip = 4.448 kN

Steel strap bracing was not a suitable means of reinforcement for all cases studied. The 9.75 m (32 ft.) endwall-4.88 m (16 ft.) door case required six 5.08 cm (2 in.) straps between each post to bring back the deflection. The 12.19 m (40 ft.) endwall 7.32 m (24 ft.) door case required twelve 5.08 cm (2 in.) straps between each post. Referring to Figure 21 for an illustration of bracing locations, a total of 48 straps would be required for the 9.75 m (32 ft.) wide endwall, 6.50 m (21.3 ft.) tall. Likewise, 96 straps would be required for the 12.19 m (40 ft.) endwall of the same height. Steel strapping is not practical for bracing these endwalls.

A more general statement can be made from these two cases. Endwalls having a large portion, approximately 50 percent or more, of their endwall devoted to doors or large openings, render strap bracing an impractical means of reinforcement. This statement is based on the opinion that more than three braces between each post, in each direction, is not a practical solution to bracing.

The remaining endwall-door combinations fell within the criteria of a maximum of three straps per bracing location. That is to say that one, two, or three 5.08 cm (2 in.) straps per bracing location would bring back the maximum endwall deflection to that of a solid endwall. Instead of using 5.08 cm (2 in.) straps, the 19.51 m (64 ft.) through 24.4 m (80 ft.) wide endwalls could also use 3.18 cm (1.25 in.) wide straps for reinforcement.

Perhaps the most variable dimension of endwalls is the ceiling height. Since McFadden's panel size could not be altered, the endwall heights for Table 4 were based on multiples of 3.25 m (10.67 ft.). The vast majority of buildings being constructed have heights other than a multiple of 3.25 m (10.67 ft.). A design rule for bracing requirements for varying endwall heights can be adopted: if the number of straps in Table 4 is two, then for any endwall height less than the stated height, two straps per bracing location are required. Depending on the endwall height, what will vary is the bracing angle or whether multiple rows of bracing are required.

There are three practical regions of building heights that must be considered:

1. 2.44 m (8 ft.) to 3.25 m (10.67 ft.),

2. 3.25 m (10.67 ft.) to 6.50 m (21.3 ft.),
3. 6.50 m (21.3 ft.) and above.

Considering the strap bracing angle, as long as the angle is not steeper than the angle used in this study, the bracing capacity will be equivalent to, or exceed, the cases in Table 4. A height of 2.44 m (8 ft.) is typically the minimum endwall height for post-frame buildings. Any endwall taller than 6.50 m (21.3 ft.) is beyond the scope of this study. For heights between 2.44 m (8 ft.) and 3.25 m (10.67 ft.), the strap bracing should be placed between the foundation and the full ceiling height in the same manner used in Figure 20. For heights between 3.25 m (10.67 ft.) and 6.50 m (21.3 ft.), two identical rows of strap bracing should be placed in the endwall in the same manner used in Figure 21.

4.2 Corner Post Uplift

When the 6.50 m (21.3 ft.) height was being studied in the experimental design phase, high corner post uplift was observed on the windward side of the endwall. Table 5 presents the corner post uplifts for all cases studied.

Table 5 also presents the percentage difference of uplift for the two heights. On the basis of equivalent height, the maximum difference is approximately 6 percent, and averages about 3 percent. Considering the uncertainty inherent to soil resistance values, the differences are negligible for design purposes.

Table 5. Comparison of corner post uplift for 10.67 ft. and 21.3 ft. endwalls ¹

Endwall Width (ft)	Door Width (ft)	10.67 ft. Corner Uplift (kips)	10.67 ft. Uplift Doubled (kips)	21.3 ft. Corner Uplift (kips)	Difference (percent)
32	16	4.46	8.92	8.67	2.88
40	24	5.69	11.38	10.96	3.83
48	16	3.37	6.74	6.67	1.05
56	24	3.92	7.84	7.85	0.01
64	16	3.01	6.02	5.79	3.97
72	24	3.35	6.70	6.53	2.60
80	16	2.85	5.70	5.39	5.75

¹ Comparisons based on bracing requirements using 2 in. straps.

² Corner post uplift based on an endwall shear of 200 lb./ft.

Concerning post uplift, there is a discrepancy between analysis results and the results implied by practice. An endwall analysis using a plane analog requires that the sidewalls be excluded. In an actual building, Figure 22 shows that the corner post uplift is resisted by shear loading of the adjacent sidewall diaphragm. Assuming that the sidewall is a rigid diaphragm, it will behave as a diaphragm panel in a cantilever type test, where the corner post provides the loading. This shear load is then distributed to the interior posts of the sidewall. And so, not only does the endwall diaphragm resist horizontal shear loading from the roof, sidewall diaphragms (and endwall as well) also resist the vertical uplift forces on the corner post.

The highest uplift force occurred when only one diaphragm panel was on each side of the door. The calculated uplift values on the 9.75 m (32 ft.) and 12.19 m (40 ft.) endwalls were 38.6 kN (8.67 kips) and 48.8 kN (10.96 kips), respectively. In comparison, the calculated uplift on the solid endwalls for these cases were respectively 18.68 kN (4.20 kips) and 18.73 kN (4.21 kips.) A problem encountered with the 9.75 m (32 ft.) and 12.19 m (40 ft.) endwalls was that there was only one diaphragm panel on each side of the door, which probably is not a typical construction. For both cases, the shear resulting from the uplift would be resisted by the endwall, and the sidewall when the endwall has a significant number of panels on each side of the door opening. However, by the endwall analysis, on each side of the door, the posts on the windward side of the panels are in tension and the posts on the leeward side of the panels are in compression, thus not permitting a sharing of the uplift by the interior endwall posts.

The distribution of the uplift load to the inner posts in the sidewall is not equal and it almost certainly decreases on each successive post from the endwall to the center of the building. To visualize the decreasing profile of load from the corner posts to the center, it is helpful to consider the sidewall as a beam on an elastic foundation, or springs. The embedded posts act as springs. Figure 22 shows that a sidewall will act as a long beam due to the uplift loading by the corner posts, and thus curvature results. The curvature of the sidewall will cause the post adjacent to the corner post to carry less load than the corner post. The next inner post will carry even less load, and so on.

Since the actual post loading distribution is not known, it is assumed for analysis and design purposes that the load is distributed equally among the sidewall tributary posts.

The term "tributary posts" refers to one half the sidewall posts nearest the corner, and the endwall posts nearest that corner that are in tension. If there are interior endwall posts in uplift, between the corner post and the door, then it is recommended that they may be included when the corner post uplift is distributed to tributary posts.

Of the remaining scenarios, the 17.07 m (56 ft.) wide endwall having a 7.32 m (24 ft.) door had a 34.9 kN (7.85 kip) corner post uplift load. Practically speaking, this is nearly four tons of uplift force on the corner post. When using this value in the design of foundation resistance to uplift, it may be difficult to attain adequate resistance in most soils. In the following discussion, the tributary posts on the sidewall are studied. Figure 23 shows that none of the interior posts in the endwall on the windward side of the door are in tension. And so, for the 17.07 m (56 ft.) wide building, no interior endwall posts may be considered as tributary posts. However, some of the other cases do have interior endwall posts adjacent to the corner that are in tension. If the designer chooses to use this procedure to perform an analysis, it is reasonable to include interior endwall posts that are in tension if they are between the corner post and the door.

If the roof shear is higher than 2.92 kN/m (200 lb./ft.), the post uplift will be higher in the same proportion. To determine the corner post uplift for a specific design problem, interpolation of the corner post uplifts in Table 5 is necessary. The value for post uplift should be based on the roof shear of 2.92 kN/m (200 lb./ft.), that was used to generate the results of Table 5, and so it will also be necessary to proportion the design roof shear value to 2.92 kN/m (200 lb./ft.). For example, consider a 19.51 m (64 ft.) wide endwall having a 4.88 m (16 ft.) door width, a 4.27 m (14 ft.) ceiling height, and a 3.39 kN/m (232 lb./ft.) design roof shear. Interpolating between the 3.25 m (10.67 ft.) and 6.50 m (21.3 ft.) height values for corner post uplift result in 17.26 kN (3.88 kips) of uplift at the corner post of the 4.27 m (14 ft.) height. Next, proportioning the design roof shear to the

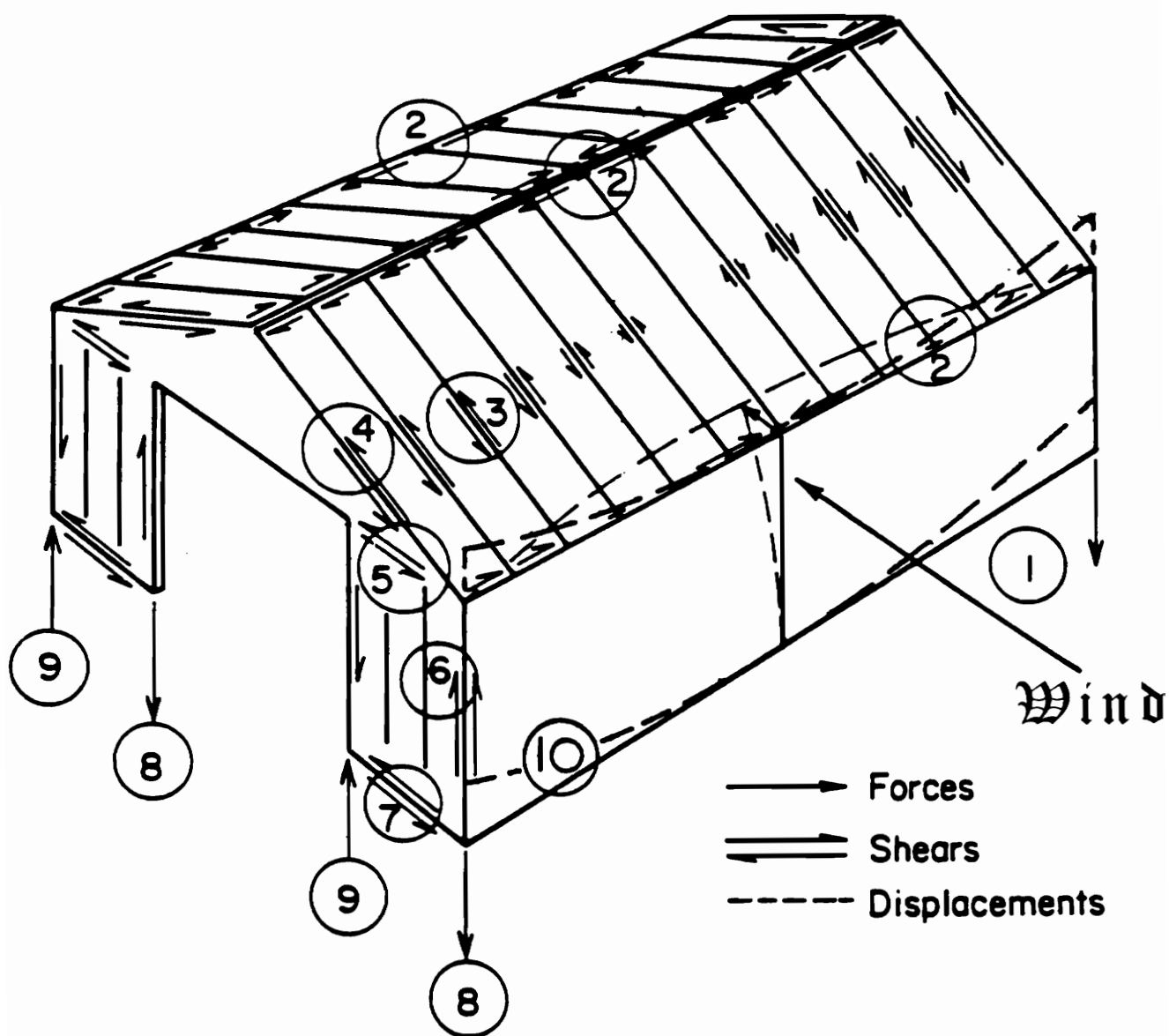


Figure 22. Sidewall acting as a beam on an elastic foundation and resisting corner post uplift: (Taken from ASAE EP484).

1. Assumed critical wind direction.
2. Edge-purlin splice for tension caused by roof bending moment.
3. Metal roofing side lap shear connection.
4. Roofing-to-endwall truss shear connection.
5. Truss-to-endwall shear connection.
6. Endwall cladding shear translates to post uplift.
7. Shear transfer, endwall-to-ground.
8. Uplift anchorage, post-to-ground.
9. Footing reaction (bearing on soil).
10. Sidewall acting as a long beam on an elastic foundation.

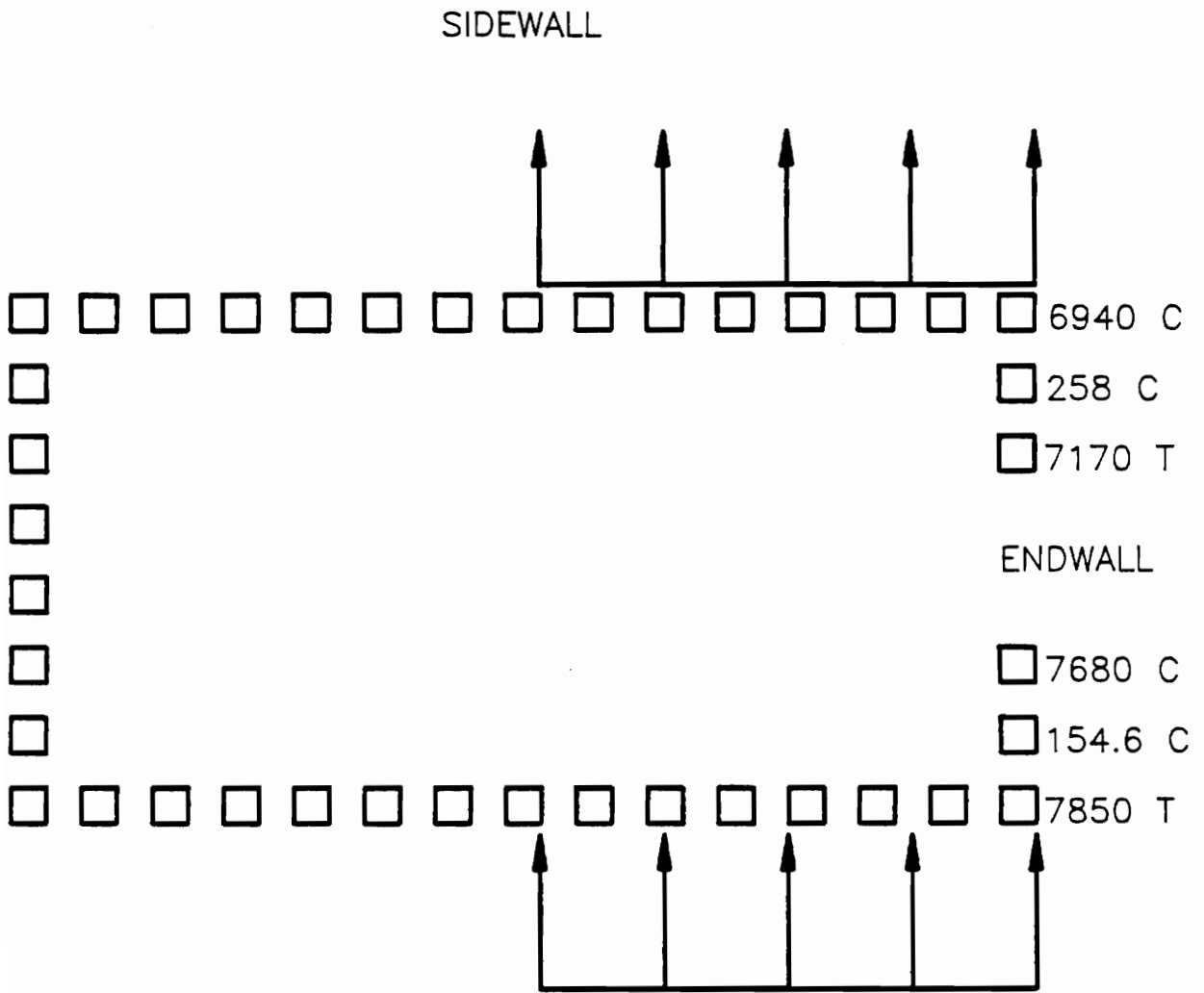


Figure 23. Plan view of 56 ft. x 112 ft. post-frame building having a 24 ft. door in one endwall: The post spacing is 8 ft., axial loads at the base of the posts are given and are based on computer analysis of an endwall that is 21.3 ft. tall, having a 200 lb./ft. shear loading. This shear intensity results from one-half of the total sidewall wind loading.

shear that Table 5 was based on: $232 / 200 \times 3.88$ or 20.0 kN (4.50 kips). And thus for design purposes, the corner post uplift for this endwall may be taken as 20.0 kN (4.50 kips).

4.3 Sidewall Contribution to Uplift Resistance

The expected path of wind load forces, based on this study of structural analogs of post-frame buildings, may be used to aid in the design of endwall posts for uplift. Consider how design wind pressure is obtained in Hoyle and Woeste (1989), where the roof wind pressure is equal for both slopes of a symmetrical roof. Thus for simplicity of illustration, the roof wind pressures cancel, and the only loads contributing to horizontal loading of the structure are the wind loads on the windward and leeward walls. From this analysis, the sidewall loading produced by wind, that is not transferred directly to the ground is transferred to the roof diaphragm, to the gable end trusses, into the endwalls, and into the posts at ground level. This load is then transferred to the ground through shear and compression and tension on endwall posts. At least one corner post is subject to a large uplift force, and this force is resisted by the sidewall posts through diaphragm shear action of the sidewall panels. This analysis and theory is offered for design purposes and has not been confirmed by experiments on actual buildings. But, based on these structural analyses alone, there is sufficient evidence of the role of the sidewall in resisting corner post uplift to recommend that large doors not be placed in the sidewall at the corner post, unless special consideration is given to corner post uplift.

4.4 Foundation Resistance to Uplift

4.4.1 A Design Example

A primary concern with this wind loading is whether the post-to-ground connection has the capacity to resist the uplift loads. Consider a building having a 2:1 length to width ratio. For the same reason of maximum uplift of the endwall cases being considered, use the 17.07 m (56 ft.) wide endwall. Figure 23 shows a plan view of the post configuration. The net uplift load on each sidewall post of the windward wall is equal to the post uplift resulting from a distribution of the corner post uplift, plus the uplift from the roof, minus gravity loads of roof and framing dead loads. The corner post uplift force is based on the results in Table 5. The gravity loading must be based on the estimated actual weight of the structure and not published values. Remaining is the roof uplift loads.

Roof uplift can be determined by working backwards in the analysis. Since the results for post uplift were produced by an endwall shear intensity of 2.92 kN/m (200 lb./ft.), and the roof shear load is equal to the endwall shear, the roof uplift that produces the corner post force can be calculated for the study building. To find the corresponding roof uplift load, the wind pressure that produced the shear on the endwall must be determined. To do this, the frame/roof interactions must be determined, requiring a diaphragm design procedure. The diaphragm design procedure used is in accordance with ASAE Engineering Practice EP484, concerning buildings without ceilings, and is covered in Appendix C. The roof uplift and the other vertical components of sidewall post uplift are determined in Appendix D.

4.4.1.1 Equilibrium Check on Sidewall Post

The required foundation resistance, F_R , for uplift for each post is calculated by summing all of the vertical forces acting on the sidewall posts.

By equilibrium (upward forces positive),

$$\Sigma F_v = -F_R + \text{sidewall post uplift} + \text{roof uplift} - \text{roof dead load} - \text{sidewall dead load}$$

$$F_R = 1,047 + 3,080 - 896 - 511 = 2,720 \text{ lb. per sidewall post.}$$

4.4.1.2 Post-to-Collar Resistance for Uplift Design of Sidewall Posts

The capacity of the connection of the post to a wooden spread footing is the next consideration. Figure 24 illustrates a wood collar fastened to the foot of a post to act as a spread footing. A 2 x 6 treated wood collar is selected, 8-20d stainless steel (or equivalent) nails per side. The use of 1.33 for duration of load factor is appropriate (NFPA, para. 2.2.5.3, 1986). The joint capacity is

$$8 \text{ nails per side} \times 2 \text{ sides} \times 1.33 \times 139 \text{ lb. per nail} = 2,960 \text{ lb.}$$

which meets the requirement of 2,720 lb.

In addition to the duration of load factor, 1.33, another factor can be used. Most building codes permit a 33 percent increase of working stresses when considering wind or earthquake forces (UBC 1114.3, p 275). With the addition of this factor of 1.33, 6-20d nails can be used instead of eight, on each side of the wood collar to provide a resistance of 2,950 lb. for the nailed wood collar to-post-connection, which is greater than the 2,720 lb. average uplift on each sidewall post.

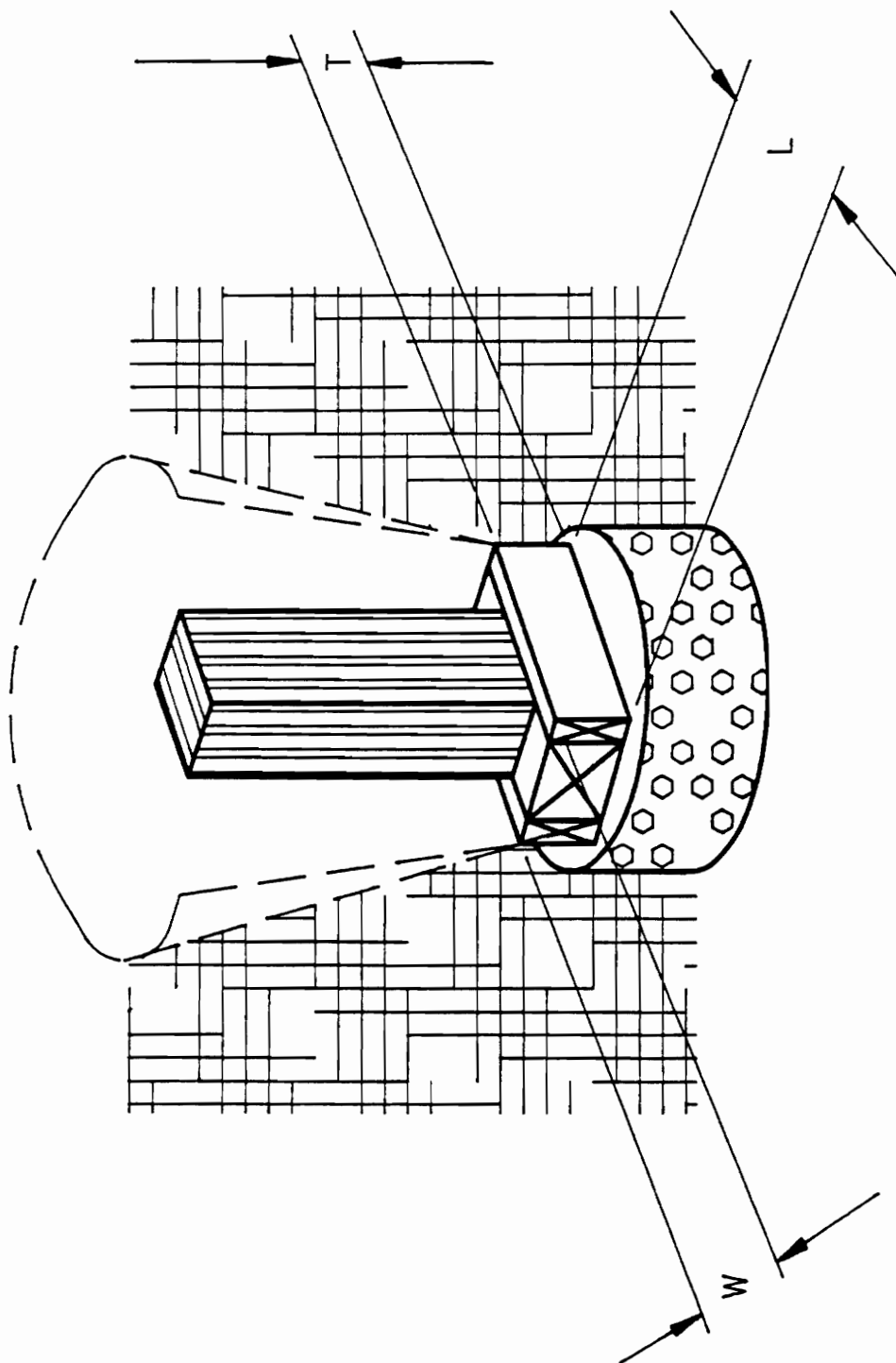


Figure 24. Wooden collar attached to foundation of the post acts as a spread footing.: (ASAE EPX486)

4.4.1.3 Soil Resistance to Uplift

Next, consider the uplift capacity of the spread footing. The wood collar is assumed to displace a truncated prismatic volume of soil above the collar surface (Figure 24), and the equation of this potential volume is given by Equation 4.1, taken from section 5.3.2.2 of ASAE EP X486. In the absence of available soil test data, Table 1 values in EP X486 may be presumed. Using the most conservative soil data in Table 1, which is a loose sandy gravel, along with a 2 x 6 wooden collar 16 in. long, and a post embedment depth of 4 ft. EP X486 5.3.2.2, values from Table 1, category No. 3 are for this type of soil.

$$U = \alpha G[(wl - A_p)(d - t) + (w + l)(d - t)^2 \tan \theta + 0.33\pi(d - t)^3 \tan^2 \theta] \quad [4.1]$$

where

U = Soil Resistance, lb.

α = 90 lb./ft. ³

G = 1 lbf./lb.

w = 5.5 + 1.5 + 1.5 in. = 8.5 in. = 0.708 ft.

l = 16 in. = 1.333 ft.

A_p = 5.5 in. x 5.5 in. = 0.21007 ft²

d = 4 ft.

t = 5.5 in. = 0.458 ft.

θ = 32°

Solving, U = 3,290 lb.

Foundation resistance of 3,290 lb. exceeds net sidewall post uplift of 2,720 lb. The foundation soil resistance is sufficient for the design case considered.

4.4.1.4 Shear Transfer from Post to Wall

A factor that must be considered is whether the diaphragm panels on the sidewall and endwall, adjacent to the corner posts, have the capacity to carry shear loads imposed by the corner posts. Recall that the load transferred from the roof has been predetermined. When the sidewall and endwall are resisting vertical corner post uplift, the rib orientation is parallel to the loading; matching the roof rib orientation to load. If the roof has been designed to carry the given wind loading, and the wall construction is similar to the roof, then the sidewall has at least the same shear capacity, as argued in Section 2.5. To illustrate this point, consider the shear loading of 2.92 kN/m (200 lb./ft.) used to generate the values in Table 5. Since all endwall posts adjacent to the corner post are in compression (Figure 23), the shear loading may only be distributed to the sidewall posts. In other words, the capacity for shear transfer from the corner post is 2.92 kN/m (200 lb./ft.) x 6.50 m or 18.98 kN (4,260 lb.). When considering the sidewall as a rigid body, it is necessary to distribute all of the corner post uplift force equally throughout the tributary posts. However, the sidewall allowable diaphragm capacity is 18.98 kN (4,260 lb.). Hence, 34.9 kN (7.85 kips) minus 18.98 kN (4,260 lb.) or 15.92 kN (3,590 lb.) of the corner post uplift force must be resisted by the corner post foundation connection.

4.4.1.5 Additional Requirements for Corner Post

Although the sidewall posts are adequate in design, the corner posts will need additional resistance to uplift. To design the corner posts for uplift:

1. Select a post embedment depth and collar length that will provide a soil resistance of at least 15.92 kN (3,590 lb.).

2. Design the post-to-collar connection so that it will provide at least 15.92 kN (3,590 lb.) of resistance.

4.5 Door Post Uplift

While studying the results of uplift, it was realized that the high corner post uplifts were unrealistic. The sidewall resistance was neglected since the endwall was studied using plane-frame analysis. If for some reason the sidewalls were cut away from the corners, yet the roof remained attached, then high values would probably be accurate. However, from the results depicted in Figure 23, the door post in tension also has high uplift, lacking only about 10 percent from being the same magnitude as the corner post uplift. Figure 22, taken from ASAE EP484, suggests that the door post foundation connection may be critical. The door post uplift loads can not be distributed to adjacent posts, because the remaining posts on that side of the door are in compression. To compensate for this high uplift, the door posts will require a reinforced foundation, using the same options that can be used to enhance corner post foundation resistance discussed in Section 4.4.1.5.

4.6 Other Options Available

The bracing results indicate that reinforcing endwalls with steel strapping is not practical when approximately 50 percent or more of the endwall area is devoted to a door opening. When the criteria of three straps at each bracing location is exceeded, other options may be considered:

1. Interior endwall diaphragms.

Building applications sometimes require interior cladding, especially when the inside environment is harsh. This option is suitable when no more than one-half of the endwall is open, allowing the endwall strength to be restored to at least full capacity by full interior cladding.

If 4 ft. by 8 ft. gypsum wallboard is applied to the interior of the endwall, horizontally or vertically, on 2 by 4 wood studs spaced 24 in. on center, the ICBO Evaluation Service, Inc. provides shear values for the assembly. Table 6 gives shear values for 1/2 in. and 5/8 in. thick wallboard which might be used in conjunction with the steel diaphragm shear capacity.

2. Stronger construction.

Documented methods of endwall construction producing stronger diaphragms may be suitable. Stronger construction in conjunction with interior endwall diaphragms may be necessary when door area exceeds one-half of the total endwall area.

3. Door in sidewall.

If truss reactions above a door placed in the sidewall do not require excessive header capacity, then it may be more feasible to place the door in the endwall.

Table 6. Shear values for 1/2 in. and 5/8 in. thick gypsum wallboard when used for vertical shearwalls.

Thickness of Material	Wall Construction	Nail Spacing Maximum ²	Shear Value ¹	Minimum Nail Size
1/2 in.	Unblocked	7 in.	75 psf	5d Common
		4 in.	110 psf	5d Common
5/8 in.	Unblocked	4 in.	145 psf	6d Common

¹ Values are for short time loading due to wind or earthquake and must be reduced 25 percent for normal loading.
² Applies to nailing all girts. The minimum edge distance shall be 3/8 in.

5. Conclusions

A structural analog was developed using actual endwall test data for calibration. This analog may be used in an analysis procedure to aid in designing post-frame endwalls having large centered doors. The following conclusions were reached:

1. Steel strap bracing is a practical means of bracing endwalls having large, single doors, centered in the endwall when no more than one-half of the total endwall area is open. Steel-strap bracing may not develop the full desired loading. A comparison of the bracing results in this study to common industry practice confirms the need for a structural analysis in the design of endwalls having large doors. The basis for the required bracing is that the deflection of the endwall with a door opening is restored to that of the original endwall having no doors, and with reinforcement, the strength and stiffness is also restored to that of the original endwall.
2. Steel bracing straps in endwalls may not develop full design load. Additional nails allow greater strap load, but there is a point of diminishing return in that as more nails are added, the strap load only slightly increases. Also, multiple strapping per diaphragm panel does not result in the same increase in load capacity of that of single strapping (two straps in each panel, for the same load, yield less than twice the resistance of a single strap).

3. Corner post uplift forces are large, based on a simplistic plane-frame analysis of the endwall that is assumed not to be attached to the sidewall. It is recommended for design purposes that uplift forces be apportioned to wall posts adjacent to the corner post.
4. The framing and truss may be ignored in the analysis of endwalls because of the relatively small contribution made by the truss and framing to total endwall stiffness. The truss cladding acts as a shear collector for the endwall diaphragm.
5. Provided actual endwall stiffness data are available, the building designer can develop a computer structural analog, tailoring it to specific endwall-door applications. The restriction is that, in order to use the calibrated strap data developed in this study, the building endwall to be analyzed must have a diaphragm construction that has stiffness comparable to McFadden panels.
6. If the McFadden panel configuration is used for a computer structural analog, it is necessary to only analyze the 3.25 m (10.67 ft.) height, regardless of the actual height. From the analysis, the uplift at the corner post increased linearly with building height. As for the strapping scheme, building heights above 3.25 m (10.67 ft.) require a second identical row of strapping.

6. Summary and Recommendations

6.1 *Summary*

When diaphragm action is considered in the design of post-frame buildings, the endwalls may require supplemental reinforcement to replace lost strength and stiffness, due to placement of large doors and openings in the endwalls. The diaphragm design standard, ASAE EP484, does not provide guidelines concerning such endwall reinforcement. Flat steel strapping is proposed for use as supplemental reinforcement to replace this lost strength and stiffness.

Research reveals the fact that the mechanical behavior of diaphragms is complex and predictive models are not available. Current work is underway at Pennsylvania State to produce a finite element model that can accurately predict diaphragm behavior. However, a practical design solution is needed now for the post-frame industry.

The target of this research was to "bring back" the deflection of an endwall with a door to that of the same solid endwall. Then, based on the results of Gebremedhin and Irish (1986), the endwall

is at least as strong and stiff as the roof, if they are of the same configuration. It is assumed that the roof has been designed to meet the requirements of ASAE EP484.

McFadden et al. (1988) laterally loaded a full-scale post-frame building to study the diaphragm action of the roof. Endwall data supplied in this study allowed for calibration of a computer structural analog using PPSA 3.00. First, the model was used to "mimic" the deflection of the McFadden endwall framing. Next, the model was used to "mimic" the McFadden endwall deflection with framing and cladding. Diagonal fictitious members were placed between each post to simulate diaphragm panels, and the truss cladding was omitted. The wall girts were omitted from analysis without a significant change in the overall deflection.

Using the same calibrated diaphragm fictitious members and framing, a general model 12.19 m (40 ft.) wide was developed. A total roof shear loading of 2.92 kN/m (200 lb./ft.) was divided equally at the top nodes of each post. The truss was omitted without significant change in overall deflection.

The endwall height was doubled to simulate the maximum height to be studied at 6.50 m (21.3 ft.). The deflection of the solid endwall, modeled for widths of 9.75 m (32 ft.) to 24.4 m (80 ft.) in 2.44 m (8 ft.) increments, provided the target deflections.

Center sections, full height, were removed to simulate 4.88 m (16 ft.) or 7.32 m (24 ft.) wide door openings.

Fictitious members representing either 3.18 cm (1.25 in.) or 5.08 cm (2 in.) wide flat steel strapping was placed over the diaphragm fictitious members. Included in the steel strap fictitious members was a nail stiffness fictitious member to account for the stiffness of the nail connections at the ends of each strap. The nail connection stiffness used in this study was equivalent to twelve times the stiffness of a 16d nail, on the end of each strap.

To determine the required strap bracing for each endwall case studied, the stiffness of the strap-nail fictitious members was increased until the target deflection of the solid endwall case was met. In other words, if the required stiffness was three times that of one strap, then three straps between each post, in each direction, are required. Each strap likewise required a nail connection stiffness equivalent to twelve times the stiffness of a 16d nail on each end.

There is evidence from this study that high corner post uplift forces are significantly resisted by the sidewall, and sometimes endwall, diaphragms. There is also evidence of high door post uplift forces. High post uplift forces may be considered in design by deeper post embedment, and higher resistance of post-to-collar connection to uplift.

When steel strap bracing is not practical or desired, the designer might consider using interior metal cladding on the endwall, stronger endwall construction, or placing the door in the sidewall.

6.1.1 Assumptions Highlighted

Many assumptions were made throughout this study that profoundly affect the limitations of the conclusions. A digest of these assumptions follows.

1. If the deflection of an endwall with a large door can be "brought back" to that of the same endwall being solid, then likewise the strength and stiffness are "brought back".
2. The cladding and fasteners in an endwall with a large door are no more stressed than their counterparts in the same endwall that is solid.
3. Building construction for a solid endwall is adequate to meet serviceability requirements.
4. The roof has been properly designed to resist shear loading.
5. Roof shear is transferred to the endwall posts by the gable end truss; the truss cladding is neglected in analysis.

6. All endwall posts carry an equal share of roof shear.
7. Shear deflection is linear along the building height, as in plywood design.
8. All of the steel strap bracing, regardless of how far from the door, provides uniform bracing of the endwall diaphragm.
9. All nails in the end of each strap equally share strap load and contribution to connection stiffness.
10. In reference to corner post uplift, the sidewall and endwall diaphragms behave as rigid diaphragms.
11. The corner post uplift force is equally shared by the tributary posts.
12. The wood collar at the post footing displaces a truncated prismatic volume of soil above the collar surface.

6.2 Recommendations for Future Research

The resistance supplied by each steel strap in the endwall depends on the nail connection stiffness of each strap. It appears that nail connection stiffness is a limiting factor in potential strap resistance. The nail connection stiffness for a multiple nail connection in the end of a steel strap cannot be predicted without actual testing. It is recommended that actual strap-to-post connections be tested to determine how each nail contributes to the overall nail connection stiffness.

Model validation remains open ended in this study. The computer structural analogs used in this study were only intended to provide an analysis procedure as input to endwall diaphragm design. Full-scale endwall testing was cost prohibitive. Coincidentally, Kifle Gebremedhin ² is currently

² Dr. Gebremedhin is Professor of Agricultural Engineering, Cornell University, Ithaca, NY.

conducting full-scale endwall tests that include large openings. With the addition of steel strap bracing, validation of this analysis procedure is conceivable.

6.3 Recommended Design Procedure for Endwalls

This recommended design procedure represents a "first generation" effort for designing endwalls with doors. The following steps are recommended for estimating the required bracing using flat steel strapping.

6.3.1 Construction with Endwall Stiffness Comparable to McFadden's Data

1. When endwall construction is comparable to McFadden's data, such as 0.368 mm (29 ga.) steel cladding with no stitching, the results of Table 4 provide the required bracing using 3.18 cm (1.25 in.) or 5.08 cm (2 in.) flat steel strapping.
2. If the endwall ceiling height is between 3.25 m (10.67 ft.) and 6.50 m (21.3 ft.), two identical rows of strapping are required, and should be placed in the same manner as depicted in Figure 21.
3. Design for corner post uplift. The corner post uplift force is determined by the proportion of the actual height to 3.25 m (10.67 ft.) and the proportion of actual roof shear to 2.92 kN/m (200 lb./ft.), with respect to the corner post uplift results in Table 5.
4. By assuming that the sidewall diaphragm (and in some cases the endwall diaphragm) acts as a rigid body, the corner post uplift force may be equally distributed among the sidewall tributary posts (and any endwall posts between the corner and the door that are in tension). Determine the

net uplift on these posts (Appendix D) and design for the required foundation resistance, F_R , in accordance with ASAE EPX486. Two steps are involved in designing for sidewall post uplift resistance:

4.1 Select a post embedment depth and collar length that will provide the necessary soil resistance to uplift.

4.2 Select a post-to-collar connection that will provide the necessary resistance to uplift.

5. The required corner post uplift resistance is equal to the uplift force from the endwall structural analysis minus the allowable shear capacity of the sidewall-corner post connection.

6. Door post uplift design follows the same procedure as the corner post uplift design. The exception is that the tributary posts are only the adjacent endwall posts, between that door post and the corner, that are in tension.

A design procedure for endwalls having a stiffness significantly different from the McFadden endwall construction is given in Appendix B for consideration by the building designer.

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Appendix A. Glossary of Terms

The following terminology is taken from Section 2 of ASAE EP484.

A.1 Diaphragm: A structural assembly - including the timber framing (truss chords and purlins), metal cladding, fasteners and fastening patterns - capable of transferring in-plane shear forces through the cladding and framing members.

A.2 Diaphragm design: The design of a post frame, including the roof truss, sidewall posts, endwalls, shear connectors, chord splices and ground anchorages, in which the diaphragm strength and diaphragm stiffness are utilized to transfer applied horizontal loads to the ground.

A.3 Endwall diaphragm: The endwall of the building where the endwall cladding and framing are constructed so as to transfer in-plane shear forces from the roof and/or ceiling diaphragm to the ground.

A.4 Metal cladding: The metal exterior and interior coverings, usually cold-formed aluminum or steel sheet, fastened to the timber framing.

A.5 Post frame: A structural frame consisting of a wood truss connected to vertical timber columns.

A.6 Shear transfer: The transfer of the resultant forces between individual sheets of cladding, between the edges of roof diaphragms and the top of the endwalls, between the edges of ceiling diaphragms and the endwalls, or between the bottom of the endwall diaphragms and the ground.

A.7 Shear wall: A vertical diaphragm in a structural framing system. The wall may be an endwall or an intermediate wall. In either case, the wall transfers shear forces from the roof or ceiling diaphragms to the groundline.

A.8 Test diaphragm: A diaphragm model of sufficient size to simulate the behavior of the diaphragm in the building. Except for overall size, the test diaphragm construction is functionally equivalent to the building diaphragm and is supported in a manner similar to that encountered in the building application.

Appendix B. Construction with Endwall Stiffness Differing with McFadden's Data

When endwall construction is more or less stiff than McFadden's data, it is recommended, for design purposes, that McFadden's data be adjusted proportionately for calibration and design of an endwall. For example, it may be known that stitching increases endwall stiffness by a known measure. This procedure should be evaluated as resources become available.

1. Develop a computer model of the endwall. Include the truss bottom chord and all endwall posts. All post to truss lower chord joints are rigid. Use a reaction analog that best describes the foundation.
2. Pin fictitious diaphragm members between each post, in the direction resisting the wind load, from the groundline to the truss bottom chord (Figure 12). If the angle from the horizontal exceeds 53° (McFadden's panel and endwall), two rows of diaphragm fictitious members should be used. This will require an intermediate horizontal stiffener (Figure 13), which is also pinned to the posts, has the elasticity of steel, and the same cross-section area as the posts.

3. Using the adjusted endwall stiffness data, apply a load at one eave node. It is suggested as an initial starting point, that for a 1 in. by 1 in. diaphragm fictitious member, the elasticity be 5 million psi. Record the deflection of the other eave. Adjust the elasticity of the diaphragm fictitious members until the load-deflection relationship at that eaves matches the actual stiffness data.
4. Divide the design roof shear load equally among the nodes along the truss bottom chord. Conduct an analysis and record the maximum nodal deflection. This is the deflection to which an endwall with a door opening must be restored.
5. Remove the sections where the door will be located. The cladding above the door, if present, is neglected; i.e. the door is considered to be full height (See Figures 14 and 15). Use 3-2x12's above the door to serve as a header and to act as a drag strut in the absence of the truss.
6. Add steel strap fictitious members to the model. The members are placed "over top" of the diaphragm fictitious members. Use the elasticity of steel and the width and thickness of the strapping, based on the manufacturer's specification. Add an additional fictitious member on one end to account for the total nail connection stiffness. For a stiffness equivalent to twelve times a single 16d threaded, hardened nail, on each end of each strap, use a 2 in. long by 3.09 in. square member, having an elasticity of 12,000 psi. The entire member is pinned at each end and the joint between the strap and nail fictitious members is rigid.
7. To model using two or three straps per bracing location, double or triple the strap thickness. For the nail connection stiffness of a two strap member, use a 2 in. long by 4.38 in. square member having an elasticity of 12,000 psi. For the nail connection stiffness of a three strap member, use a 2 in. long by 5.36 in. square member also having an elasticity of 12,000 psi.
8. Begin with one strap in each section. Conduct an analysis and record the maximum deflection along the top of the model. If this deflection is less than the solid endwall deflection, then this strapping scheme is adequate for endwall bracing. If not, continue adding strap bracing (with cor-

responding nail connection stiffness) until the deflection is at least restored to that of the solid endwall. If three straps still do not restore the endwall deflection to that of the solid endwall, it may be concluded that steel strapping is not a practical means of bracing endwalls with large doors. If this is the case, consider the options in Section 4.6.

9. Design for corner post uplift. If the McFadden panel configuration is used for analysis, the corner post uplift force is determined by the proportion of actual height to 3.25 m (10.67 ft.) and the proportion of actual roof shear to 2.92 kN/m (200 lb./ft.), with respect to the corner post uplift of computer results.

10. By assuming that the sidewall diaphragm (and in some cases the endwall diaphragm) acts as a rigid body, the corner post uplift force may be equally distributed among the sidewall tributary posts (and any endwall posts between the corner and the door that are in tension). Determine the net uplift on these posts and design for foundation resistance to uplift in accordance with Appendix D and ASAE EPX486. Two steps are involved in designing for sidewall post uplift resistance:

10.1 Select a post embedment depth and collar length that will provide the necessary soil resistance to uplift.

10.2 Select a post-to-collar connection that will provide the necessary resistance to uplift.

11. The required corner post uplift resistance is equal to the uplift force from the computer analysis minus the allowable shear capacity of the sidewall.

12. Door post uplift design follows the same procedure as the corner post uplift design. The exception is that the tributary posts are only the adjacent endwall posts, between that door post and the corner, that are in tension.

Appendix C. Diaphragm Test

The roof diaphragm stiffness for this example is based on actual data of typical diaphragm construction. A test panel 16 ft. wide and 22 ft. deep was tested using the simple beam diaphragm test (Woeste and Townsend, 1991). By nomenclature from Figure 3 of ASAE EP484:

$a = 8 \text{ ft.}$

$b = 22 \text{ ft.}$

Panel stiffness is then

$$c = \frac{1}{2} \frac{P}{D_s} \frac{a}{b} \quad [\text{C.1}]$$

$P = 0.4P_{ult}$

$P = 0.4 \times 10,600. \text{ lb.} = 4,240 \text{ lb.}$

D_s at $0.4P_{ult}$ was 0.594 inches.

Thus

$$c = \frac{1}{2} \times \frac{4,240.\text{lb.}}{0.594\text{in.}} \times \frac{8\text{ft.}}{22\text{ft.}} = 1,298. \frac{\text{lb.}}{\text{in.}} \quad [\text{C.2}]$$

Hence, 1,298 lb./in. is the stiffness for a 22 ft. panel. Next solve for K_2 using Luttrell's equation and the test data results

$$c' = \frac{Et}{2(1 + \nu)g/p + K_2/(b't)^2} \quad [C.3]$$

where for the material tested

$$E = 29 \times 10^6 \text{ psi}$$

$$t = 0.016 \text{ in.}$$

$$\nu = 0.3$$

$$g/p = 1.16$$

$$b' = 22 \text{ ft.} \times 12 \text{ in./ft.} = 264 \text{ in.}$$

$$1,298. = \frac{29 \times 10^6 \times 0.016}{2(1 + 0.3)1.16 + K_2/(264 \times 0.016)^2} \quad [C.4]$$

Solving, $K_2 = 6,324$.

Consider a post frame building 112 ft. by 56 ft. having 21.3 ft. ceiling height, 3:12 roof slope, and 8 ft. post spacing (Figure 18). For a 56 ft. span the roof slope length is

$$b' = \frac{56 \text{ ft.}}{2} \div \cos 14.04 = 28.9 \text{ ft.} \quad [C.5]$$

$$b' = 347 \text{ in.}$$

Now applying Luttrell's equation for the roof slope

$$c' = \frac{Et}{2(1 + \nu)g/p + K_2/(b't)^2} \quad [C.6]$$

$$c' = \frac{29 \times 10^6 \times 0.016}{2(1. + 0.3)1.16 + 6324./(347 \times 0.016)^2} \quad [C.7]$$

$$c' = 2,230 \text{ lb./in.}$$

For a sloped roof

$$c_h = c' \cos^2 \theta (b'/a) \quad [C.8]$$

where b'/a is the aspect ratio of the roof diaphragm. For this example $b'/a = 28.9 \text{ ft.} / 8 \text{ ft.} = 3.61$.

Then

$$c_h = 2,230 \cdot \cos^2 14.04^\circ \times 3.61 \quad [C.9]$$

$$c_h = 7,580 \text{ lb./in.}$$

Since there are two roof slopes, not attached at the roof ridge

$$c_h = 2 \times 7,580 \text{ lb./in.} = 15,160 \text{ lb./in.}$$

The horizontal frame stiffness, k , is defined as the ratio of the applied force to the respective deflection at the eave node (Figure 6, ASAE EP484). Using computer analysis, the frame stiffness was calculated to be $k = 100./3.15 = 31.7 \text{ lb./in.}$ The ratio of frame stiffness to roof stiffness is

$$k/c_h = 31.7/15,160 = 0.0021.$$

This ratio is very low and is off the force modifier charts in the diaphragm design standard. The roof stiffness is much greater than the frame stiffness, therefore nearly all of the load is carried by the roof diaphragm. This outcome leads to a more significant method of analysis to follow in Appendix D.

Appendix D. Alternative Design Approach

When the posts are supported at the floor level in such a way that rotation is prevented at the base, and the roof diaphragm is very stiff compared to the frame stiffness, the posts behave as a propped cantilevers. The maximum roof shear intensity, I , at the eave line can then be calculated as the reaction at the pinned end of a propped cantilever plus the net horizontal force of the roof wind loading according to Bender et al. (1990) as

$$I = \frac{\left\{ \frac{3}{8} (q_{ww}H_1L - q_{lw}H_1L) + q_{wr}H_2L - q_{lr}H_2L \right\}}{2w} \quad [D.1]$$

where

I = The maximum roof shear intensity, lb./ft.

q_{ww} = Design windward wall pressure, psi

q_{lw} = Design leeward wall pressure, psi

q_{wr} = Design windward roof pressure, psi

q_{lr} = Design leeward roof pressure, psi

H_1 = Sidewall height, ft.

H_2 = Peak roof height from eave, ft.

L = Building length, ft.

w = Building width, ft.

Hoyle and Woeste (1989) defines the pressure coefficients for both the leeward and windward roofs of a 3:12 roof slope to be 0.7 (suction), thus cancelling the horizontal components. With this setup, the loading reduces to that of the wind pressures exerted along the propped cantilever; that is, the roof shear loading is from the wind on the walls only.

And so, q_{wr} and q_{lr} cancel, yielding

$$I = \frac{3}{8} \frac{(q_{ww}H_1L - q_{lw}H_1L)}{2w} \quad [D.2]$$

or

$$I = \frac{3}{16} \frac{H_1L}{w} (q_{ww} - q_{lw}) \quad [D.3]$$

The design wind pressure is defined as

$$q = P_b C_p I_f$$

$$I = \frac{3}{16} \frac{H_1L}{w} (P_{bw}C_{pw}I_f - P_{blw}C_{plw}I_f) \quad [D.4]$$

I_f is an importance factor, not to be confused with the maximum roof shear intensity.

For:

$$I = 200 \text{ lb./ft.}$$

$$L = 112 \text{ ft.}$$

$$w = 56 \text{ ft.}$$

$$C_{pw} = 0.8$$

$$C_{plw} = -0.5$$

$$I_f = 1.0$$

$$H_1 = 21.3 \text{ ft}$$

$$P_{bw70} = 14.28 \text{psf}$$

$$P_{blw70} = 14.77 \text{psf}$$

The pressures may be expressed as proportions.

$$P_{blw} = 1.034 P_{bw}$$

$$P_{bw} = 0.967 P_{blw}$$

Substituting

$$200 = \frac{3}{16} (21.3) \left(\frac{112}{56} \right) (0.967 P_{blw} \times 0.8 \times 1.0 - P_{blw} \times 0.5 \times 1.0) \quad [D.5]$$

$$P_{bw} = 19.01 \text{ psf}$$

$$P_{blw} = 19.66 \text{ psf}$$

For the effective velocity pressure (P_b)

$$P_b = \frac{x^2}{70^2} P_{b70} \quad [D.6]$$

$$19.66 = \frac{x^2}{70^2} 14.77$$

$$x = 80.7 \text{ mph}$$

Thus the effective velocity pressure (P_b) for the study building and loading theory is 81 mph basic wind speed.

Determine roof pressure

$$q_{wr} = q_{lr} = P_{br} C_{pr} I_f = 19.66 \times 0.7 \times 1.0 \quad [D.7]$$

$$q_{roof} = 13.76 \text{psf}$$

Note that 13.76 psf is a tensor, and not a vector, so vertical uplift is also 13.76 psf. Acting on an 8 ft. tributary area between the posts, $13.76 \text{ psf} \times 28 \text{ ft.} \times 8 \text{ ft.} = 3,080 \text{ lb.}$

Roof design dead loads are 1 psf for the lower chord dead load and 4 psf for the top chord, or 5 psf total. Five psf is a typical design dead load for a storage building having no ceiling. Actual dead loads can be significantly lower than design dead loads. When taking dead loads into consideration in uplift calculations, the actual dead load should be used. Given that 5 psf is the design dead load, 4 psf was considered to be an estimate of the actual dead load

$$4 \text{ psf} \times 28 \text{ ft.} \times 8 \text{ ft.} = 896 \text{ lb. per post (down)}$$

Roof dead load is 896 lb. per post downward.

Sidewall dead load of 3 psf was calculated as the actual dead load of an 8 ft. wall section having 6 x 6 Southern Pine posts spaced 8 ft., 2 x 4 wall girts spaced 24 in., and 29 gage steel cladding:

$$3 \text{ psf} \times 21.3 \text{ ft.} \times 8 \text{ ft.} = 511 \text{ lb. per post (down).}$$

With the assumptions that the sidewall and endwall diaphragms act as rigid bodies and that the corner post uplift is equally distributed to the tributary sidewall posts, the sidewall post uplift is determined using the results of PPSA. For a 56 ft. wide endwall having a 24 ft. door, that is 112 ft. long and 21.3 ft. tall, having 200 lb./ft. shear, the corner post uplift is 7,850 lb. Figure 23 shows that this load may not be distributed to any interior endwall posts because they are in compression. Considering the sidewall posts nearest the corner post, on the windward side, it is assumed that the corner post uplift load is equally distributed among them. There are 15 sidewall posts. Taking half of these as the tributary area, the sidewall post uplift is

$$\frac{7,850}{7.5} = 1,047$$

Hence, 1,047 lb. per sidewall post.

Appendix E. PPSA Analysis of 48 ft. Solid Endwall

```
*****
***
***      PURDUE PLANE STRUCTURES ANALYZER 3.00 ***
***
***      COPYRIGHT (1987) ***
***      PURDUE RESEARCH FOUNDATION ***
***
***      THIS PROGRAM IS INTENDED TO FACILITATE ***
***      THE ANALYSIS OF WOOD STRUCTURES. IT ***
***      GIVES AN ACCURATE REPORT OF STRUCTURAL ***
***      RESPONSE OF THE INPUT ANALOG. ***
***      ACCURACY OF THE ANALOG AND INTERPRETA- ***
***      TION OF STRUCTURAL ADEQUACY ARE THE ***
***      RESPONSIBILITY OF THE USER. ***
***
*****
48 ft. solid endwall.
```

NUMBER OF NODES	= 28
NUMBER OF MEMBERS	= 45
NUMBER OF ROLLER SUPPORTS	= 7
NUMBER OF PINNED SUPPORTS	= 7
NUMBER OF FIRL SUPPORTS	= 0

NUMBER OF FIXED SUPPORTS = 0

NUMBER OF LOADING ARRANGEMENTS = 1

TABLE I ALLOWABLE MEMBER STRESSES IN PSI
NORMAL LOAD DURATION

MEMBER GROUP	USE TYPE	BEND	ALLOWABLE COMP	TENS	WIDTH	DEPTH	MODULUS OF ELASTICITY
1	U	750.	700.	475.	5.500	5.500	0.140E+07
2	UI	750.	700.	475.	5.500	5.500	0.140E+07
3	SI	725.	675.	375.	3.000	11.250	0.140E+07
4	SI	725.	675.	375.	1.500	5.500	0.140E+07
5	FI	9000.	0.	0.	0.040	1.250	0.290E+08
6	FI	9000.	0.	0.	0.060	0.060	0.290E+08
7	FI	9000.	0.	0.	1.000	1.000	0.544E+07
8	FI	9000.	0.	0.	5.500	5.500	0.290E+08

TABLE II NODE COORDINATES

NODE NO.	X-COORD (IN)	Y-COORD (IN)
1	0.000	0.000
2	0.000	36.000
3	0.000	164.000
4	0.000	292.000
5	96.000	0.000
6	96.000	36.000
7	96.000	164.000
8	96.000	292.000
9	192.000	0.000
10	192.000	36.000
11	192.000	164.000
12	192.000	292.000
13	288.000	0.000
14	288.000	36.000
15	288.000	164.000
16	288.000	292.000
17	384.000	0.000
18	384.000	36.000
19	384.000	164.000
20	384.000	292.000
21	480.000	0.000

22	480.000	36.000
23	480.000	164.000
24	480.000	292.000
25	576.000	0.000
26	576.000	36.000
27	576.000	164.000
28	576.000	292.000

TABLE III MEMBER LAYOUT

MEMBER NUMBER	NEGATIVE END		POSITIVE END		MEMBER GROUP
	NODE	CONDITION	NODE	CONDITION	
1	1	PNND	2	RIGD	1
2	2	RIGD	3	RIGD	1
3	3	RIGD	4	RIGD	1
4	2	PNND	7	PNND	7
5	3	PNND	7	PNND	8
6	3	PNND	8	PNND	7
7	4	RIGD	8	RIGD	4
8	5	PNND	6	RIGD	2
9	6	RIGD	7	RIGD	2
10	7	RIGD	8	RIGD	2
11	6	PNND	11	PNND	7
12	7	PNND	11	PNND	8
13	7	PNND	12	PNND	7
14	8	RIGD	12	RIGD	4
15	9	PNND	10	RIGD	2
16	10	RIGD	11	RIGD	2
17	11	RIGD	12	RIGD	2
18	10	PNND	15	PNND	7
19	11	PNND	15	PNND	8
20	11	PNND	16	PNND	7
21	12	RIGD	16	RIGD	4
22	13	PNND	14	RIGD	2
23	14	RIGD	15	RIGD	2
24	15	RIGD	16	RIGD	2
25	14	PNND	19	PNND	7
26	15	PNND	19	PNND	8
27	15	PNND	20	PNND	7
28	16	RIGD	20	RIGD	4
29	17	PNND	18	RIGD	2
30	18	RIGD	19	RIGD	2
31	19	RIGD	20	RIGD	2
32	18	PNND	23	PNND	7
33	19	PNND	23	PNND	8
34	19	PNND	24	PNND	7
35	20	RIGD	24	RIGD	4

36	21	PNND	22	RIGD	2
37	22	RIGD	23	RIGD	2
38	23	RIGD	24	RIGD	2
39	22	PNND	27	PNND	7
40	23	PNND	27	PNND	8
41	23	PNND	28	PNND	7
42	24	RIGD	28	RIGD	4
43	25	PNND	26	RIGD	2
44	26	RIGD	27	RIGD	2
45	27	RIGD	28	RIGD	2

TABLE IV REACTION CONDITIONS

NODE NUMBER	REACTION TYPE	HORIZ DISPL	VERT DISPL
1	pin	0.00	0.00
2	roll	0.00	1.00
5	pin	0.00	0.00
6	roll	0.00	1.00
9	pin	0.00	0.00
10	roll	0.00	1.00
13	pin	0.00	0.00
14	roll	0.00	1.00
17	pin	0.00	0.00
18	roll	0.00	1.00
21	pin	0.00	0.00
22	roll	0.00	1.00
25	pin	0.00	0.00
26	roll	0.00	1.00

TABLE V MEMBER PROPERTIES

MEMBER NUMBER	LENGTH (IN)	IN PLANE COLUMN (IN)	PRP PLANE COLUMN (IN)	BEAM LENGTH (IN)
1	36.000	0.000	0.000	75.180
2	128.000	0.000	0.000	235.520
3	128.000	0.000	0.000	235.520
4	160.000	0.000	0.000	0.000
5	96.000	0.000	0.000	0.000
6	160.000	0.000	0.000	0.000
7	96.000	0.000	0.000	0.000
8	36.000	0.000	0.000	75.180
9	128.000	0.000	0.000	235.520

10	128.000	0.000	0.000	235.520
11	160.000	0.000	0.000	0.000
12	96.000	0.000	0.000	0.000
13	160.000	0.000	0.000	0.000
14	96.000	0.000	0.000	0.000
15	36.000	0.000	0.000	75.180
16	128.000	0.000	0.000	235.520
17	128.000	0.000	0.000	235.520
18	160.000	0.000	0.000	0.000
19	96.000	0.000	0.000	0.000
20	160.000	0.000	0.000	0.000
21	96.000	0.000	0.000	0.000
22	36.000	0.000	0.000	75.180
23	128.000	0.000	0.000	235.520
24	128.000	0.000	0.000	235.520
25	160.000	0.000	0.000	0.000
26	96.000	0.000	0.000	0.000
27	160.000	0.000	0.000	0.000
28	96.000	0.000	0.000	0.000
29	36.000	0.000	0.000	75.180
30	128.000	0.000	0.000	235.520
31	128.000	0.000	0.000	235.520
32	160.000	0.000	0.000	0.000
33	96.000	0.000	0.000	0.000
34	160.000	0.000	0.000	0.000
35	96.000	0.000	0.000	0.000
36	36.000	0.000	0.000	75.180
37	128.000	0.000	0.000	235.520
38	128.000	0.000	0.000	235.520
39	160.000	0.000	0.000	0.000
40	96.000	0.000	0.000	0.000
41	160.000	0.000	0.000	0.000
42	96.000	0.000	0.000	0.000
43	36.000	0.000	0.000	75.180
44	128.000	0.000	0.000	235.520
45	128.000	0.000	0.000	235.520

LOADING 1

THE STRESS ADJUSTMENT FACTOR IS 1.33

TABLE VI C

THE STRUCTURE HAS NODE LOADS AS FOLLOWS

NODE NUMBER	DIRECTION OF LOAD	LOAD IN LBS OR IN-LBS AS APPROPRIATE
----------------	----------------------	---

4	1	1371.0
8	1	1371.0
12	1	1371.0
16	1	1371.0
20	1	1371.0
24	1	1371.0
28	1	1371.0

***** RESULTS *****

TABLE VII REACTIONS

REACTION NODE	HOR. COMP. (LBS)	VERT. COMP. (LBS)	MOMENT (IN-LBS)
1	57.800	-4225.794	0.000
2	-1647.422	-0.001	0.000
5	57.012	-204.000	0.000
6	-1681.674	-0.000	0.000
9	57.964	73.987	0.000
10	-1685.891	0.000	0.000
13	58.826	64.697	0.000
14	-1686.330	-0.000	0.001
17	59.359	59.559	0.000
18	-1676.617	-0.000	-0.000
21	59.610	327.854	0.000
22	-1549.723	-0.000	-0.000
25	61.271	3903.697	0.000
26	-81.332	-0.000	0.000

SUM OF LOADS	9597.000	0.000	-0.280E+07
SUM OF REACTS	-9597.148	-0.002	0.280E+07
DIFFERENCE	0.148	0.002	23.500

*** STRENGTH ANALYSIS ***

TABLE VIII MEMBER END ACTIONS

MEMBER NUMBER	LOCATION	AXIAL (LBS)	SHEAR (LBS)	MOMENT (IN-LBS)
1	NEG END	-4225.794	-57.800	0.000
	POS END	4225.794	57.800	-2080.816
2	NEG END	-2127.798	16.124	2080.816
	POS END	2127.798	-16.124	-16.959
3	NEG END	-18.588	7.941	16.959
	POS END	18.588	-7.941	999.448
4	NEG END	-2622.496	0.000	0.000
	POS END	2622.496	0.000	0.000
5	NEG END	1573.651	0.000	0.000
	POS END	-1573.651	0.000	0.000
6	NEG END	-2636.512	0.000	0.000
	POS END	2636.512	0.000	0.000
7	NEG END	1363.061	-18.588	-999.448
	POS END	-1363.061	18.588	-785.013
8	NEG END	-204.000	-57.012	0.000
	POS END	204.000	57.012	-2052.434
9	NEG END	1941.737	15.359	2052.435
	POS END	-1941.737	-15.359	-86.477
10	NEG END	2109.410	13.909	86.477
	POS END	-2109.410	-13.909	1693.879
11	NEG END	-2682.172	0.000	0.000
	POS END	2682.172	0.000	0.000
12	NEG END	1698.031	0.000	0.000
	POS END	-1698.031	0.000	0.000
13	NEG END	-2832.087	0.000	0.000
	POS END	2832.087	0.000	0.000
14	NEG END	1138.248	-18.387	-908.865
	POS END	-1138.248	18.387	-856.279
15	NEG END	73.987	-57.964	0.000
	POS END	-73.987	57.964	-2086.702
16	NEG END	2222.684	16.405	2086.702
	POS END	-2222.684	-16.405	13.131

17	NEG END	2267.072	13.028	-13.131
	POS END	-2267.072	-13.028	1680.688
18	NEG END	-2685.871	0.000	0.000
	POS END	2685.871	0.000	0.000
19	NEG END	1728.127	0.000	0.000
	POS END	-1728.127	0.000	0.000
20	NEG END	-2737.656	0.000	0.000
	POS END	2737.656	0.000	0.000
21	NEG END	796.968	-16.985	-824.409
	POS END	-796.968	16.985	-806.128
22	NEG END	64.697	-58.826	0.000
	POS END	-64.697	58.826	-2117.722
23	NEG END	2211.558	17.358	2117.723
	POS END	-2211.558	-17.358	104.132
24	NEG END	2190.953	11.581	-104.133
	POS END	-2190.953	-11.581	1586.447
25	NEG END	-2683.577	0.000	0.000
	POS END	2683.577	0.000	0.000
26	NEG END	1706.994	0.000	0.000
	POS END	-1706.994	0.000	0.000
27	NEG END	-2660.115	0.000	0.000
	POS END	2660.115	0.000	0.000
28	NEG END	513.789	-16.156	-780.318
	POS END	-513.789	16.156	-770.658
29	NEG END	59.559	-59.359	0.000
	POS END	-59.559	59.359	-2136.911
30	NEG END	2191.944	17.970	2136.911
	POS END	-2191.944	-17.970	163.202
31	NEG END	2128.713	10.585	-163.201
	POS END	-2128.713	-10.585	1518.062
32	NEG END	-2665.481	0.000	0.000
	POS END	2665.481	0.000	0.000
33	NEG END	1652.109	0.000	0.000

	POS	END	-1652.109	0.000	0.000
34	NEG	END	-2604.538	0.000	0.000
	POS	END	2604.538	0.000	0.000
35	NEG	END	278.131	-15.536	-747.403
	POS	END	-278.131	15.536	-744.018
36	NEG	END	327.854	-59.610	0.000
	POS	END	-327.854	59.610	-2145.946
37	NEG	END	2290.290	18.286	2145.945
	POS	END	-2290.290	-18.286	194.709
38	NEG	END	2082.594	10.004	-194.710
	POS	END	-2082.594	-10.004	1475.273
39	NEG	END	-2453.045	0.000	0.000
	POS	END	2453.045	0.000	0.000
40	NEG	END	1488.019	0.000	0.000
	POS	END	-1488.019	0.000	0.000
41	NEG	END	-2405.862	0.000	0.000
	POS	END	2405.862	0.000	0.000
42	NEG	END	76.400	-16.572	-731.253
	POS	END	-76.400	16.572	-859.649
43	NEG	END	3903.697	-61.271	0.000
	POS	END	-3903.697	61.271	-2205.747
44	NEG	END	3903.697	20.061	2205.747
	POS	END	-3903.697	-20.061	362.112
45	NEG	END	1941.261	3.887	-362.112
	POS	END	-1941.261	-3.887	859.650

TABLE IX

INTERACTION ANALYSIS

USING MAX MOMENT AND MAX FORCE

MEMBER NUMBER - TYPE	ADDNL INFO	MAX INT VAL	LOC FROM M (IN)	NEG END P (IN)	STRESS BENDING (PSI)	AXIAL (PSI)	L/D
1-U		0.296	36.00	36.00	-75.04	139.70	6.545

2-U		0.187	0.00	128.00	-75.04	70.34	23.273
3-U		0.037	128.00	128.00	36.04	0.61	23.273
7-SI		0.321	0.00	96.00	132.16	-165.22	9.958
8-UI		0.085	36.00	36.00	-74.02	6.74	5.236
9-UI	P	0.150	0.00	128.00	-74.02	-64.19	18.618
10-UI	P	0.143	128.00	128.00	61.09	-69.73	18.618
14-SI		0.278	0.00	96.00	120.18	-137.97	9.778
15-UI		0.078	36.00	36.00	-75.25	-2.45	6.545
16-UI	P	0.162	0.00	128.00	-75.25	-73.48	18.618
17-UI	P	0.149	128.00	128.00	60.61	-74.94	18.618
21-SI		0.221	0.00	96.00	109.01	-96.60	9.778
22-UI		0.079	36.00	36.00	-76.37	-2.14	6.545
23-UI	P	0.163	0.00	128.00	-76.37	-73.11	18.618
24-UI	P	0.143	128.00	128.00	57.21	-72.43	18.618
28-SI		0.176	0.00	96.00	103.18	-62.28	9.778
29-UI		0.079	36.00	36.00	-77.06	-1.97	6.545
30-UI	P	0.163	0.00	128.00	-77.06	-72.46	18.618
31-UI	P	0.138	128.00	128.00	54.75	-70.37	18.618
35-SI		0.140	0.00	96.00	98.83	-33.71	9.778
36-UI		0.089	36.00	36.00	-77.39	-10.84	6.545
37-UI	P	0.167	0.00	128.00	-77.39	-75.71	18.618
38-UI	P	0.134	128.00	128.00	53.20	-68.85	18.618
42-SI		0.128	96.00	96.00	-113.67	-9.26	9.958
43-UI		0.218	36.00	36.00	-79.55	-129.05	6.545
44-UI	P	0.232	0.00	128.00	-79.55	-129.05	18.618
45-UI	P	0.107	128.00	128.00	31.00	-64.17	18.618

P INTERACTION VALUE CRITICAL FOR PERPENDICULAR PLANE

TABLE IXA

MEMBER ANALYSIS DATA

MEMBER NUMBER	LENGTH (IN)	FINAL ADJUSTED STRESSES			
		FAXIAL (PSI)	FBEND (PSI)	FBEND PRIME (PSI)	IN PLANE L/D RATIO
1	36.000	631.75	997.50	997.50	6.545
2	128.000	631.75	997.50	997.50	23.273
3	128.000	631.75	997.50	997.50	23.273
7	96.000	-897.75	964.25	964.25	9.958
8	36.000	631.75	997.50	997.50	5.236
9	128.000	-849.66	997.50	997.50	14.822
10	128.000	-849.66	997.50	997.50	16.430
14	96.000	-897.75	964.25	964.25	9.778

15	36.000	-931.00	997.50	997.50	6.545
16	128.000	-849.66	997.50	997.50	14.822
17	128.000	-849.66	997.50	997.50	16.430
21	96.000	-897.75	964.25	964.25	9.778
22	36.000	-931.00	997.50	997.50	6.545
23	128.000	-849.66	997.50	997.50	14.822
24	128.000	-849.66	997.50	997.50	16.430
28	96.000	-897.75	964.25	964.25	9.778
29	36.000	-931.00	997.50	997.50	6.545
30	128.000	-849.66	997.50	997.50	14.822
31	128.000	-849.66	997.50	997.50	16.430
35	96.000	-897.75	964.25	964.25	9.778
36	36.000	-931.00	997.50	997.50	6.545
37	128.000	-849.66	997.50	997.50	14.822
38	128.000	-849.66	997.50	997.50	16.430
42	96.000	-897.75	964.25	964.25	9.958
43	36.000	-931.00	997.50	997.50	6.545
44	128.000	-849.66	997.50	997.50	14.822
45	128.000	-849.66	997.50	997.50	17.371

TABLE X SHEAR STRESS ANALYSIS

MEMBER	MAX. SHEAR STRESS (PSI)	LOC. FROM NEG. END (IN)	MEMBER LENGTH (IN)
1	-2.87	36.000	36.000
2	0.80	128.000	128.000
3	0.39	128.000	128.000
7	-3.38	96.000	96.000
8	-2.83	36.000	36.000
9	0.76	128.000	128.000
10	0.69	128.000	128.000
14	-3.34	96.000	96.000
15	-2.87	36.000	36.000
16	0.81	128.000	128.000
17	0.65	128.000	128.000
21	-3.09	96.000	96.000
22	-2.92	36.000	36.000
23	0.86	128.000	128.000
24	0.57	128.000	128.000
28	-2.94	96.000	96.000
29	-2.94	36.000	36.000
30	0.89	128.000	128.000
31	0.52	128.000	128.000
35	-2.82	96.000	96.000
36	-2.96	36.000	36.000
37	0.91	128.000	128.000

38	0.50	128.000	128.000
42	-3.01	96.000	96.000
43	-3.04	36.000	36.000
44	0.99	128.000	128.000
45	0.19	128.000	128.000

*** DEFLECTION ANALYSIS ***

TABLE XI MAXIMUM MEMBER DEFLECTIONS

MEMBER	MAX. DEFL. (IN)	LOC. FROM NEG. END (IN)	MEMBER LENGTH (IN)
1	0.002	21.000	36.000
2	-0.141	128.000	128.000
3	-0.311	128.000	128.000
7	-0.012	96.000	96.000
8	0.002	21.000	36.000
9	-0.141	128.000	128.000
10	-0.300	128.000	128.000
14	-0.018	24.000	96.000
15	0.002	21.000	36.000
16	-0.141	128.000	128.000
17	-0.290	128.000	128.000
21	-0.018	20.000	96.000
22	0.002	21.000	36.000
23	-0.141	128.000	128.000
24	-0.284	128.000	128.000
28	-0.017	20.000	96.000
29	0.002	21.000	36.000
30	-0.140	128.000	128.000
31	-0.279	128.000	128.000
35	-0.017	20.000	96.000
36	0.002	21.000	36.000
37	-0.140	128.000	128.000
38	-0.277	128.000	128.000
42	-0.021	96.000	96.000
43	0.002	21.000	36.000
44	-0.140	128.000	128.000
45	-0.277	128.000	128.000

TABLE XII NODE DISPLACEMENTS

DISPL. HORIZ. OR	DISPLACEMENT	DISPLACEMENT
------------------	--------------	--------------

NODE NUMBER	IN DIR. OF ROLLER (IN)	VERTICAL (IN)	ROTATIONAL (RADIAN)
1	0.000	0.000	0.000E+00
2	0.000	0.004	-0.260E-03
3	0.141	0.010	-0.152E-02
4	0.311	0.010	-0.929E-03
5	0.000	0.000	0.000E+00
6	0.000	0.000	-0.257E-03
7	0.141	-0.006	-0.154E-02
8	0.300	-0.012	-0.575E-03
9	0.000	0.000	0.000E+00
10	0.000	-0.000	-0.261E-03
11	0.141	-0.007	-0.150E-02
12	0.290	-0.014	-0.488E-03
13	0.000	0.000	0.000E+00
14	0.000	-0.000	-0.265E-03
15	0.141	-0.007	-0.147E-02
16	0.284	-0.013	-0.458E-03
17	0.000	0.000	0.000E+00
18	0.000	-0.000	-0.267E-03
19	0.140	-0.007	-0.145E-02
20	0.279	-0.013	-0.442E-03
21	0.000	0.000	0.000E+00
22	0.000	-0.000	-0.268E-03
23	0.140	-0.007	-0.144E-02
24	0.277	-0.013	-0.437E-03
25	0.000	0.000	0.000E+00
26	0.000	-0.003	-0.276E-03
27	0.140	-0.015	-0.138E-02
28	0.277	-0.021	-0.649E-03

* * * * PROGRAM STOP.

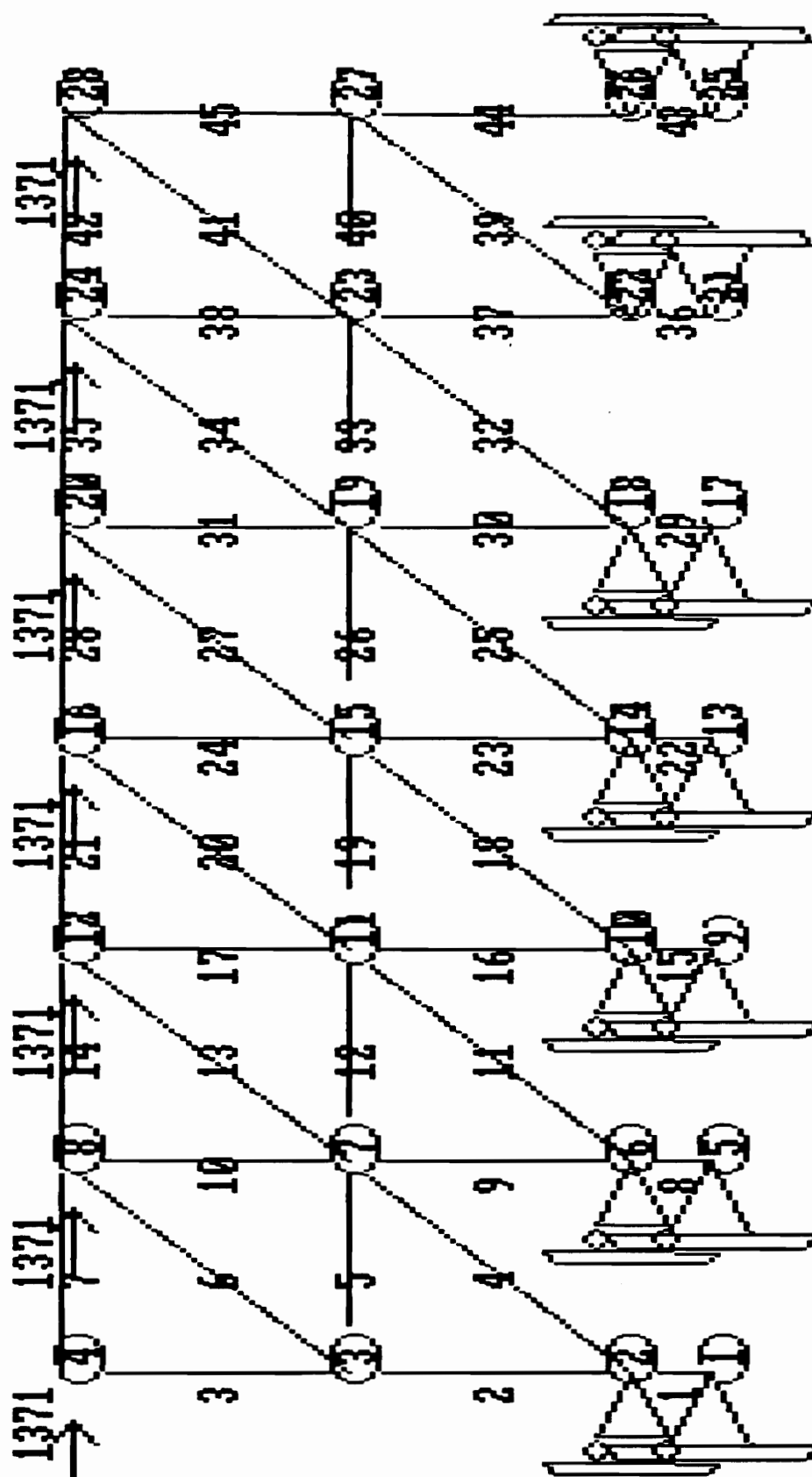


Figure 25. PPSA analog of 14.6 m (48 ft.) wide by 6.50 m (21.3 ft.) tall solid endwall.

Appendix F. PPSA Analysis of Braced 48 ft. Endwall with 16 ft. Door in Center

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*****
***                                     ***
***      PURDUE PLANE STRUCTURES ANALYZER 3.00      ***
***                                     ***
***              COPYRIGHT (1987)                    ***
***      PURDUE RESEARCH FOUNDATION                  ***
***                                     ***
***      THIS PROGRAM IS INTENDED TO FACILITATE      ***
***      THE ANALYSIS OF WOOD STRUCTURES. IT        ***
***      GIVES AN ACCURATE REPORT OF STRUCTURAL      ***
***      RESPONSE OF THE INPUT ANALOG.              ***
***      ACCURACY OF THE ANALOG AND INTERPRETA-     ***
***      TION OF STRUCTURAL ADEQUACY ARE THE         ***
***      RESPONSIBILITY OF THE USER.                ***
***                                     ***
*****
48 ft. braced endwall with 16 ft. centered door.
```

NUMBER OF NODES	= 31
NUMBER OF MEMBERS	= 48
NUMBER OF ROLLER SUPPORTS	= 6
NUMBER OF PINNED SUPPORTS	= 6
NUMBER OF FIRC SUPPORTS	= 0
NUMBER OF FIXED SUPPORTS	= 0
NUMBER OF LOADING ARRANGEMENTS	= 1

TABLE I ALLOWABLE MEMBER STRESSES IN PSI
NORMAL LOAD DURATION

MEMBER GROUP	USE TYPE	ALLOWABLE			WIDTH	DEPTH	MODULUS OF ELASTICITY
		BEND	COMP	TENS			
1	U	750.	700.	475.	5.500	5.500	0.140E+07
2	UI	750.	700.	475.	5.500	5.500	0.140E+07
3	SI	725.	675.	375.	3.000	11.250	0.140E+07
4	SI	725.	675.	375.	1.500	5.500	0.140E+07
5	FI	9000.	0.	0.	0.100	2.000	0.290E+08
6	FI	9000.	0.	0.	4.380	4.380	0.120E+05
7	FI	9000.	0.	0.	1.000	1.000	0.544E+07
8	FI	9000.	0.	0.	5.500	5.500	0.290E+08

TABLE II NODE COORDINATES

NODE NO.	X-COORD (IN)	Y-COORD (IN)
1	96.000	0.000
2	96.000	36.000
3	97.200	37.600
4	96.000	164.000
5	96.000	292.000
6	192.000	0.000
7	192.000	36.000
8	192.000	164.000
9	192.000	292.000
10	288.000	292.000
11	384.000	0.000
12	384.000	36.000
13	385.200	37.600
14	384.000	164.000
15	384.000	292.000
16	480.000	0.000
17	480.000	36.000
18	480.000	164.000
19	480.000	292.000
20	0.000	0.000
21	0.000	36.000
22	1.200	37.600
23	0.000	164.000
24	0.000	292.000
25	576.000	0.000

26	576.000	36.000
27	481.200	37.600
28	576.000	164.000
29	576.000	292.000
30	1.200	165.600
31	385.200	165.600

TABLE III MEMBER LAYOUT

MEMBER NUMBER	NEGATIVE END NODE	CONDITION	POSITIVE END NODE	CONDITION	MEMBER GROUP
1	1	PNND	2	RIGD	2
2	2	RIGD	4	RIGD	2
3	4	RIGD	5	RIGD	2
4	2	PNND	3	RIGD	6
5	3	RIGD	8	PNND	5
6	2	PNND	8	PNND	7
7	4	PNND	9	PNND	7
8	5	RIGD	9	RIGD	4
9	4	PNND	8	PNND	8
10	9	RIGD	10	RIGD	3
11	10	RIGD	15	RIGD	3
12	11	PNND	12	RIGD	2
13	12	RIGD	14	RIGD	2
14	14	RIGD	15	RIGD	2
15	12	PNND	13	RIGD	6
16	13	RIGD	29	PNND	5
17	12	PNND	18	PNND	7
18	14	PNND	19	PNND	7
19	15	RIGD	19	RIGD	4
20	14	PNND	18	PNND	8
21	6	PNND	7	RIGD	2
22	7	RIGD	8	RIGD	2
23	8	RIGD	9	RIGD	2
24	16	PNND	17	RIGD	2
25	17	RIGD	18	RIGD	2
26	18	RIGD	19	RIGD	2
27	20	PNND	21	RIGD	1
28	21	RIGD	23	RIGD	1
29	23	RIGD	24	RIGD	1
30	21	PNND	22	RIGD	6
31	22	RIGD	9	PNND	5
32	21	PNND	4	PNND	7
33	23	PNND	5	PNND	7
34	24	RIGD	5	RIGD	4
35	23	PNND	4	PNND	8
36	25	PNND	26	RIGD	1

37	26	RIGD	28	RIGD	1
38	28	RIGD	29	RIGD	1
39	17	PNND	27	RIGD	6
40	27	RIGD	28	PNND	5
41	17	PNND	28	PNND	7
42	18	PNND	29	PNND	7
43	19	RIGD	29	RIGD	4
44	18	PNND	28	PNND	8
45	23	PNND	30	RIGD	6
46	30	RIGD	5	PNND	5
47	14	PNND	31	RIGD	6
48	31	RIGD	19	PNND	5

TABLE IV REACTION CONDITIONS

NODE NUMBER	REACTION TYPE	HORIZ DISPL	VERT DISPL
1	pin	0.00	0.00
2	roll	0.00	1.00
6	pin	0.00	0.00
7	roll	0.00	1.00
11	pin	0.00	0.00
12	roll	0.00	1.00
16	pin	0.00	0.00
17	roll	0.00	1.00
20	pin	0.00	0.00
21	roll	0.00	1.00
25	pin	0.00	0.00
26	roll	0.00	1.00

TABLE V MEMBER PROPERTIES

MEMBER NUMBER	LENGTH (IN)	IN PLANE COLUMN (IN)	PRP PLANE COLUMN (IN)	BEAM LENGTH (IN)
1	36.000	0.000	0.000	75.180
2	128.000	0.000	0.000	235.520
3	128.000	0.000	0.000	235.520
4	2.000	0.000	0.000	0.000
5	158.000	0.000	0.000	0.000
6	160.000	0.000	0.000	0.000
7	160.000	0.000	0.000	0.000
8	96.000	0.000	0.000	0.000
9	96.000	0.000	0.000	0.000

10	96.000	0.000	0.000	0.000
11	96.000	0.000	0.000	0.000
12	36.000	0.000	0.000	75.180
13	128.000	0.000	0.000	235.520
14	128.000	0.000	0.000	235.520
15	2.000	0.000	0.000	0.000
16	318.000	0.000	0.000	0.000
17	160.000	0.000	0.000	0.000
18	160.000	0.000	0.000	0.000
19	96.000	0.000	0.000	0.000
20	96.000	0.000	0.000	0.000
21	36.000	0.000	0.000	75.180
22	128.000	0.000	0.000	235.520
23	128.000	0.000	0.000	235.520
24	36.000	0.000	0.000	75.180
25	128.000	0.000	0.000	235.520
26	128.000	0.000	0.000	235.520
27	36.000	0.000	0.000	75.180
28	128.000	0.000	0.000	235.520
29	128.000	0.000	0.000	235.520
30	2.000	0.000	0.000	0.000
31	318.000	0.000	0.000	0.000
32	160.000	0.000	0.000	0.000
33	160.000	0.000	0.000	0.000
34	96.000	0.000	0.000	0.000
35	96.000	0.000	0.000	0.000
36	36.000	0.000	0.000	75.180
37	128.000	0.000	0.000	235.520
38	128.000	0.000	0.000	235.520
39	2.000	0.000	0.000	0.000
40	158.000	0.000	0.000	0.000
41	160.000	0.000	0.000	0.000
42	160.000	0.000	0.000	0.000
43	96.000	0.000	0.000	0.000
44	96.000	0.000	0.000	0.000
45	2.000	0.000	0.000	0.000
46	158.000	0.000	0.000	0.000
47	2.000	0.000	0.000	0.000
48	158.000	0.000	0.000	0.000

LOADING 1

THE STRESS ADJUSTMENT FACTOR IS 1.33

TABLE VI C
THE STRUCTURE HAS NODE LOADS AS FOLLOWS

NODE NUMBER	DIRECTION OF LOAD	LOAD IN LBS OR IN-LBS AS APPROPRIATE
9	1	1371.0
5	1	1371.0
10	1	1371.0
24	1	1371.0
15	1	1371.0
19	1	1371.0
29	1	1371.0

***** RESULTS *****

TABLE VII REACTIONS

REACTION NODE	HOR. COMP. (LBS)	VERT. COMP. (LBS)	MOMENT (IN-LBS)
1	56.264	136.438	0.000
2	-2329.956	0.000	0.000
6	50.852	6491.937	0.000
7	-61.734	0.000	0.001
11	44.364	-6162.537	0.000
12	-2523.649	-0.003	0.001
16	52.199	181.120	0.000
17	-2171.422	-0.000	0.000
20	56.086	-6663.365	0.000
21	-2753.157	-0.002	-0.000
25	53.559	6016.410	0.000
26	-70.169	0.000	-0.001

SUM OF LOADS	9597.000	0.000	-0.280E+07
SUM OF REACTS	-9596.764	-0.000	0.280E+07
DIFFERENCE	0.236	0.000	36.000

*** STRENGTH ANALYSIS ***

TABLE VIII MEMBER END ACTIONS

MEMBER NUMBER	LOCATION	AXIAL (LBS)	SHEAR (LBS)	MOMENT (IN-LBS)
1	NEG END	136.438	-56.264	0.000
	POS END	-136.438	56.264	-2025.493
2	NEG END	3146.131	16.423	2025.493
	POS END	-3146.131	-16.423	76.683
3	NEG END	3083.917	8.399	-76.685
	POS END	-3083.917	-8.399	1151.806
4	NEG END	-1693.437	0.001	0.000
	POS END	1693.437	-0.001	0.002
5	NEG END	-1693.437	-0.000	-0.002
	POS END	1693.437	0.000	0.000
6	NEG END	-2068.678	0.000	0.000
	POS END	2068.678	0.000	0.000
7	NEG END	-2260.698	0.000	0.000
	POS END	2260.698	0.000	0.000
8	NEG END	422.764	-4.903	-461.810
	POS END	-422.764	4.903	-8.841
9	NEG END	2239.915	0.000	0.000
	POS END	-2239.915	0.000	0.000
10	NEG END	-868.786	-34.990	-3182.185
	POS END	868.786	34.990	-176.820
11	NEG END	502.195	-34.990	176.819
	POS END	-502.195	34.990	-3535.824
12	NEG END	-6162.537	-44.364	0.000
	POS END	6162.537	44.364	-1597.119
13	NEG END	-2866.053	6.919	1597.120
	POS END	2866.053	-6.919	-711.472
14	NEG END	34.893	31.671	711.472
	POS END	-34.893	-31.671	3342.454
15	NEG END	-1950.254	-0.010	0.000
	POS END	1950.254	0.010	-0.020
16	NEG END	-1950.254	0.000	0.020

	POS END	1950.254	-0.000	-0.000
17	NEG END	-2170.355	0.000	0.000
	POS END	2170.355	0.000	0.000
18	NEG END	-1993.935	0.000	0.000
	POS END	1993.935	0.000	0.000
19	NEG END	1841.517	-0.097	193.371
	POS END	-1841.517	0.097	-202.680
20	NEG END	2200.436	0.000	0.000
	POS END	-2200.436	0.000	0.000
21	NEG END	6491.937	-50.852	0.000
	POS END	-6491.937	50.852	-1830.671
22	NEG END	6491.938	10.882	1830.672
	POS END	-6491.938	-10.882	-437.814
23	NEG END	3482.245	28.350	437.815
	POS END	-3482.245	-28.350	3191.026
24	NEG END	181.120	-52.199	0.000
	POS END	-181.120	52.199	-1879.174
25	NEG END	2986.581	15.127	1879.174
	POS END	-2986.581	-15.127	57.104
26	NEG END	2883.301	7.812	-57.103
	POS END	-2883.301	-7.812	1057.027
27	NEG END	-6663.365	-56.086	0.000
	POS END	6663.365	56.086	-2019.100
28	NEG END	-3088.821	16.161	2019.100
	POS END	3088.821	-16.161	49.486
29	NEG END	-15.788	6.063	-49.487
	POS END	15.788	-6.063	825.592
30	NEG END	-2129.718	-0.001	0.000
	POS END	2129.718	0.001	-0.002
31	NEG END	-2129.719	0.000	0.002
	POS END	2129.719	-0.000	-0.000
32	NEG END	-2338.465	0.000	0.000
	POS END	2338.465	0.000	0.000

33	NEG END	-2112.215	0.000	0.000
	POS END	2112.215	0.000	0.000
34	NEG END	1364.946	-15.787	-825.591
	POS END	-1364.946	15.787	-689.997
35	NEG END	2294.591	0.000	0.000
	POS END	-2294.591	0.000	0.000
36	NEG END	6016.410	-53.559	0.000
	POS END	-6016.410	53.559	-1928.118
37	NEG END	6016.411	16.610	1928.117
	POS END	-6016.411	-16.610	198.023
38	NEG END	3210.950	5.086	-198.024
	POS END	-3210.950	-5.086	849.010
39	NEG END	-1578.523	-0.008	0.000
	POS END	1578.523	0.008	-0.017
40	NEG END	-1578.523	0.000	0.017
	POS END	1578.523	-0.000	0.000
41	NEG END	-1928.303	0.000	0.000
	POS END	1928.303	0.000	0.000
42	NEG END	-2041.254	0.000	0.000
	POS END	2041.254	0.000	0.000
43	NEG END	1028.993	-17.743	-854.348
	POS END	-1028.993	17.743	-849.012
44	NEG END	2115.826	0.000	0.000
	POS END	-2115.826	0.000	0.000
45	NEG END	-1729.075	0.003	0.000
	POS END	1729.075	-0.003	0.006
46	NEG END	-1729.075	-0.000	-0.006
	POS END	1729.075	0.000	-0.000
47	NEG END	-1632.248	-0.005	0.000
	POS END	1632.248	0.005	-0.009
48	NEG END	-1632.249	0.000	0.009
	POS END	1632.249	-0.000	-0.000

TABLE IX

INTERACTION ANALYSIS

USING MAX MOMENT AND MAX FORCE

MEMBER NUMBER - TYPE	ADDNL INFO	MAX INT VAL	LOC FROM NEG END		STRESS		L/D
			M (IN)	P (IN)	BENDING (PSI)	AXIAL (PSI)	
1-UI		0.078	36.00	36.00	-73.05	-4.51	6.545
2-UI	P	0.196	0.00	128.00	-73.05	-104.00	18.618
3-UI	P	0.162	128.00	128.00	41.54	-101.95	18.618
8-SI		0.120	0.00	96.00	61.07	-51.24	9.336
10-SI		0.104	0.00	96.00	50.29	25.74	6.827
11-SI		0.075	96.00	96.00	-55.87	-14.88	8.533
12-UI		0.380	36.00	36.00	-57.60	203.72	5.236
13-UI		0.208	0.00	128.00	-57.60	94.75	18.618
14-UI	P	0.122	128.00	128.00	120.54	-1.15	18.618
19-SI		0.276	96.00	96.00	-26.80	-223.21	9.335
21-UI		0.297	36.00	36.00	-66.02	-214.61	6.545
22-UI	P	0.319	0.00	128.00	-66.02	-214.61	18.618
23-UI	P	0.251	128.00	128.00	115.08	-115.12	18.618
24-UI		0.074	36.00	36.00	-67.77	-5.99	6.545
25-UI	P	0.184	0.00	128.00	-67.77	-98.73	18.618
26-UI	P	0.150	128.00	128.00	38.12	-95.32	18.618
27-U		0.422	36.00	36.00	-72.82	220.28	6.545
28-U		0.235	0.00	128.00	-72.82	102.11	23.273
29-U		0.031	128.00	128.00	29.77	0.52	23.273
34-SI		0.298	0.00	96.00	109.17	-165.45	9.958
36-U		0.283	36.00	36.00	-69.53	-198.89	6.545
37-U	P	0.341	0.00	128.00	-69.53	-198.89	23.273
38-U	P	0.176	128.00	128.00	30.62	-106.15	23.273
43-SI		0.256	0.00	96.00	112.97	-124.73	9.958

P INTERACTION VALUE CRITICAL FOR PERPENDICULAR PLANE

TABLE IXA

MEMBER ANALYSIS DATA

		FINAL ADJUSTED STRESSES			

MEMBER NUMBER	LENGTH (IN)	FAXIAL (PSI)	FBEND (PSI)	FBEND PRIME (PSI)	IN PLANE L/D RATIO
1	36.000	-931.00	997.50	997.50	6.545

2	128.000	-849.66	997.50	997.50	14.822
3	128.000	-849.66	997.50	997.50	16.430
8	96.000	-897.75	964.25	964.25	9.336
10	96.000	498.75	964.25	964.25	6.827
11	96.000	-897.75	964.25	964.25	8.533
12	36.000	631.75	997.50	997.50	5.236
13	128.000	631.75	997.50	997.50	18.618
14	128.000	-849.66	997.50	997.50	14.197
19	96.000	-897.75	964.25	964.25	9.335
21	36.000	-931.00	997.50	997.50	6.545
22	128.000	-849.66	997.50	997.50	14.822
23	128.000	-849.66	997.50	997.50	14.197
24	36.000	-931.00	997.50	997.50	6.545
25	128.000	-849.66	997.50	997.50	14.822
26	128.000	-849.66	997.50	997.50	16.430
27	36.000	631.75	997.50	997.50	6.545
28	128.000	631.75	997.50	997.50	23.273
29	128.000	631.75	997.50	997.50	23.273
34	96.000	-897.75	964.25	964.25	9.958
36	36.000	-931.00	997.50	997.50	6.545
37	128.000	-732.40	997.50	997.50	14.822
38	128.000	-732.40	997.50	997.50	17.371
43	96.000	-897.75	964.25	964.25	9.958

TABLE X SHEAR STRESS ANALYSIS

MEMBER	MAX. SHEAR STRESS (PSI)	LOC. FROM NEG. END (IN)	MEMBER LENGTH (IN)
1	-2.79	36.000	36.000
2	0.81	128.000	128.000
3	0.42	128.000	128.000
8	-0.89	96.000	96.000
10	-1.56	96.000	96.000
11	-1.56	96.000	96.000
12	-2.20	36.000	36.000
13	0.34	128.000	128.000
14	1.57	128.000	128.000
19	-0.02	96.000	96.000
21	-2.52	36.000	36.000
22	0.54	128.000	128.000
23	1.41	128.000	128.000
24	-2.59	36.000	36.000
25	0.75	128.000	128.000
26	0.39	128.000	128.000
27	-2.78	36.000	36.000
28	0.80	128.000	128.000

29	0.30	128.000	128.000
34	-2.87	96.000	96.000
36	-2.66	36.000	36.000
37	0.82	128.000	128.000
38	0.25	128.000	128.000
43	-3.23	96.000	96.000

*** DEFLECTION ANALYSIS ***

TABLE XI MAXIMUM MEMBER DEFLECTIONS

MEMBER	MAX. DEFL. (IN)	LOC. FROM NEG. END (IN)	MEMBER LENGTH (IN)
1	0.002	21.000	36.000
2	-0.135	128.000	128.000
3	-0.284	128.000	128.000
8	-0.037	64.000	96.000
10	-0.036	0.000	96.000
11	0.014	96.000	96.000
12	0.001	21.000	36.000
13	-0.126	128.000	128.000
14	-0.281	128.000	128.000
19	-0.018	96.000	96.000
21	0.001	21.000	36.000
22	-0.135	128.000	128.000
23	-0.281	128.000	128.000
24	0.001	21.000	36.000
25	-0.126	128.000	128.000
26	-0.266	128.000	128.000
27	0.002	21.000	36.000
28	-0.135	128.000	128.000
29	-0.295	128.000	128.000
34	-0.019	96.000	96.000
36	0.002	21.000	36.000
37	-0.125	128.000	128.000
38	-0.257	128.000	128.000
43	-0.033	96.000	96.000

TABLE XII NODE DISPLACEMENTS

NODE NUMBER	DISPL. HORIZ. OR IN DIR. OF ROLLER (IN)	DISPLACEMENT VERTICAL (IN)	DISPLACEMENT ROTATIONAL (RADIAN)
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1	0.000	0.000	0.000E+00
2	0.000	-0.000	-0.253E-03
3	0.010	0.011	-0.768E-03
4	0.135	-0.010	-0.142E-02
5	0.284	-0.019	-0.685E-03
6	0.000	0.000	0.000E+00
7	0.000	-0.006	-0.229E-03
8	0.135	-0.025	-0.159E-02
9	0.281	-0.036	0.618E-04
10	0.282	-0.009	0.351E-03
11	0.000	0.000	0.000E+00
12	0.000	0.005	-0.200E-03
13	0.011	0.018	-0.714E-03
14	0.126	0.014	-0.158E-02
15	0.281	0.014	-0.637E-05
16	0.000	0.000	0.000E+00
17	0.000	-0.000	-0.235E-03
18	0.126	-0.009	-0.133E-02
19	0.266	-0.018	-0.659E-03
20	0.000	0.000	0.000E+00
21	0.000	0.006	-0.252E-03
22	0.012	0.020	-0.779E-03
23	0.135	0.015	-0.143E-02
24	0.295	0.015	-0.909E-03
25	0.000	0.000	0.000E+00
26	0.000	-0.005	-0.241E-03
27	0.009	0.010	-0.713E-03
28	0.125	-0.023	-0.128E-02
29	0.257	-0.033	-0.650E-03
30	0.146	0.026	-0.871E-03
31	0.136	0.024	-0.820E-03

* * * * PROGRAM STOP.

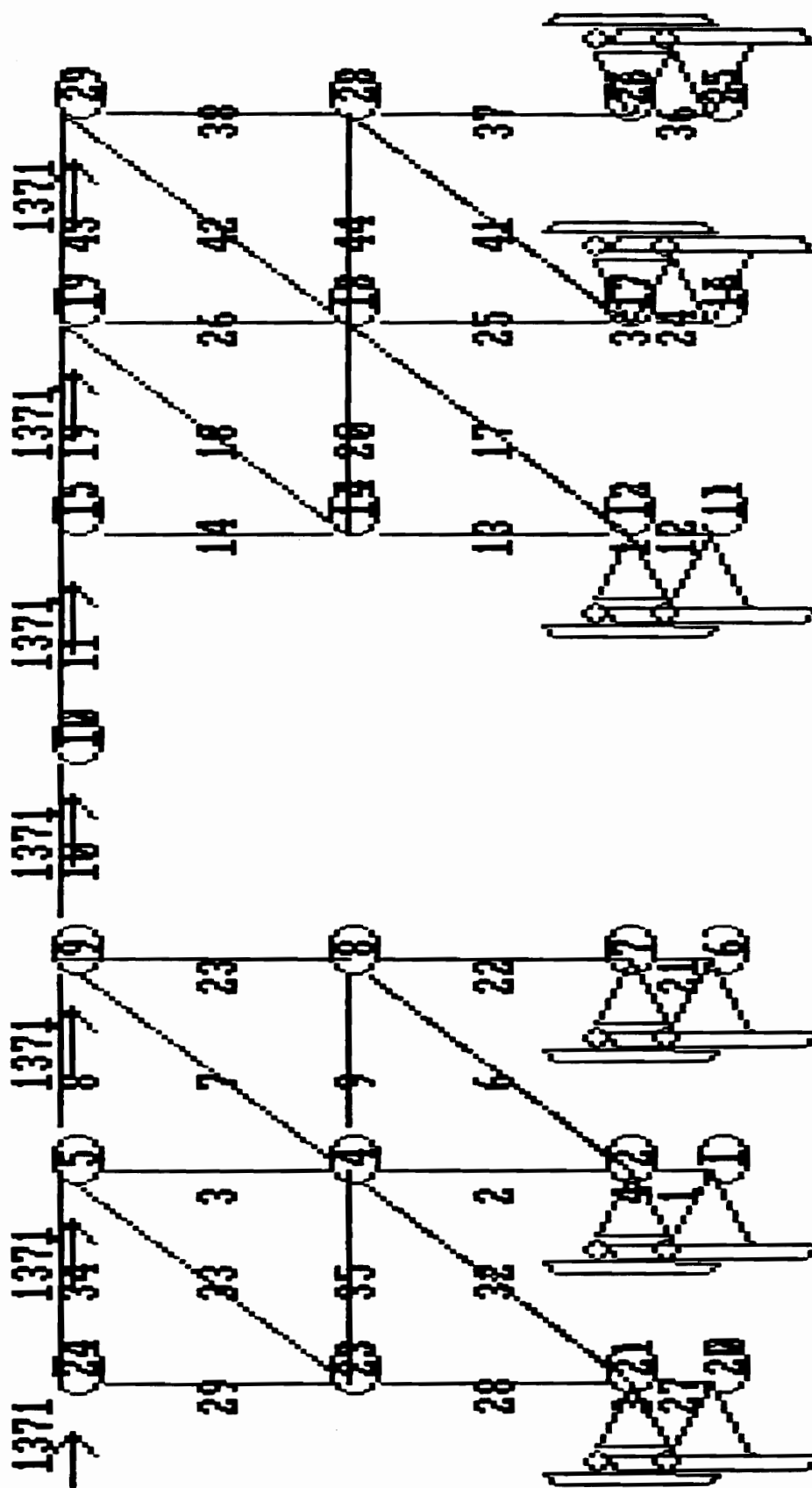


Figure 26. PPSA analog of 14.6 m (48 ft.) tall by 6.50 m (21.3 ft.) wide endwall having a 4.88 m (16 ft.) wide door.

Vita

Donald Lee Wirt Jr. was born in Radford, Virginia on July 22, 1966. He is the son of Mr. Donald L. Wirt and Mrs. Wanda B. Wirt of Christiansburg, Virginia. He graduated from Christiansburg High School in June 1984 and obtained a Bachelor of Science degree in Agricultural Engineering from Virginia Polytechnic Institute and State University in May 1989. He is a member of several professional and honor societies including the American Society of Agricultural Engineering, the Forest Products Research Society, and the Alpha Epsilon honor society. From 1989 to 1990 he was employed as a graduate research assistant at Virginia Polytechnic Institute and State University while studying for a Master of Science degree in Agricultural Engineering.