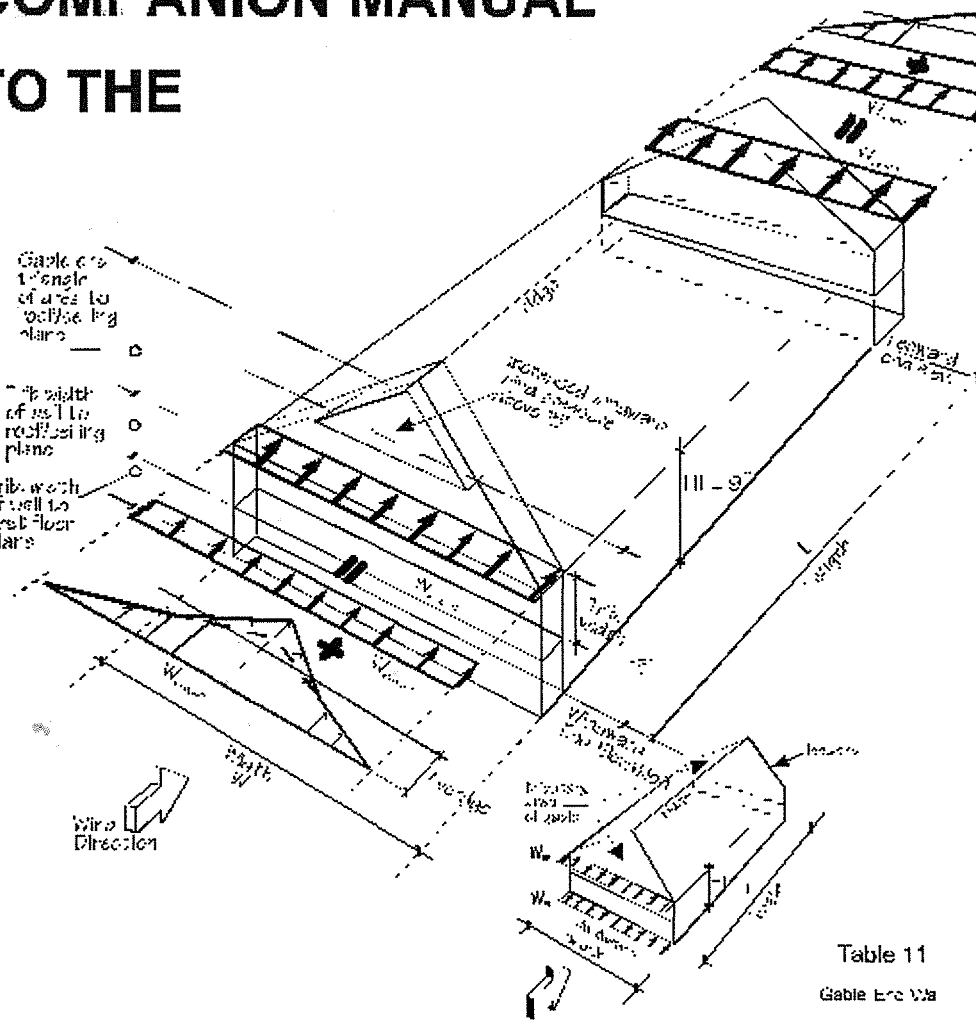
COMPANION MANUAL

TO THE



F 28'

1 STORY

Table 11

Gable End Wa

Wind Parallel to Ridge of Roof

Roof Slope	Lateral Wind Load to Diaphragms LB/- of Length							
	Fastest Mile Wind Speed (MPH)							
	80		90		100		110	
	W	W	W	W	W	W	W	W
3	108	202	137	257	169	315	205	384
4	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	333	226	435
5	130	224	165	265	203	350	247	[illegible]
6	141	235	179	299	221	369	266	446
7	152	246	193	313	232	385	280	467
8	162	257	208	327	256	405	310	459
9	[illegible]	[illegible]	[illegible]	[illegible]	273	420	332	511
10	187	281	237	357	292	439	355	533
12	213	309	271	393	335	483	404	585

1. Table based on ASCE 7-93 wind requirements for MWFRS.
2. H1 9FT based on 8FT floor to ceiling height. If 10FT floor to ceiling height, multiply table values by 1.25.
3. L 60 FT was used for tabled values.

WINDSTORM MITIGATION MANUAL

FOR

LIGHT FRAME CONSTRUCTION

December, 1999

COMPANION MANUAL

to the

WINDSTORM MITIGATION MANUAL

for

LIGHT FRAME CONSTRUCTION

CONTAINING TABLES, CHARTS AND GRAPHICS TO ASSIST ARCHITECTS, CONTRACTORS AND OWNERS IN DEVELOPING WIND RESISTANT SYSTEMS FOR GENERIC HOUSE CONSTRUCTION

JULY, 2000

Funds for research and compilation of the above Manual were provided by the Federal Emergency Management Agency under the Hazard Mitigation Grant Program, Project #1170.0003. Matching funds were provided by State Farm Fire and Casualty Company, Bloomington, Illinois and the Simpson Strong-Tie Company, Inc., Pleasanton, California.

The Manual project was administered by the Illinois Emergency Management Agency with assistance from the Building Research Council - School of Architecture, University of Illinois at Urbana-Champaign.

For more information regarding the Manual or the original Windstorm Mitigation Manual for Light Frame Construction, August, 1997, or for additional copies of either document, please contact the Illinois Emergency Management Agency, 110 East Adams, Springfield, IL 62701-1109 or call the IEMA Mitigation Staff at 217/782-8719.

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Foreword

This companion manual has been written for the sole purpose of mitigating storm damage from high wind conditions in residential light frame construction. The original manual, *Windstorm Mitigation Manual for Light Frame Construction*, explained the wind resistance philosophy and revealed typical wind related problems. This manual emphasizes structural planning principles that are essential to minimizing the cost of wind resistant wood frame construction. It also introduces tables and charts to assist architects, contractors and owners in developing a wind resistant system for generic house construction. The manual has taken a graphics approach to illustrate technically difficult concepts and engineering design techniques. The companion manual is written for the layperson, rather than the academic; the contractor rather than the engineer.

The information contained in chapter 1 is intended to establish the difference between tornado and straight-line winds. It explains a comprehensive wind design program initiated by the Building Safety Division of the City of Urbana, IL. The program was intended to educate the public, the residential construction industry and the design professionals within the community about the importance of wind resistant wood framed construction. It charts the cost of wind resistant construction for two distinctly different categories of home.

Chapter 2 reviews the structural planning issues for resistance to wind uplift which is the tension chain from roof to foundation. Two techniques are emphasized: total structural panel approach and total metal connector approach.

Chapter 3 reviews the structural planning issues related to the stability of the residence subjected to wind load, overturning and sliding. It introduces the concept of a diaphragm, a shear wall and a drag strut. This Chapter illustrates and explains the behavior of each stability issue and why each presents a problem in construction. The Chapter tells how to design to resist the effects of overturning and sliding. It conceptually reviews two examples to further clarify the stability concepts.

Chapter 4 develops a detailed wind uplift design procedure with illustrations. The procedure is for a generic house design. A numerical example is worked to aid familiarity with the procedure, and employs use of Appendix A Tables. The Tables work with this chapter to minimize much of the actual mathematics.

Chapter 5 develops a detailed wind stability design procedure, again with illustrations, to explain the Tables found in Appendix B. A numerical example is worked in total, incorporating all of the tables of Appendix B.

Selected references are found at the end of each Chapter, and nomenclature for use of the Appendices is also found at the end of Chapters 4 and 5.

Incorporation of the concepts, structural planning advice, design procedures and worked examples permit owners, architects, building officials and contractors to become conversant with wind resistant design. The use of this manual provides confidence to perform the wind design procedures by following the examples included. This manual will enhance the structural capability of the residence and provide a greater chance of mitigating structural damage from straight-line winds. The cost to incorporate these details is in the range of 3%-5% of the cost of new residential construction. A small price to pay for the potential benefit of security and safety.

A "quick start" for readers wanting to go right to the design solutions would be to read Chapters 2 and 3, and then move into Chapters 4 and 5 to use the design procedures and the tables from both Appendices.

Acknowledgements

This Manual was prepared through the Building Research Council (BRC) of the School of Architecture at the University of Illinois at Urbana/Champaign under contract to and sponsored by the Illinois Emergency Management Agency (IEMA), State Farm Insurance (SFI) and the Simpson Strong-Tie Company, Inc.

Special thanks for counsel and review of the draft versions of this manual are extended to Jan Horton, State Hazard Mitigation Officer for IEMA, Larry Sanders, Federal Hazard Mitigation Officer for FEMA, Phil Line, Staff Engineer for the American Forest & Paper Association and Randall Shackelford, P.E. and research Engineer for Simpson Strong-Tie Co., Inc.

Special appreciation is extended to the Staff at the BRC for responding to equipment and material needs, mailing and contract coordination. Particular recognition should go to Selah Peterson, graphic designer, for her dedicated work in converting my sketches into beautiful computer generated hard-line drawings, and for continuing to teach me how to edit and produce my own drawings. Special thanks for coordination of the contract go to Bill Rose, research architect.

I must also thank Joseph R. Hetzel, P.E., and Technical Director of the Door & Access Systems Manufacturers Association International (DASMA) for providing the garage door wind Tables.

Thanks go to all the individuals who reviewed drafts of this document.

David Wickersheimer, S.E., Architect, Professor of Architecture,
President of Wickersheimer Engineers. Inc.
December 31, 1999

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Chapter 1 - Wind Basics Revisited

Introduction

This *Companion Manual* complements the *Windstorm Mitigation Manual for Light Frame Construction*, published in August of 1997. References made to the *1997 Manual* occur throughout this text, and are referred to by the acronym *WMM* in Italic. There is no attempt to duplicate material found in the *WMM*. The two *Manuals* together allow decisions to be made about desired level of wind resistance, the selection of the wind forces to achieve the desired level of wind resistance, and the details which can resist those design forces.

This *Companion Manual* provides basic structural guidance in planning a home's layout and overall form to maximize the wind resistance properties of the construction, while minimizing the cost to implement the details required to achieve the selected level of wind resistance.

Tables and Charts are provided in the *Companion Manual* to aid in the selection of design uplift and stability forces, followed by the allowable capacities of specific recommended details. The appropriate choice of detail is one that provides an allowable capacity equal to or greater than the design force. This will assist contractors in establishing construction details and the cost to implement these details. Thus, the owner will know what part of the overall cost is attributable to their desired level of wind resistance.

Straight-line Winds vs. Tornadoes

Straight-line Winds

Straight-line winds are the typical winds described in Chapter 1 of the *WMM*. These straight-line winds form the probabilistic basis of the maximum wind speed isobar map used for wind design across the United States. The map of Figure 1.1 was in general usage up to 1996 [1.1], and is still used in the 1996 BOCA National Building Code. It is based on a fastest mile wind concept. Data was collected from 129 weather stations and has been statistically analyzed for a 50-year return period. Tornadoes have not been considered in developing the basic wind speed distributions.

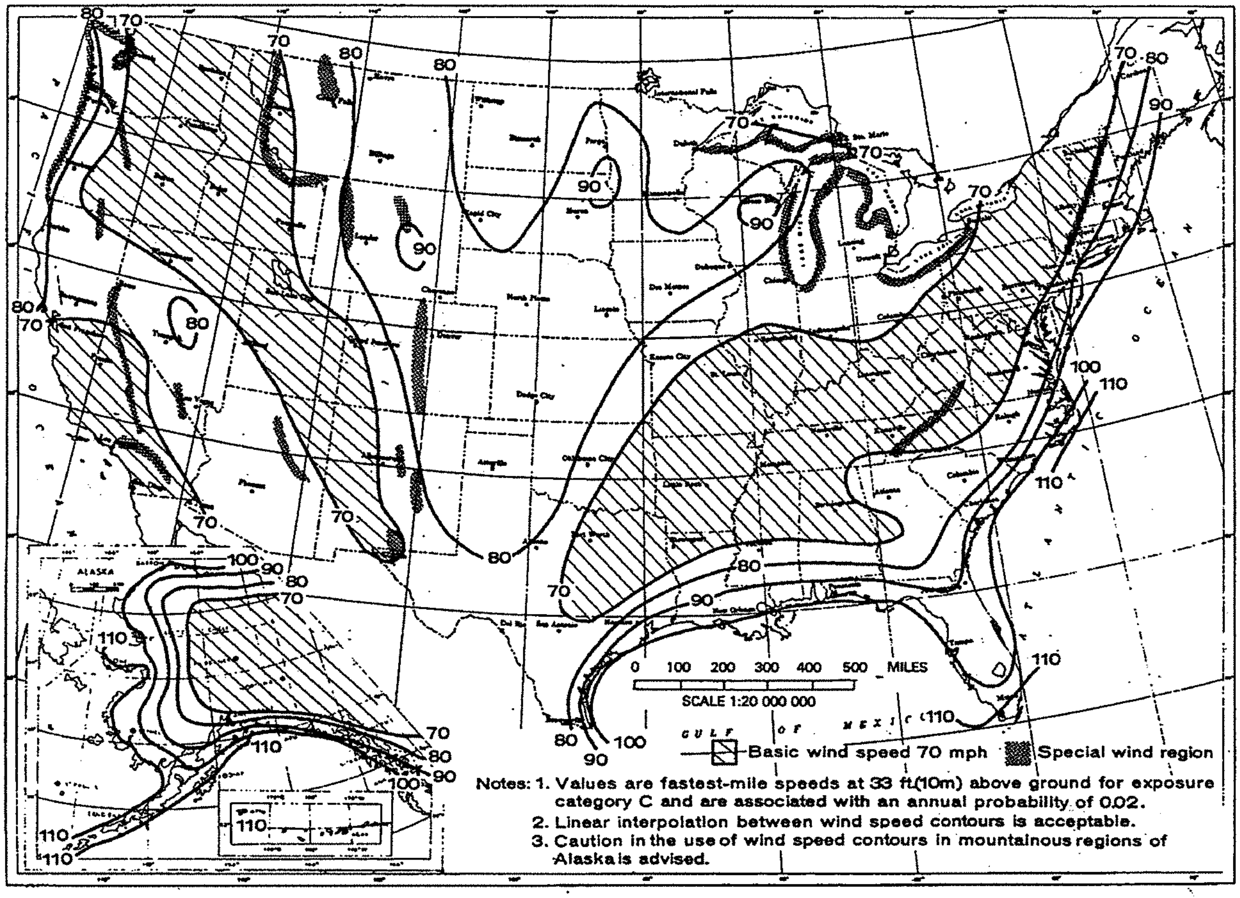


Figure 1.1 – Basic Wind Speed Map

Example:

To illustrate the use of the map, consider a 50-year wind in Champaign, IL. The map shows a 70-mph maximum wind speed. This means that Champaign has experienced or is expected to experience a wind speed greater than 70 mph on the average of once in 50 years. The probability of the wind speed exceeding 70 mph in Champaign within a given year is 2%, while the probability that this wind speed will be equaled or exceeded in 50 years is 64%[1.2].

It would seem that the scenario outlined in the example above, implies that the design wind speed for Champaign has a high probability of being exceeded within the assumed 50-year life of a building. However, buildings designed and constructed in accordance with current building codes, assuming reasonable standards of workmanship, have reserve strength to resist higher wind speeds than the design wind speed. The use of wood load duration factors, and factors of safety that establish an allowable stress level, would increase the actual exposure of the building to approximately a 500 year time period. The probability that the 70 mph wind speed will be equaled or exceeded at least once in 500 years is 100%. The inclusion of normal safety factors dramatically improves the chances for a structure to resist higher loads without collapse or even damage. Also, wind design assumes wind applied perpendicular to the longer surfaces of the building, which most often produces the worst possible wind loading case [1.3].

Why then do wind speeds, even less than the design wind speed, sometimes cause damage to buildings, or even collapse?

(1) Connections are generally the weakest link in the wind resistance system of any light frame construction. This was explained in the Chapters of the *WMM.* The studs, joists, rafters or trusses seldom fail as individual members subjected to bending, shear and axial stresses. If a member fails, it is usually after a connection fails. **Connections are the essence of adequate straight-line wind resistance.** This is not to imply that the breach of a garage door, window or patio door by the wind cannot be a catalyst for member damage. It does not imply that projectiles cannot also cause severe damage to structural members. These topics will be treated in depth in the following chapters.

The straight-line wind speed map of Figure 1.1 is shown in all three Model Building Codes: the 1996 Basic Building Code (BOCA), the 1994 Uniform Building Code (ICBO) and the 1995 Standard Building Code (SBCC). Each Code dominates in a particular region of the United States, and some States actually use a mixture of the model codes. These model codes are revised on a 3-year basis to reflect new knowledge, or clarify existing provisions. Although these three codes have their own specific provisions for wind resistance, they also reference the wind provisions of the ASCE 7 *Standard Minimum Design Loads for Buildings and Other Structures* [1.4] as an alternate procedure. The provisions of the BOCA Code and the ASCE 7 may start with different procedures, but they arrive at the same wind pressures and suctions used for design in most cases.

(2) Compliance with the minimum requirements of the accepted building code should imply adequate wind resistance with respect to the use of nails as defined therein. Yet residential construction techniques cited in Chapter 2 through 4 of the *WMM* clearly illustrated the potential inadequacies of platform framing. These inadequacies involve the type of exterior sheathing and associated nailing of that sheathing to the studs, plates, bandboards, mudsills, rafters and trusses. **"Accepted" construction practices that inadvertently introduce weak links into the tension uplift chain include**:

1) the use of nails for stud walls in conjunction with non-structural sheathing;
2) the use of nails for stud walls in conjunction with structural sheathing that does not completely tie the wood framed skeleton together; and
3) the methods by which the wood framing is anchored to the foundation and the amount of uplift resistance provided by that foundation.

The first two practices can produce withdrawal loading on nails that is a direct violation of all three model codes, while the third practice leads to separation of the home's superstructure from its foundation. Chapter 7 of the *WMM* provides options to improve the strength of light frame construction and overcome the three issues described above.

The three model codes are performance based and use similar language for structural design in wood construction. For example the 1996 BOCA Code states as follows:

2303.1 Structural design: All structural wood members **and connections** shall be of sufficient size or capacity to carry all design loads as required by Chapter 16 [Structural Loads] without exceeding the allowable design values specified in AFPA NDS [National Design Specification for Wood Construction, Revised 1991 Edition].

2305.2 Fastening: The quantity and size of fasteners connecting wood frame members together and sheathing materials to wood frame members **shall not be less than** that specified in Table 2305.2. [The Table establishes a minimum connection required for any framing condition. A portion of the Table related to the exterior wall tension chain is shown in Chapter 4 of the WMM].

Neither of the above provisions mention non-structural rigid-board (pink or blue) insulation as the likely "sheathing" that will cover the majority of the residence and create "weak links" in the uplift tension chain. The first BOCA provision above references the 1991 NDS for allowable stresses in members and the allowable loads on fasteners.

Part XII: Nails and Spikes in the NDS states:
12.2.2-Withdrawal from End Grain
Nails and spikes shall not be loaded in withdrawal from end grain of wood.
It becomes necessary for connection design to be done by an engineer when non-structural sheathing is used, since the Tables seemingly only provide minimum fasteners for conditions where structural wood sheathing is employed. The other two model codes either states the above NDS provision in their own words or reference the NDS.

Tornado Winds

Every year approximately one thousand tornadoes touch down in the United States. Only a small percentage actually strike occupied buildings; however, every year a number of people are killed or injured and an extensive amount of property damage results [1.5]. It has been stated in an article in the *Journal of the Structural Division of the ASCE* that "Tornadoes are the most violent of the weather phenomenon with which engineers must contend. To develop designs for tornadic loading, it is necessary to establish limits on ground level wind speeds that can occur in tornadoes"[1.6]. Wind Engineering by Henry Liu states "Tornado wind speed probability is based on fastest-gust speed rather than on fastest mile straight-line wind." Using this approach, it can be stated that "50% of tornadoes have maximum gust speeds lower than 100 mph, and only 10% of tornadoes have maximum speeds higher than 150 mph" [1.7]. These facts are re-phrased in the ASCE's Minimum Design Loads for Buildings and Other Structures Commentary as follows: "It is recognized that tornadic wind speeds have a significantly lower probability of occurrence at a point than the probability for basic wind speeds. In addition, it is found that in approximately one-half of the recorded tornadoes, gust speeds are less than the gust speeds" associated with basic wind speeds [1.8]. This adds credibility to the goal to improve straight-line wind load resistance in residential construction, since "straight-line winds cause far more cumulative damage than tornadoes"[1.9]. **It is feasible and economical to build straight-line wind resistant structures to withstand higher wind speeds than probability concepts would anticipate.**

As mentioned in the WMM, **it is not economically feasible to build entire tornado resistant residences**. It would be necessary to build underground structures or totally reinforced concrete structures to accomplish such resistance.

Current research leans toward construction of a "safe room", meaning a small interior room of reinforced masonry, concrete or steel walls with a concrete top [1.10]. One such safe room for occupants was proven effective in Tulsa, Oklahoma during the May 3, 1999 Category 4 tornado. The entire house was destroyed while the safe room remained. Owners will need to decide if the "safe room" alone, or in combination with straight-line windstorm resistance detailing is the desired approach for safety and also for minimizing construction damage.

Reasonable Expectations of Wind Resistant Design

Employing the wind resistant design details recommended in both Manuals has potential advantages for both the owner and the contractor.

The owner should generally benefit from:
- Improved overall safety
- Reduced damage to the residence
- Less chance of being displaced from the residence
- Less loss of personal property

The contractor should benefit from:
- Improved quality control
- Fewer call backs
- Enhanced reputation

Wind Resistant Construction Program

The Cities of Urbana, Decatur and Ogden Illinois experienced a tornado on April 19, 1996. The damage was devastating to all three communities; however, after a review of the repetitive failure mechanisms by engineers, FEMA/IEMA officials, and building inspectors, it was clear that much of the damage could have been reduced or even eliminated in many cases. After the publication of *Mitigating Storm Damage in Light Frame Construction* and the public presentations that followed in all three cities, the major question was "how much will it cost to rebuild with wind resistant construction techniques"?

The Building Safety Division of the City of Urbana proposed a program of increased awareness by the residential construction industry and the general public. A voluntary use of wind resistant techniques by local contractors and homeowners was promoted. The proposal received a grant in 1997 from the Federal Emergency Management Agency's (FEMA) Hazard Mitigation Grant Program which was administered by the Illinois Emergency Management Agency's (IEMA). The Illinois Department of Commerce and Community Affairs and the State Farm Fire and Casualty Company awarded funds to the City of Urbana.

The program consisted of several components:

- Presentations to builders and code officials of the basic chapters in the *WMM* concerning the mechanics of wind, the basics of the structural behavior of nails, the ideosyncrancies of platform framing, the wind tension chain, wind stability, wind resistant design techniques applied to new construction.
- Application of wind resistant construction techniques on two new homes and one retrofit project. These homes would serve as demonstration homes, and their techniques videotaped.
- Development of a video that focuses on the effects of wind on homes, evaluate present building performance, recommend alternative construction techniques for improved wind resistance, and provide a bibliography and resource list for further information.
- Initiate a permit rebate program, which offers an incentive to builders and potential homeowners to utilize a 90-mph minimum wind resistant design to construct their home in Urbana.

All three homes have been completed and the contractors have provided a cost breakdown of material and labor increases that were a result of incorporating wind resistant design techniques following the *WMM*. Table 1.1 summarizes basic information about the homes in the study, and the associated costs.

The two new homes represented both ends of the design spectrum: (1) a modest one-story home with approximately 1300 square feet of usable living space and a cost of $70,000; (2) a custom home with a variety of spatial features, numerous amenities and a price of $405,000. The retrofit house was a modest one-story home that was completely gutted and converted into a contemporary home with a partial second floor addition that cost $160,000.

All three homes were engineered to structurally handle a 90-mph straight-line wind. It is clear from the data in Table 1.1 that **the more expensive the home, the less impact the cost of including wind resistant construction has on the overall cost of the home**. This makes perfect sense on a percentage basis. The $70,000 home included approximately $5,600 for material, labor and engineering for the added wind resistance. The dollar amount does not seem high, but it becomes an almost 8% cost increase. The $405,000 home added almost $14,600, but this only amounted to 3.6% of the total cost of the house. The retrofit house is unique. It only had a crawl space of concrete block foundation walls that provided very little mass to resist the addition of a second story wind condition. Twelve cubic yards of concrete were added to provide overturning stability. The added cost for the engineering and construction amounted to $12,900 of the $160,000 total. **One can assume from this small study that wind resistant construction will cost between 5 % and 8 % of the construction cost when engineering is included.** It should be noted that all three homes had installed 90 mph rated garage doors and associated hardware. This added only approximately $200 to each house. Obviously, this is a small price to pay given the observed damages attributable to houses that had their garage doors breached.

Table 1.1 - Cost Comparison

	New	New	Retrofit
Construction Cost	$70,000	$405,000	$160,000
Description	1 story, crawl, flat bottom gable roof trusses, 2 car garage, conform insulation forms + construction foundation	2 story, partial flat bottom gable trusses, (2) 1 car garages, 5 fireplaces, partial cathedral ceiling, full basement, concrete cast in place	1 story, crawl, flat bottom gable trusses, concrete block foundation, partial 2 story
Square Feet	1308 (w/o garage)	1st floor 2975 2nd floor 1415 4390 w/o garage	1st floor 1732 2nd floor 564 2296 w/o garage
Cost/Square Ft	$53.50	$92.25	$69.70
Wind Resistant Construction Costs	OSB sheathing, additional A.B.s, footing & foundation, wall concrete, metal storm connectors, garage door upgrade	Metal wind connectors, fabricated steel for drag struts, extra reinforcement, epoxy anchors, upgrade foundation to all poured, garage door upgrade	Shear walls for added 2nd floor, 12 cubic yards of concrete, tie-downs in crawl space, OSB board, metal connectors, steel beam drag strut, garage door upgrade
Materials	$1942	$6122	
Labor	$2816	$3620	
Total	$4758	$9742	$7100
Cost/Square Ft	$3.64 (6.8%)	$2.22 (2.4%)	$3.09 (4.4%)
Engineering	$640	$4000	$4000
Observation visits	$160	$800	$800
Total	$800	$4800	$4800
Cost/Square Ft	$0.61 (1.14%)	$1.10 (1.2%)	$2.10 (3%)
% Const. Cost Eng. + Const.	7.9%	3.6%	7.4%

Wind Speed Map: A Footnote

The 1995 Edition of the ASCE-7 Minimum Design loads for Buildings and Other Structures [1.11], has introduced a new concept for wind speed determination: a "three-second gust", rather than the "fastest mile" philosophy that has been in place since 1972. The basic wind speed has been redefined as the peak gust that is recorded on instruments with various response characteristics and thus are averaged over three seconds. Data has been collected at 485 weather stations for a minimum of five years. The new maps as shown in Figure 1.2 and 1.3, as well as the map of Figure 1.1, similarly reflect an exposure category C and a 50-year mean recurrence interval. The new map is divided into counties to more easily determine where contours lie. Neither map is representative of wind speeds at which structural failures are expected to occur. As stated earlier in this Chapter, employed factors of safety leads to structural resistances and corresponding wind loads that are substantially higher than the speeds shown on the maps. Note that 85 and 90 mph cover the majority of the USA, except for the Gulf and Atlantic Coast Hurricane areas.

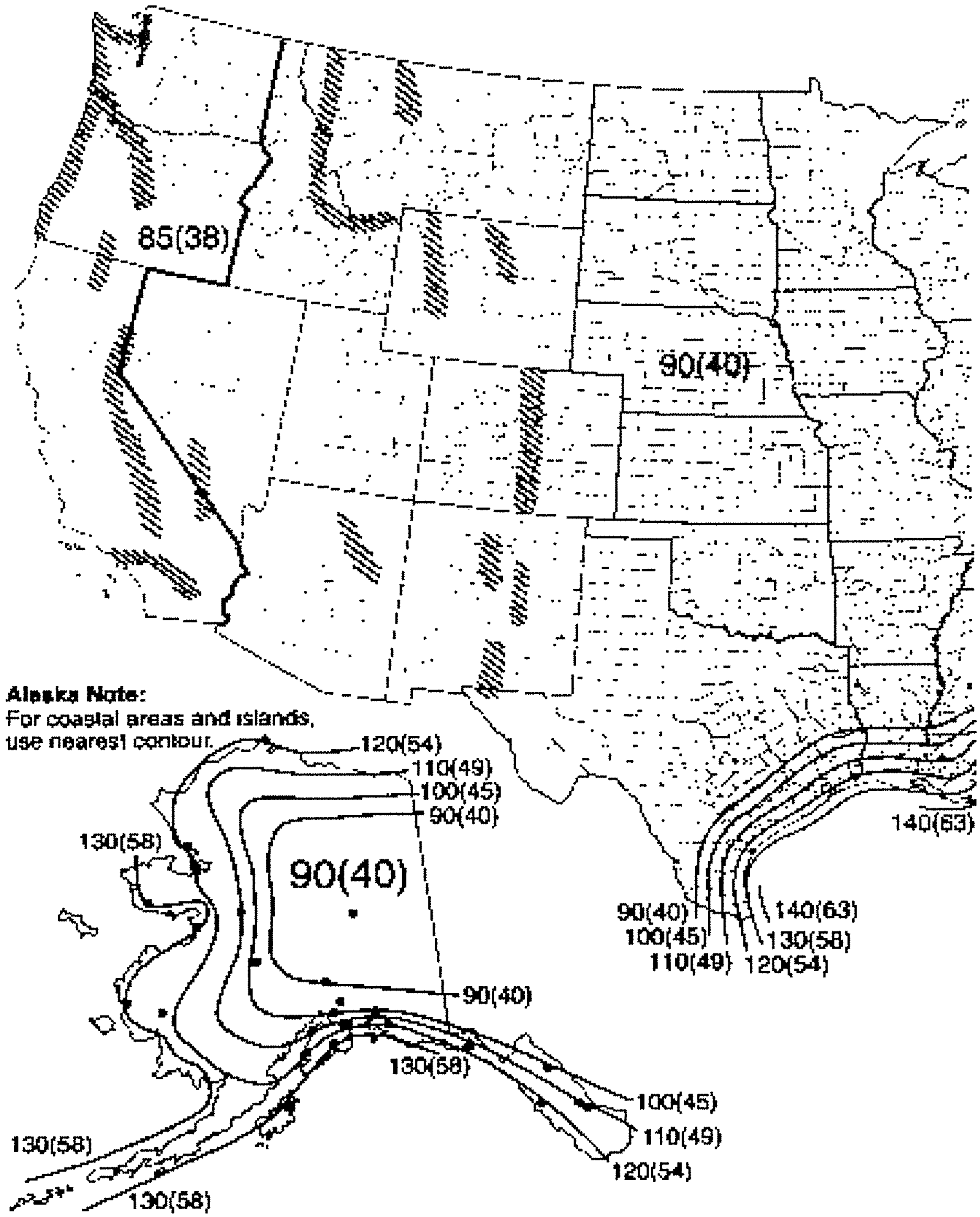
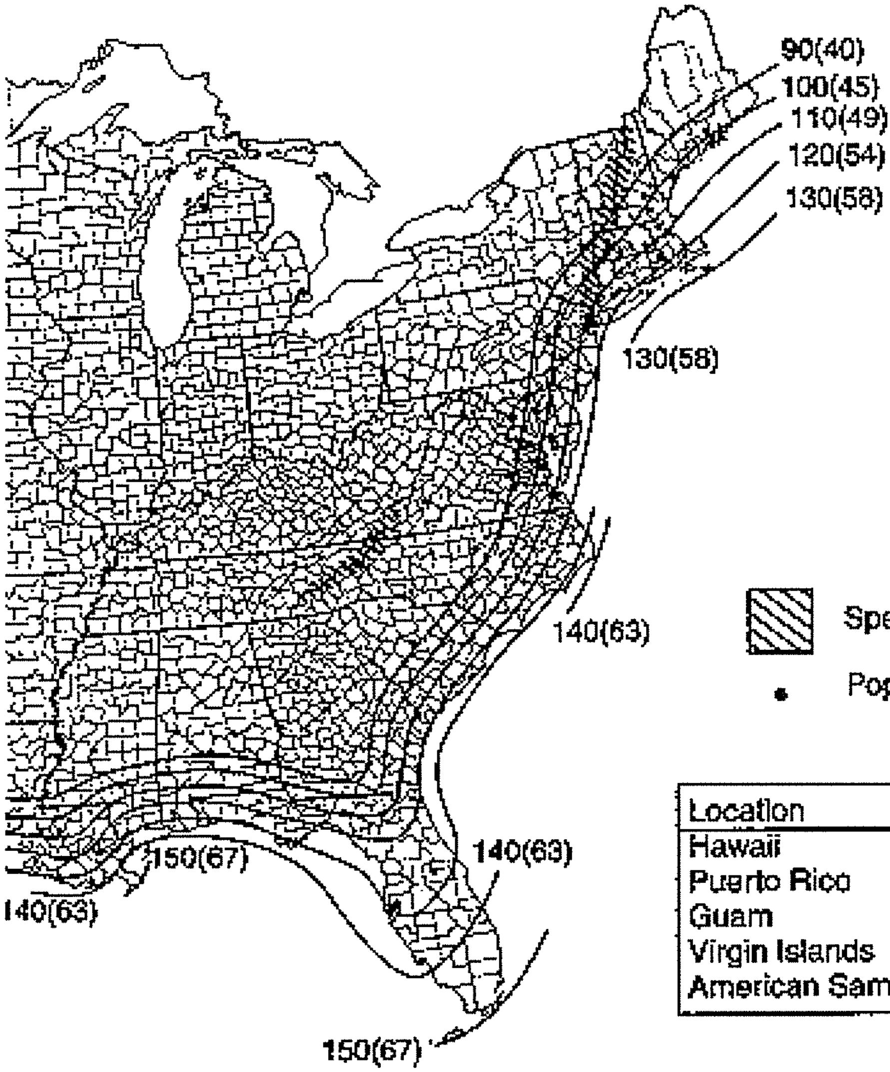


Figure 1.2 – ASCE 7-95 Wind Speed Map (part 1)



Special Wind Region

Population Center

Location	V mph	(m/s)
Hawaii	105	(47)
Puerto Rico	125	(56)
Guam	170	(76)
Virgin Islands	125	(56)
American Samoa	125	(56)

Notes:

1. Values are 3-second gust speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category and are associated with an annual probability of 0.02.
2. Linear interpolation between wind speed contours is permitted.
3. Islands and coastal areas shall use wind speed contour of coasta area.
4. Mountainous terrain, gorges, ocean promontories, and special wi regions shall be examined for unusual wind conditions.

Figure 1.3 – ASCE 7-95 Wind Speed Map (part 2)

A comparison of wind loads for one and two story structures was discussed in an article from the Journal of the American Institute of steel Construction:

"It is difficult to make a simple and direct comparison between the MWFRS loads found in the new Standard for buildings less than 60 feet in height and those in the 1993 version. ...Limited comparison based on base shear suggests a rather substantial decrease in the wind loads ...." [1.12]

One comparison between the fastest mile concept and the three-second gust concept would be to select a 100 mph wind speed and perform the calculations for main wind-force resistance system wind pressures and also for a single roof truss uplift based on components and cladding pressures. Figure 1.4a illustrates the results, which are significantly different for the two concepts. Note the base shear of 374 plf and the uplift of 485 plf is larger for the fastest mile concept of the ASCE 7-93 and BOCA 1996 documents.

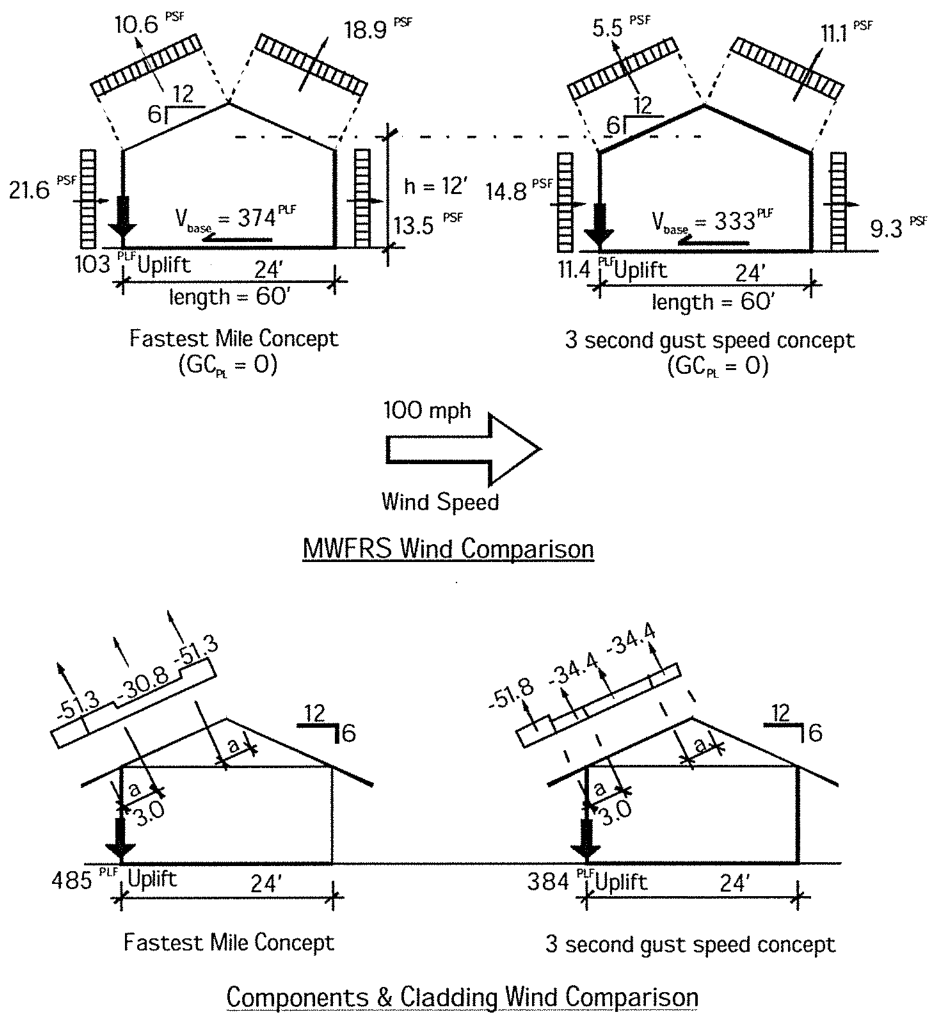


Figure 1.4a - ASCE 7-93 & 95 Comparison

Figure 1.4b compares the required wind speed for Springfield, Illinois based on the maps for a fastest mile concept and a three-second gust concept. This may be a more meaningful comparison, since it really shows what the differences are for a given geographic location that requires 70 mph using a fastest mile concept versus 90 mph required wind speed for a three-second gust concept. Note that the three-second gust concept produces the larger base shear for MWFRS and the larger uplift for a single truss component and cladding analysis.

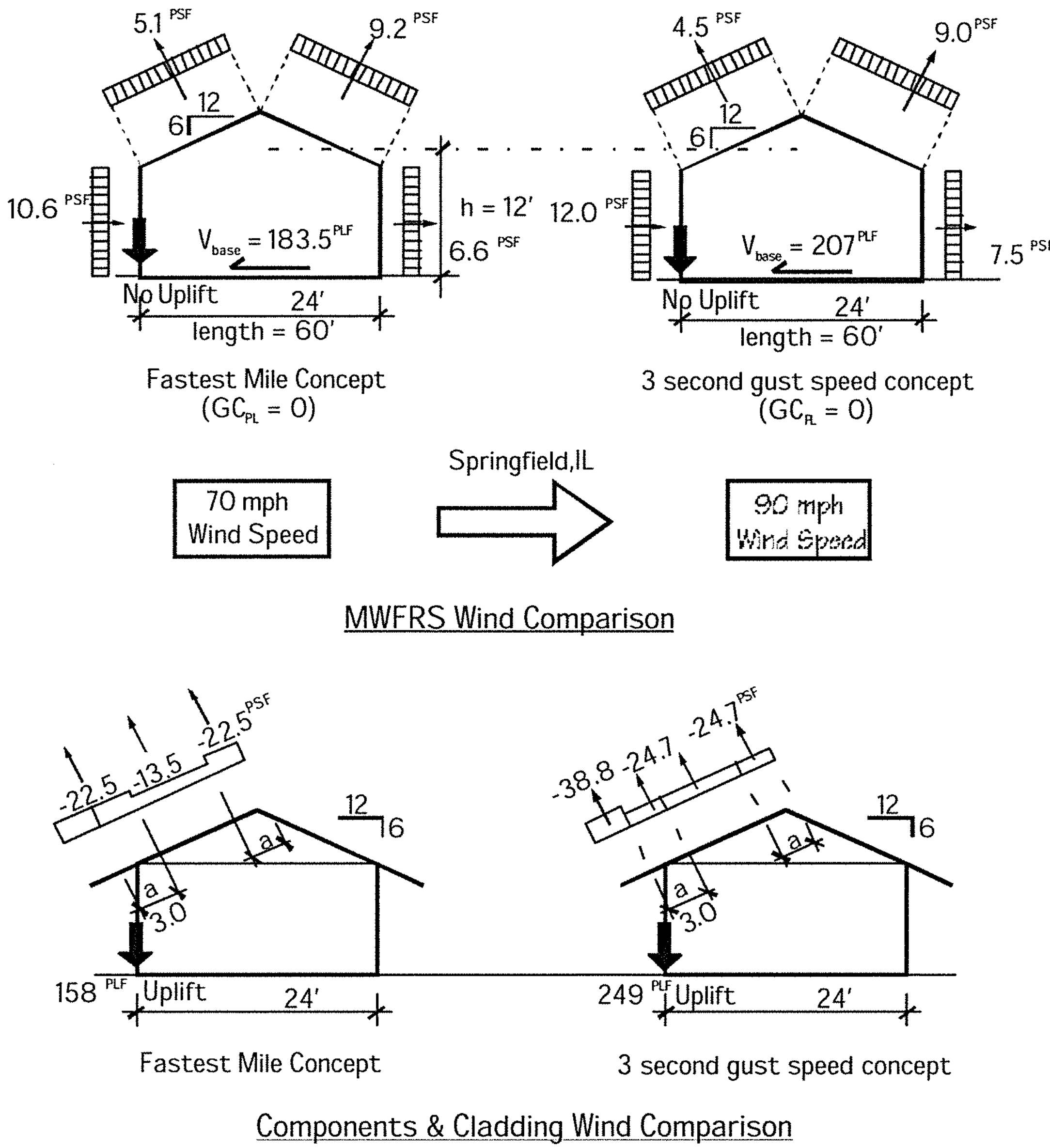


Figure 1.4b - ASCE 7-93 & 95 Geographic Location Comparison

Based on the variability of the above two comparisons, and given that the *WMM* used the fastest mile concept in all of its discussions, **this manual will continue to use the wind speed map of Figure 1.1 (ASCE 7-93).**

Conclusions

It is clear the design of buildings to resist wind loading is based on the concept of probability. Structures are designed to provide a specific degree of safety against high winds, determined by the probability of occurrence of wind speeds exceeding the design value. It is also clear that straight-line wind and associated gusting have a greater probability of causing damage to light frame construction than do tornado winds. Again, it should be emphasized this does not imply that homes can withstand the severe swirling and upward acting winds of the major tornadoes seen on television. It does suggest that…"if buildings and houses are designed using the wind load requirements referenced in the model codes and constructed properly, they should be able to resist as many as two-thirds of the tornadoes"[1.13].

Connections and structural sheathing continuity are the most important factors for wind resistant construction. These factors will be emphasized throughout this *Companion Manual.*

Selected References

1.1 "Minimum Design Loads for Buildings and Other Structures", *American Society of Civil Engineers*, ASCE 7-93, p. 13.
1.2 Liu, Henry, *Wind Engineering-A Handbook for Structural Engineers*, Prentice-Hall, New Jersey, 1991, Chapter2, pp. 37-38.
1.3 Liu, p. 37.
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1.5 Tornado Project, www.tornadoproject.com
1.6 Mehta, Kishor C., Minor, Joseph E., McDonald, James R., "Windspeed Analyses of April 3-4, 1974 Tornadoes", *Journal of the Structural Division*, ASCE, September, 1976, 1709.
1.7 Liu, p. 36.
1.8 ASCE 7-95, Commentary, p. 156.
1.9 Liu, p. 37.
1.10 "Taking Shelter From the Storm: Building a Safe Room Inside Your House", *Federal Emergency Management Agency*, First Ed., October 1998, data prepared by Wind Engineering Research Center, Texas Tech University.
1.11 "Minimum Design Loads for Buildings and Other Structures", *American Society of Civil Engineers*, ASCE 7-95, pp.18-19.
1.12 Gregory L.F. Chiu and Dale C. Perry, Low-Rise Building Wind Load Provisions – Where Are We and Where Do We Need To Go?, *Engineering Journal of The American Institute of Steel Construction*, Fourth Quarter, 1997, p. 147.
1.13 Gage, "panel discussion", *The Building Official and Code Administrator*, March/April, 1998, p. 19.

Chapter 2 - Wind Structural Planning Basics –The Tension Chain

Introduction

Chapter 2 and Chapter 3 are intended to assist homeowners, contractors and architects in the basic requirements for designing wind resistant light frame structures, as it relates to architectural house planning. There are structural "wind resistance" concepts that, if considered during the design of a house floor plan and its elevations, will reduce the cost impact of a wind resistance system. Many times the mere shifting of a wall a foot or two one way or the other will significantly improve the structural design for wind resistance, without greatly impacting the house layout. Shifting a window or door may contribute to improved wind resistance. Aligning specific first and second floor walls may greatly improve wind resistance. Alignment of the foundation with the first and second floor walls will improve wind resistance. These are just a few of the items that might make inclusion of wind resistant design affordable, and provide the expectations described in Chapter 1.

This Chapter includes a structural planning discussion of the wind uplift chain of resistance for solid walls, walls with openings, porches, second floor cantilevers, and insulating concrete forms for cast-in-place concrete construction. Also, this Chapter will explore ideas that contractors have used, after reading the *WMM*, simplifying wind resistant detailing for their operation, and reducing time and material in the process.

Chapter 4 will use the information of this Chapter to present numerical examples of uplift resistance design and design tables for general application related to the tension chain.

Wind Loads for Design

There are two categories of design wind loads on buildings and other structures: (1) Components and Cladding wind forces; and (2) Main Wind-Force Resisting System wind forces.

Category one: *Components and Cladding* wind loads are applied to individual structural elements that receive wind force directly, such as a fastener, a stud, a roof truss, a rafter or a sheet of structural sheathing. Figures 2.1a and 2.1b illustrate a few examples. A stud receives wind over an area defined by its height and by its spacing. Thus, an 8 foot high stud spaced at 16 inches on center receives wind based on an area A= 8' * (16/12)' = 10.67 square feet. A truss spanning 32 feet and spaced 24 inches on center receives wind over a projected area A=32' * 2' = 64 square feet, while a sheet of plywood receives wind over an area A = 8' * 4' = 32 square feet. It becomes apparent that all of these areas are very small compared to wind against an entire elevation 8 feet tall by 60 feet long, where A = 8' * 60' = 480 square feet.

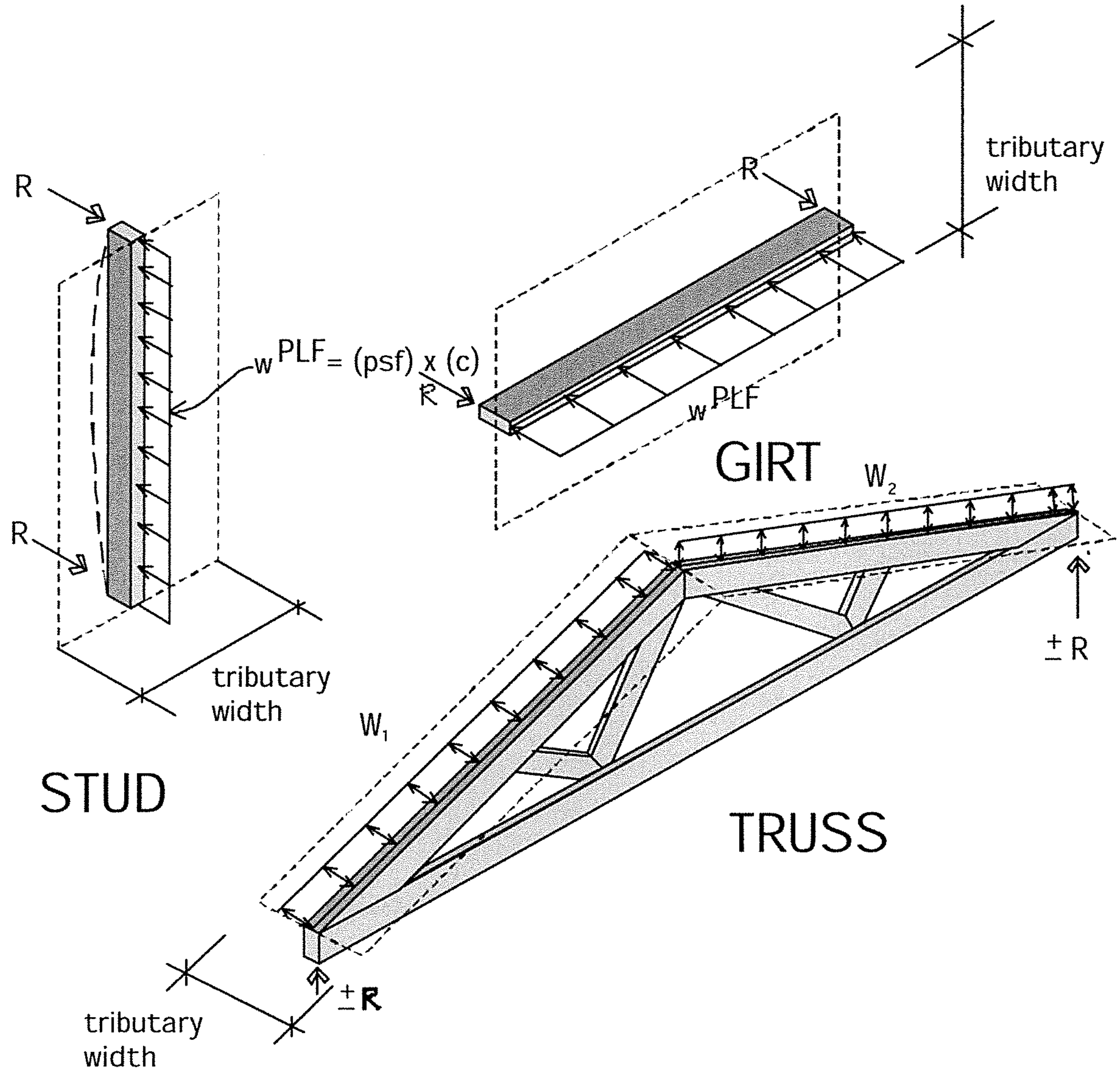
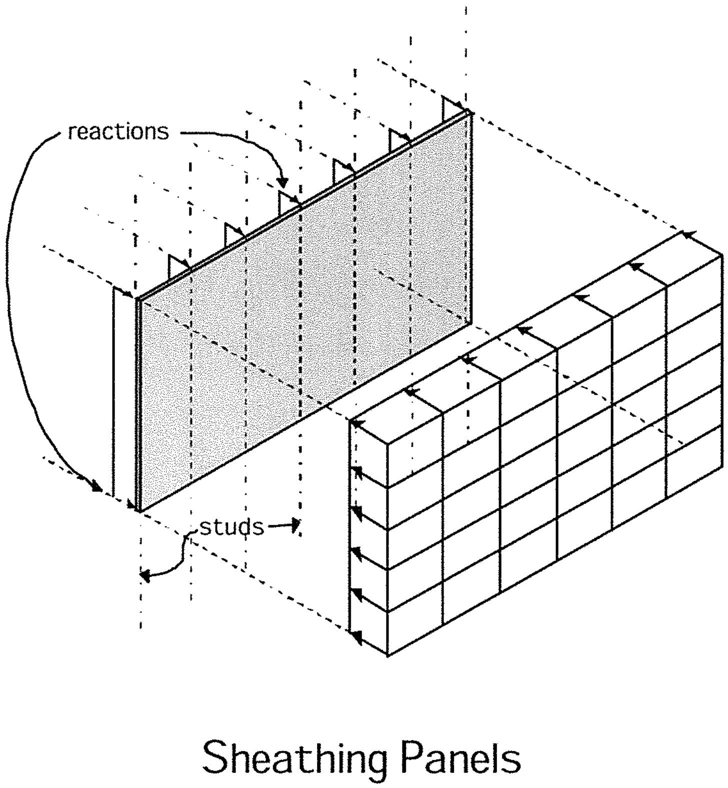


Figure 2.1a. Components that receive wind direct

The probability of a larger wind force hitting a small-localized area is greater than over a very large area. Thus, components are designed for higher localized wind forces, than the building as a total. The same logic applies to wind gusts hitting these same elements. Also, ridges and eaves receive higher wind pressures (positive or negative) than the main part of the roof, and this is reflected in all the building codes. **Note**: Negative pressure is many times referred to as "suction", and both terms will be used interchangeably in this Manual.

Technically, a negative pressure acts away from a surface, while a positive pressure acts against a surface.



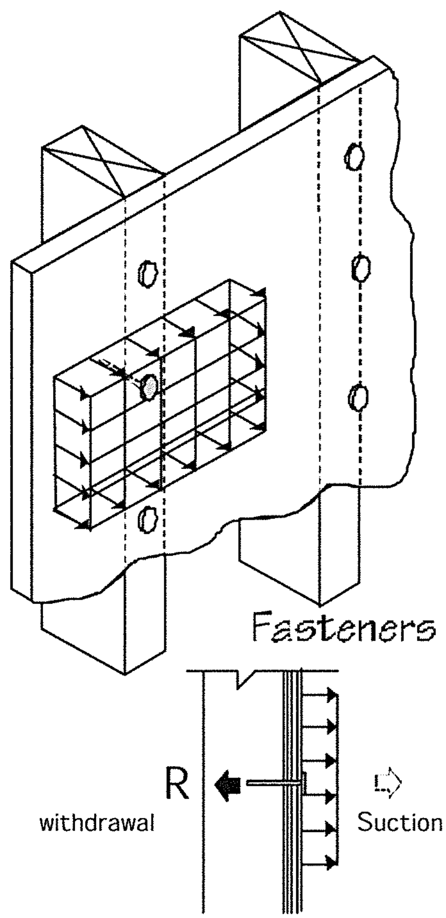
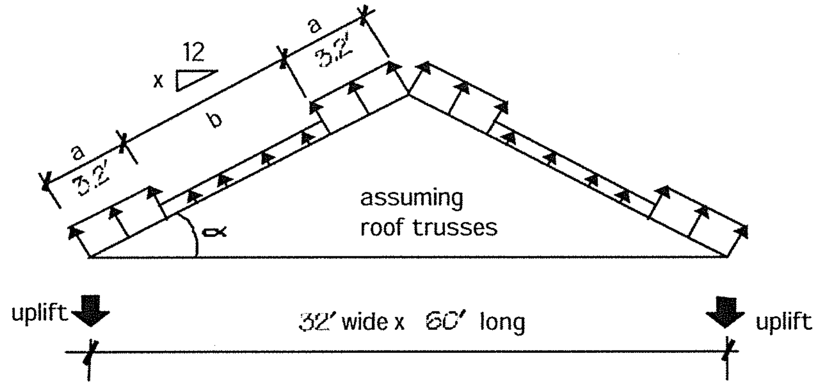


Figure 2.1b. Additional components that receive wind directly

The slope of the roof influences the magnitude of the uplift force for design. Using *Components and Cladding* wind pressures (both positive and negative) for a 90-mph wind speed produces the wind uplift comparison shown in Table 2.1. Eave and ridge locations, defined by a dimension “a”, receive a higher negative pressure or suction, and the central parts of the roof surfaces, defined by the dimension “b” receive smaller negative pressures.



x	3		4		5		6		7*		8*		9*		10*		12*	
α	14.0°		18.4°		22.6°		22.6°		30.3°		33.7°		36.9°		39.8°		45°	
	a	b	a	b	a	b	a	b	a	b	a	b	a	b	a	b	a	b
70 mph	24.5	14.7	24.8	14.9	25.2	15.1	25.2	15.1	18.6	16.4	18.8	16.5	19.0	16.7	19.0	16.7	19.5	17.1
80 mph	32.2	19.3	32.4	19.4	32.9	19.7	32.9	19.7	24.3	21.3	24.6	21.6	24.9	21.9	24.9	21.9	25.4	22.3
90 mph	40.5	24.3	41.0	24.6	41.6	25.0	41.6	25.0	30.9	27.1	31.2	27.4	31.5	27.7	31.5	27.7	32.2	28.3
100 mph	50.2	30.1	50.6	30.4	51.3	30.8	51.3	30.8	38.0	33.4	38.4	33.8	38.9	34.2	38.9	34.2	39.8	34.9

x = 3 to 6	x = 7* to 12*
similar range of values	similar range of values
Higher uplift	~ 25% Lower uplift

* indicates code required pressure value which is another load case, but not of interest to the issue of uplift

Table 2.1. Component and Cladding Uplift on Roof Trusses (in psf)

It is clear for a typical floor plan of 32' x 60' that roof slope influences uplift design suctions in two basic ranges: from a 3 in 12 to a 6 in 12 slope. A 25% decrease in uplift design suction occurs for slopes between 7 in 12 to 12 in 12. The eave and ridge locations receive higher wind suction values for an "a" distance that is calculated from a formula based on the building dimensions. Overhangs would be part of the "a" dimension and part of this higher design suction. This particular roof slope influence is only applicable to the design uplift suctions and should not be considered applicable to the lateral design wind pressures (both positive and negative) for the *main wind-force resistance system*.

Category two: *Main Wind-Force Resistance System* (MWFRS) wind loads are applied to elements of the building that do not receive wind force directly, but participate in the lateral load resistance stability (overturning and sliding) of the building. These elements receive wind load from structural floor and roof planes called diaphragms. Examples in Category two include plywood or OSB board sheathing (commonly referred to in the building codes as **structural panels**) used as shear walls, which lie in planes parallel to the wind direction. See Figure 3 in Chapter 5 of the *WMM*.

> **Note**: The term **Structural panels** will be used interchangeably with plywood or OSB sheathing.

The tension chain issues of **uplift resistance** will be discussed in this Chapter. The illustrations and text of Chapters 4 and 7 of the *WMM* will be referenced. The stability issues of **Overturning and Sliding** will follow in Chapter 3. The illustrations and text of Chapters 5 and 7 of the *WMM* will be referenced.

The Tension Chain – Structural Planning Basics

The superstructure of a house must be thought of in terms of planes. There are roof, wall and floor planes, as shown in Figure 2.2.

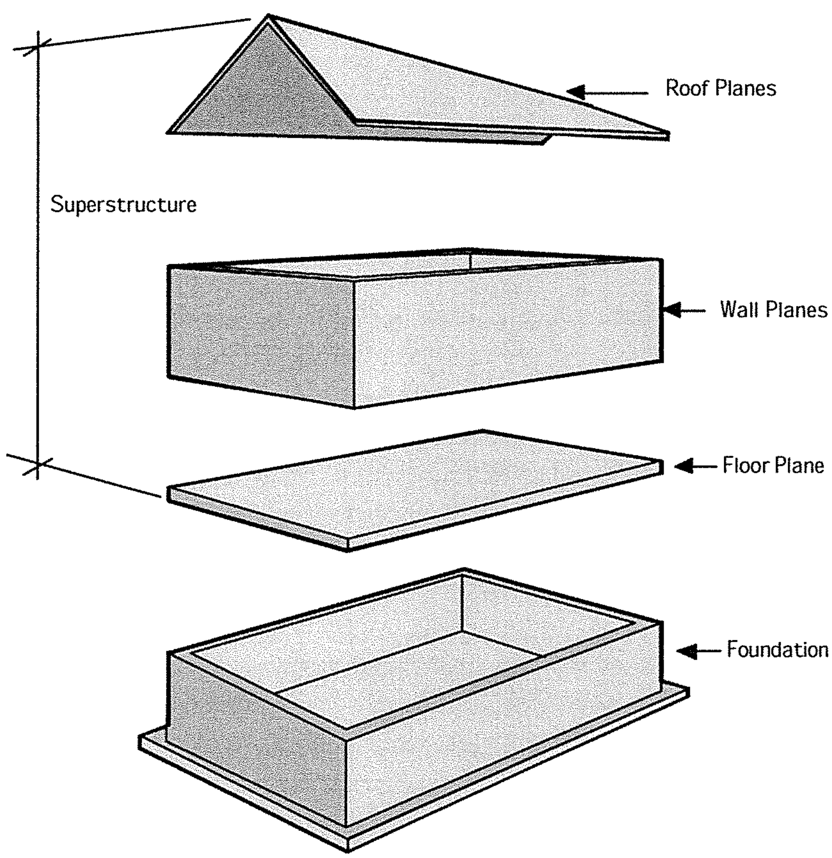


Figure 2.2. Basic Building Planes

Generally, the exterior wall planes are the major part of the tension chain that link together the roof and foundation. The wood frame of the superstructure provides only a small amount of dead load resistance to the wind uplift, thus the weight of the foundation is the most important "ballast" to resist wind uplift on the superstructure. Essentially, the wall's connection to the foundation is the final link in the tension chain that must hold down the superstructure against uplift.

Walls Sheathed with Insulation Board (non-structural) in areas without Openings

Figure 2.3a illustrates a section of a typical exterior loadbearing wall with the entire skeleton framing elements in the tension chain separated.

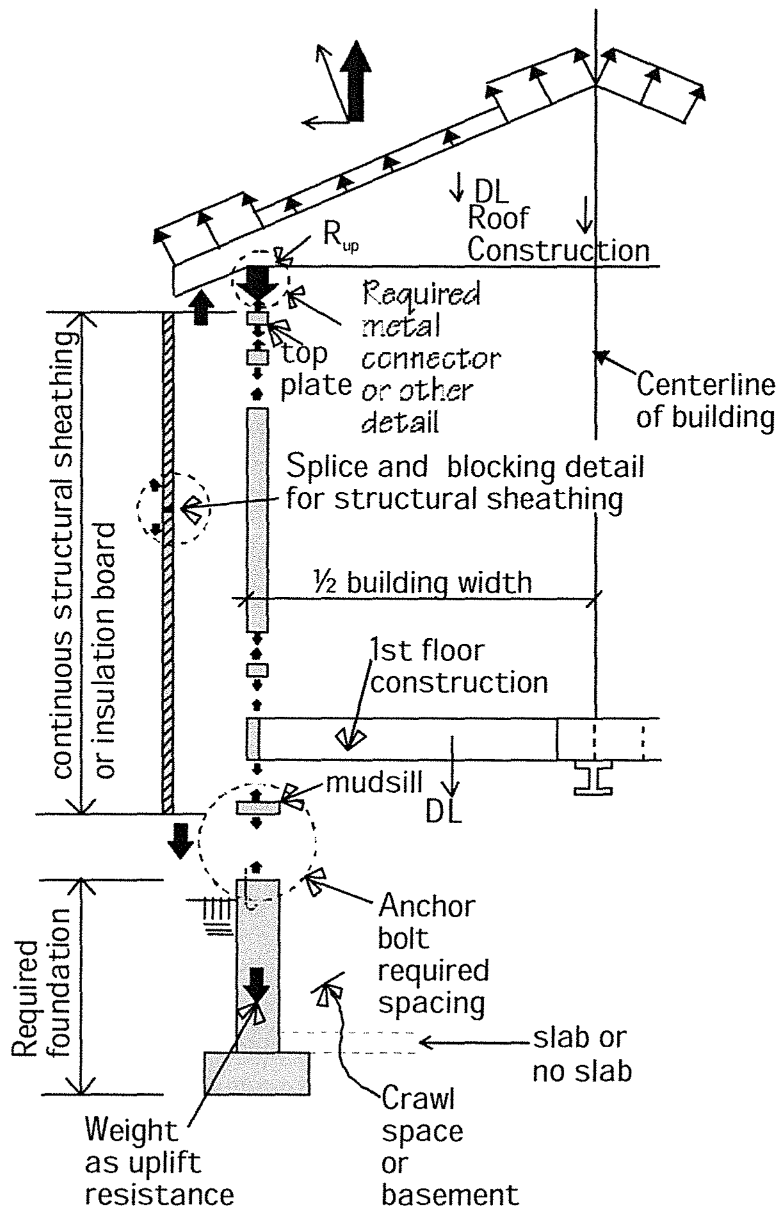


Figure 2.3a The Links of the Tension Chain - Ext. Wall

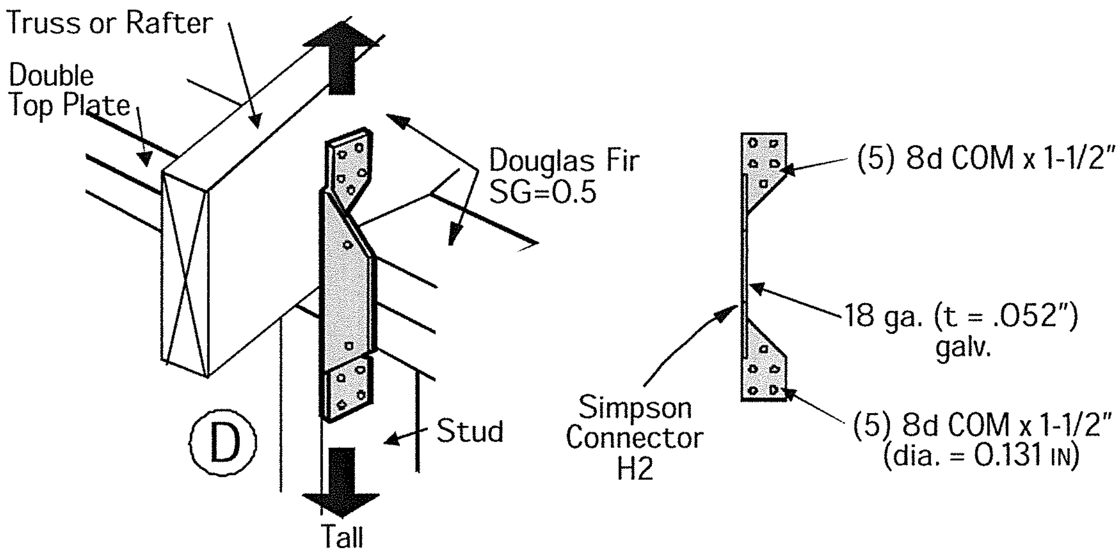
The sheathing is typically non-structural insulation board. The uplift from the *Components and Cladding* wind category is shown across the roof. All the uplift arrows between elements are indicated. Since the tension chain of resistance is dependent on properly installed metal connectors, when non-structural insulation board is used, a one or two story residence will require connections at three basic locations:

1. The roof to the double top plate and stud;
2. the wall studs to the bottom plate;
    a. include the influence of floor plane band boards for one or two stories;
3. the wall to the foundation;
    a. include the influence of floor plane band boards for one or two stories;
    b. walls that bear directly on the foundation, such as garages.

These three connections will be discussed separately and in considerable detail.

Warning: The use of metal connectors depends on filling all the holes in the connector with the manufacturer's required size, length and type of nail. If properly installed the published rated structural capacity of the connector can be relied upon. Too often an inadequate nail size or type is used, or an insufficient number of nails are installed. This practice results in connectors that cannot provide the required structural capacity, and thus creates a new weak link in the tension chain. Contractors should always keep the manufacturer's catalog handy for reference. Use of connectors for conditions not specifically addressed in the manufacturer's catalog are prohibited.

Figure 2.3b clarifies the procedure used by one manufacturer (Simpson Strong-Tie) to arrive at their catalog [2.1and 2.2] of allowable values in **tension uplift**. Assume application of connector "**D**" to resist an uplift value of 330 lbs. between the roof rafter and (bypassing the double top plate) the stud wall. The allowable tension capacity of this connector is determined mathematically from the 1997 Edition of the NDS for Wood Construction [2.3], referenced by all three model codes, and a load test to find the ultimate capacity at failure. The ultimate test value is divided by a factor of safety of 3.0 to arrive at a comparable allowable capacity for the entire group of (5) 8-d nails. The smaller of: (1) the equation derived value; and (2) the reduced ultimate value establishes the allowable capacity. The allowable capacity of 347 lbs. is greater than the required tension capacity and therefore this connector is usable. The connection would not provide the necessary tension resistance if: (1) less than 5 nails were installed to the rafter and to the stud; (2) nails with smaller shear capacity were installed; (3) the connector was less than 18 gage; or (4) the species of wood used had a different specific gravity than 0.5.



Allowable Capacity found by:

(1) NDS Nail Spec---Capacity/nail in shear = 93.9 LB

(a) modify for load duration (wind = 1.33 or 1.6)

(b) modify for penetration (gage of metal)

(c) modify for wood species density

allowable capacity = (1) x (a) x (b) x number of nails

$$\underbrace{93.9 \times 1.33 \times 0.92}_{115^{\text{LB/nail}}} \times 5 = 575^{\text{LB}}$$

(2) load test

ultimate capacity = 1040 lbs.
for Factor of Safety = 3
allowable capacity per connector = $\frac{1040}{3} = 347^{\text{LB}}$

(3) smaller value controls between (1) and (2)

Therefore: $\boxed{\text{allowable uplift capacity} = 347^{\text{LB}}}$ (335 LB in Simpson Table)

Figure 2.3b Uplift Capacity - Connector Calculations

The Roof to the Double Top Plate and Stud

The roof may be framed with trusses or rafters at a particular spacing. The spacing of trusses is typically 24”on center and rafters are usually spaced 16” or 24” on center. Either of these roof elements will receive the uplift wind force and are the first elements to be tied down. The resistance chain starts with the roof element and it must be connected to the double top plate of the wall with metal connectors.

Selection of the appropriate metal connector depends on the spacing of the wall studs and their alignment with the roof framing. Figure 2.4 shows the three stud wall construction options.

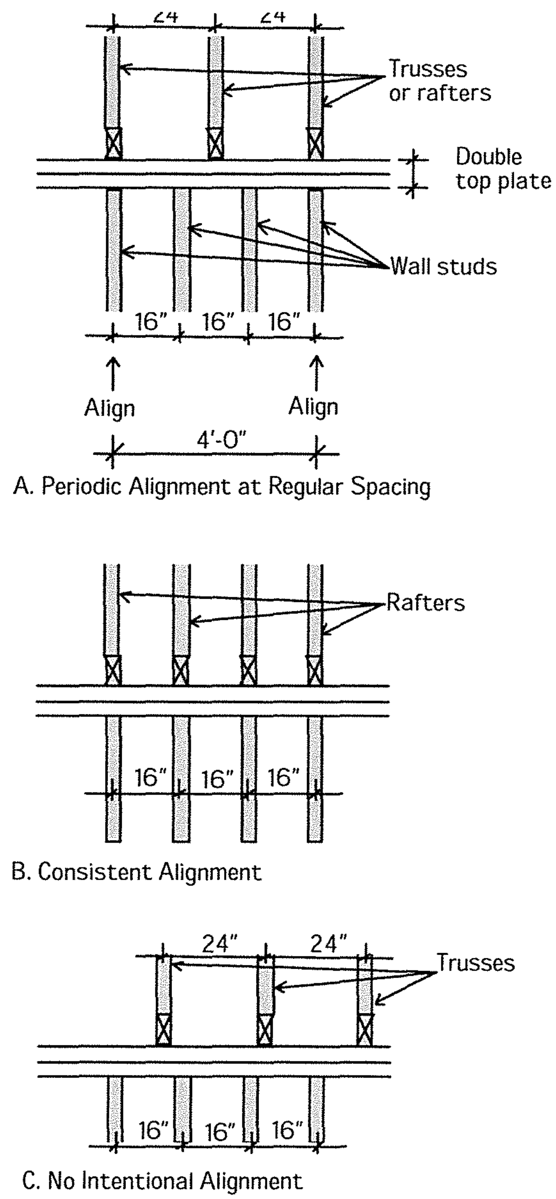


Figure 2.4 Elevations of Stud Wall/Construction Options

The top illustration "A" is the most common condition, where alignment has been planned into the framing design. Since trusses are common today, and usually spaced at 24 " on center, every fourth stud will align with the roof trusses, creating a 4'-0" regular continuous path for tension connections. Situation "B", where the roof and wall framing is all at 16" on center, is very rare today. This pattern uses more lumber and the framing layout takes more time. Situation "C", where there is no intentional alignment in the framing design and the roof framing and wall studs only occasionally align, is the most common house framing. This saves a contractor time in layout but complicates the tension chain of resistance connections.

The nails used for the tension chain must be loaded "laterally" and avoid the use of nails in "withdrawal" or " toe-nailed" as defined in Chapter 2 of the *WMM.* Illustration of a variety of metal connectors used to attach the trusses or dimension lumber rafters to the stud wall is shown in Figure 2.5, as they apply to the condition of alignment of studs and roof framing. It should be noted that when alignment exists, the double top plate could be avoided in the

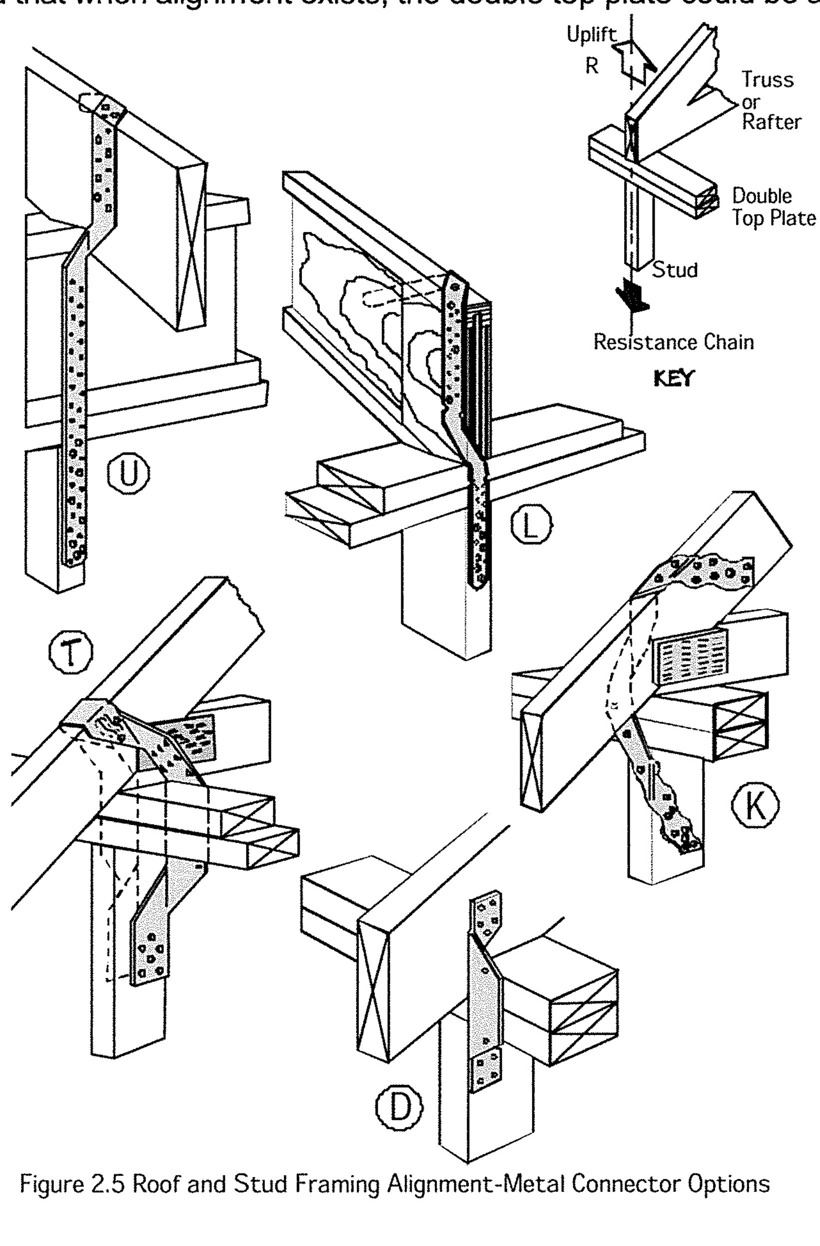


Figure 2.5 Roof and Stud Framing Alignment-Metal Connector Options

tension chain. A direct connection between a roof truss or a rafter and the wall stud has the advantage of fewer nails and a tension chain with fewer links. Connector types **D**, **L** and **K** were shown in Chapter 7 of the *WMM* and are repeated here, while types **T** and **U** are more elaborate connectors.

Non-alignment of wall and roof framing members involves an extra connector and therefore means more material and labor cost for the contractor. Illustration of a variety of metal connectors used to attach the trusses or dimension lumber rafters to the double top plate is shown in Figure 2.6a, as they apply to the condition of non-alignment of studs and roof framing. Connectors **G**, **H** and **J** were shown in Chapter 7 of the *WMM* and are repeated here, while connector **V** is another option. It is important that the connector is nailed to both plates of the double top plate. Thus, this connection becomes the first link in the tension chain of resistance.

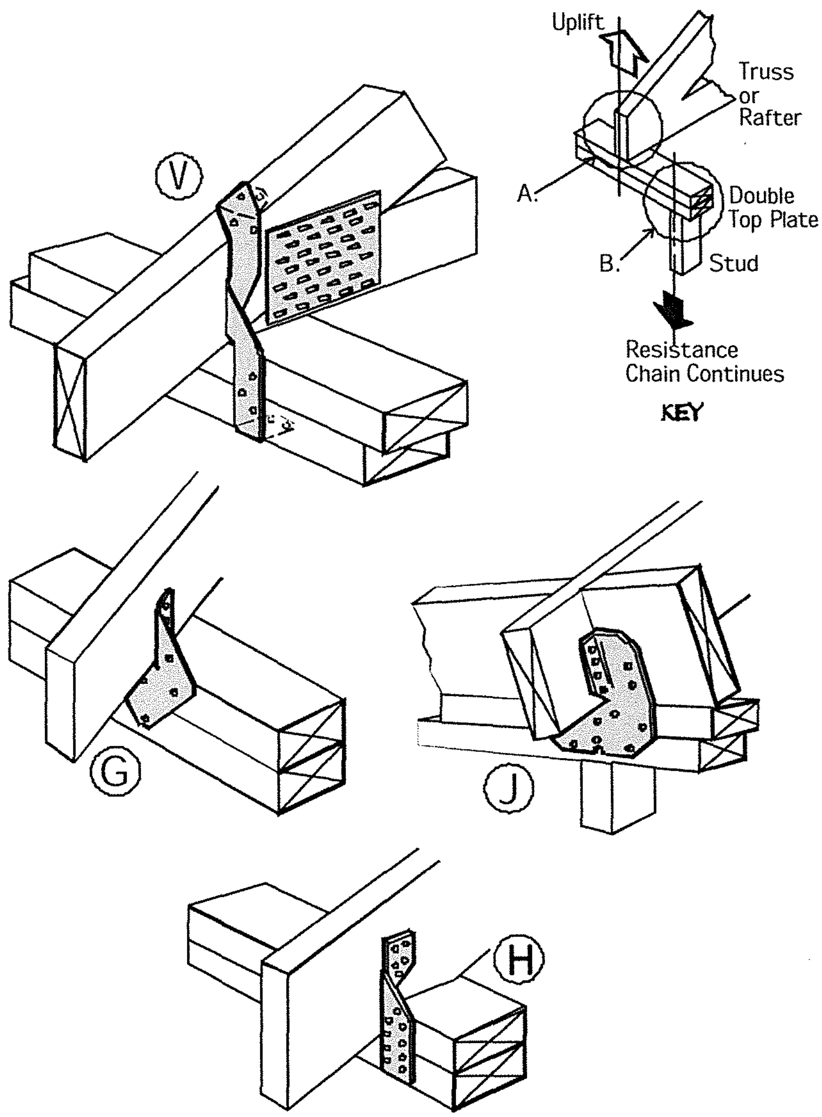


Figure 2.6a Truss/Rafter to Double Top Plate Connectors

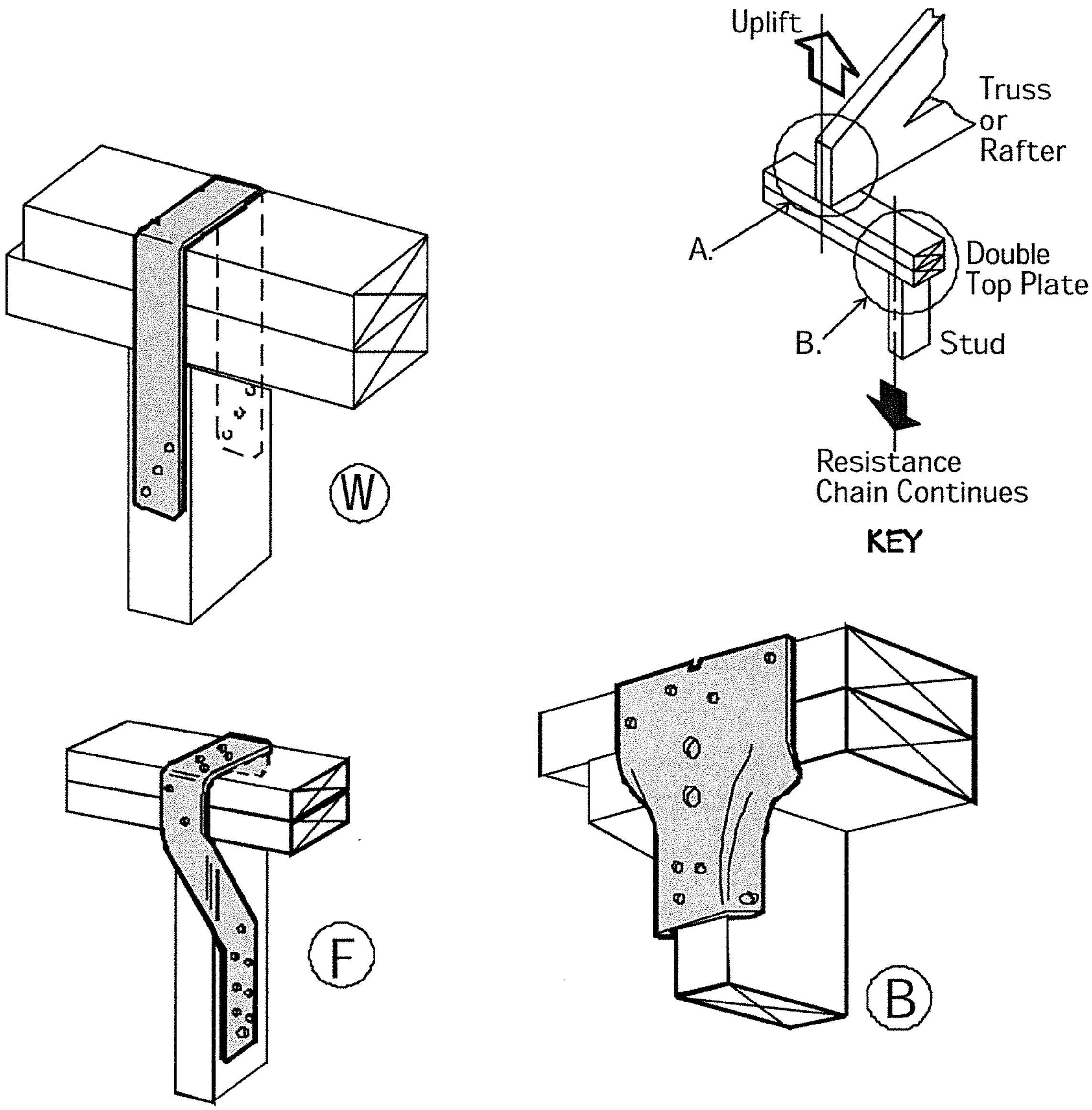


Figure 2.6b Double Top Plate to Stud Connections

Illustrations of metal connectors used to attach the double top plate to the stud are shown in Figure 2.6b. Connectors **B** and **F** were shown in Chapter 7 of the *WMM* and are repeated here, while connector **W** is another option. Each of these connectors adds another link to the tension chain of resistance, and now has the tension down the length of the stud.

The Wall Stud to the Bottom Plate and then to the Foundation

This connection in the tension chain of resistance is dependent on what material lies below the bottom plate. If a one story residence is used as a starting point, there may be:

1. A first floor sub-floor, bandboard and mudsill, followed by a crawl space or basement foundation;
2. a concrete or masonry foundation for a garage slab-on-grade, or;
3. a concrete or masonry foundation for a tri-level home, where the basement slab is built a half level into the ground.

Each of these three possible conditions will now be discussed separately:

When **condition #1** is confronted, the tension chain usually requires two connections. The first is the stud to the bandboard, bypassing the bottom plate. Illustrations of metal connectors used to attach the stud to the bandboard are shown in Figure 2.7a. Connectors **E, Q** and **R** were shown in Chapter 7 of the *WMM* and are repeated here. Connector **E** merely passes the tension chain on to the bandboard, requiring connector **R** to be the link between the bandboard and the mudsill. From there the anchor bolts transfer the tension into the concrete to complete the chain of resistance. Connector **Q** and **R** together accomplish the same thing. Connector **X** is a continuous strap that ties the stud to the band board and is also imbedded into the cast-in-place concrete. This single tension chain link accomplishes several connections in one, but has the disadvantage of needing to be properly set in the concrete prior to the wall placement. One must know where the stud will be located for this connection to work and requires extreme care in placement. Connector **Y** is another type of strap that must be set in the concrete and wrapped around the bandboard. Again care in placement and alignment is required to make this connector function properly. This connector, in combination with either connector **E** or **Q**, which connects the bandboard to the studs, completes the tension chain of resistance. These later two options (**Y** and **X**) provide considerable tension strength and should be considered when wind uplift values become high. They are also more expensive options.

When **condition #2 or #3** is confronted, the easiest connector to install is type **Z,** illustrated in Figure 2.7b. Here, nails attach the connector to the stud, and an expansion anchor or epoxy anchor is drilled into the concrete or grouted masonry foundation (passing through the bottom plate/mudsill) to complete the tension chain of resistance. This approach is easier than anchor bolts, which need to be straight and properly located to align with the bolt hole in the connector. Connector **X** in Figure 2.7b is similar to connector **X** in Figure 2.7a, except it is shorter in length.

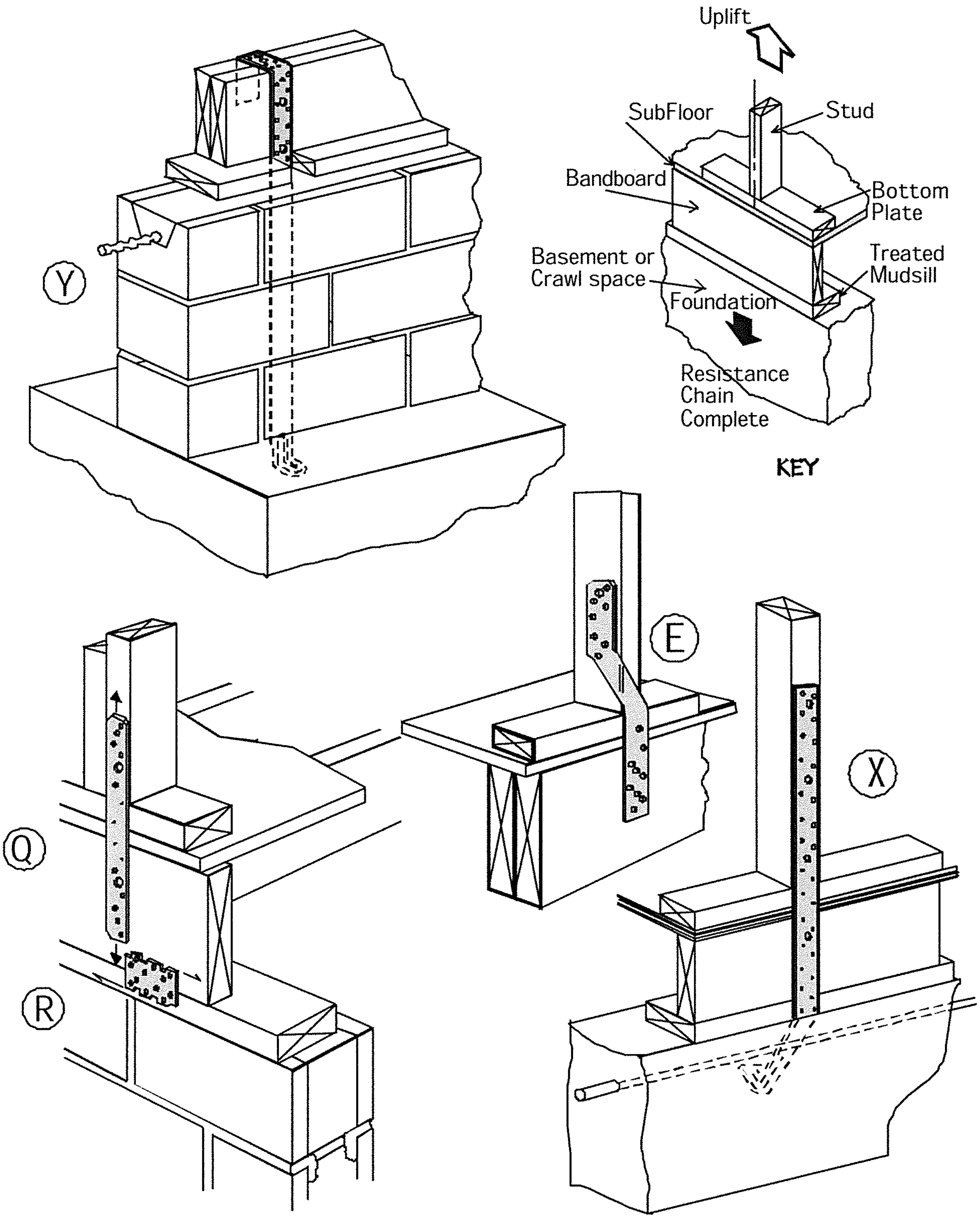


Figure 2.7a- Stud to First Floor Connections

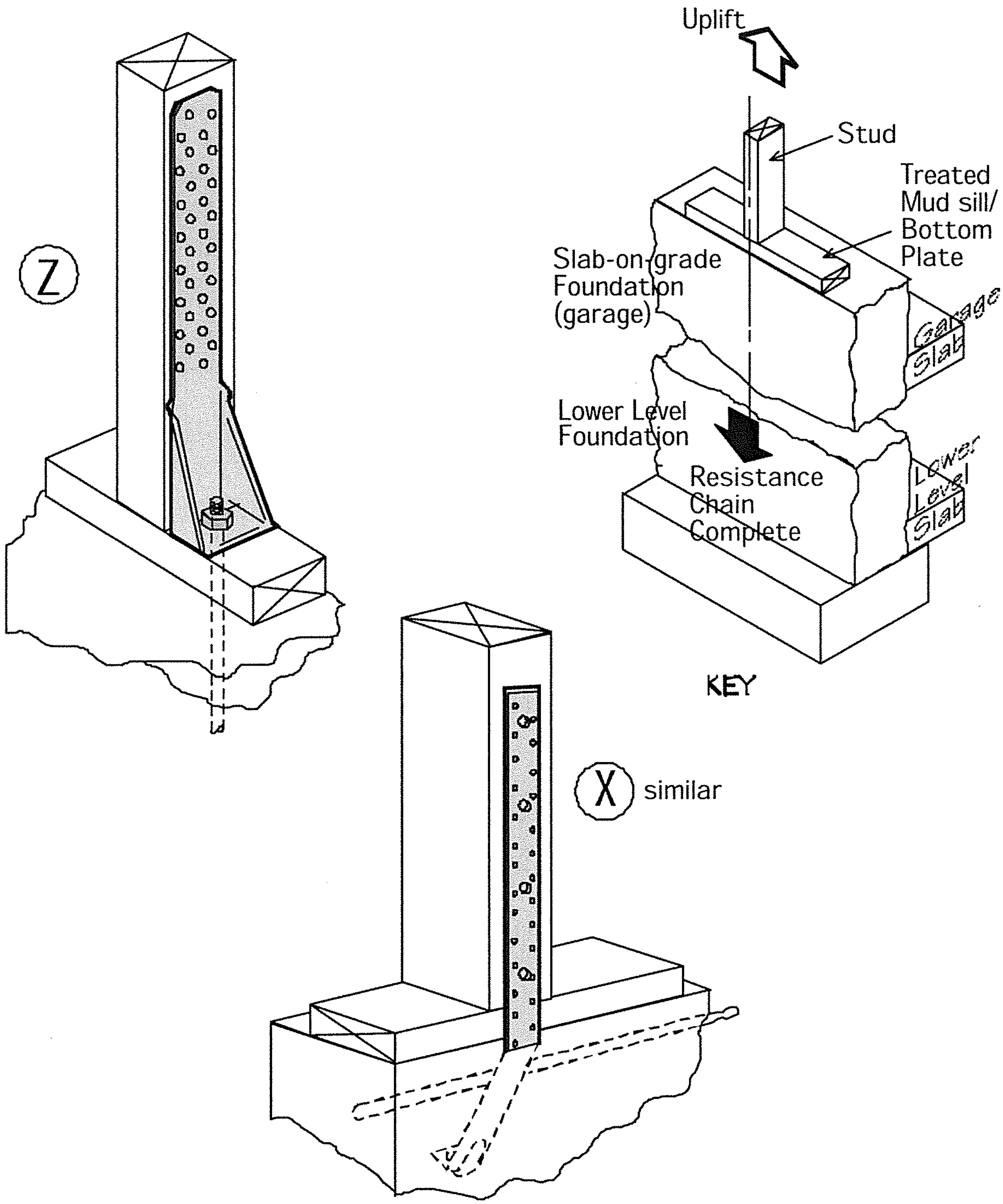


Figure 2.7b Stud to Foundation Connections

Second Floor Plane with Stud Walls Above and Below

When a two story portion of residence exists, the alignment of the studs in the walls above and below will influence the selection of a metal connector to continue the tension chain. Figure 2.8a illustrates two connector options when the studs above and below the second floor align. Connector type **P** is a continuous strap that nails all the second floor framing components between the studs, as well as the studs themselves. This is an excellent approach that is reasonably economical and easy to install. Connector **N** actually connects only the stud above and below the second floor framing components. This is another reasonable option.

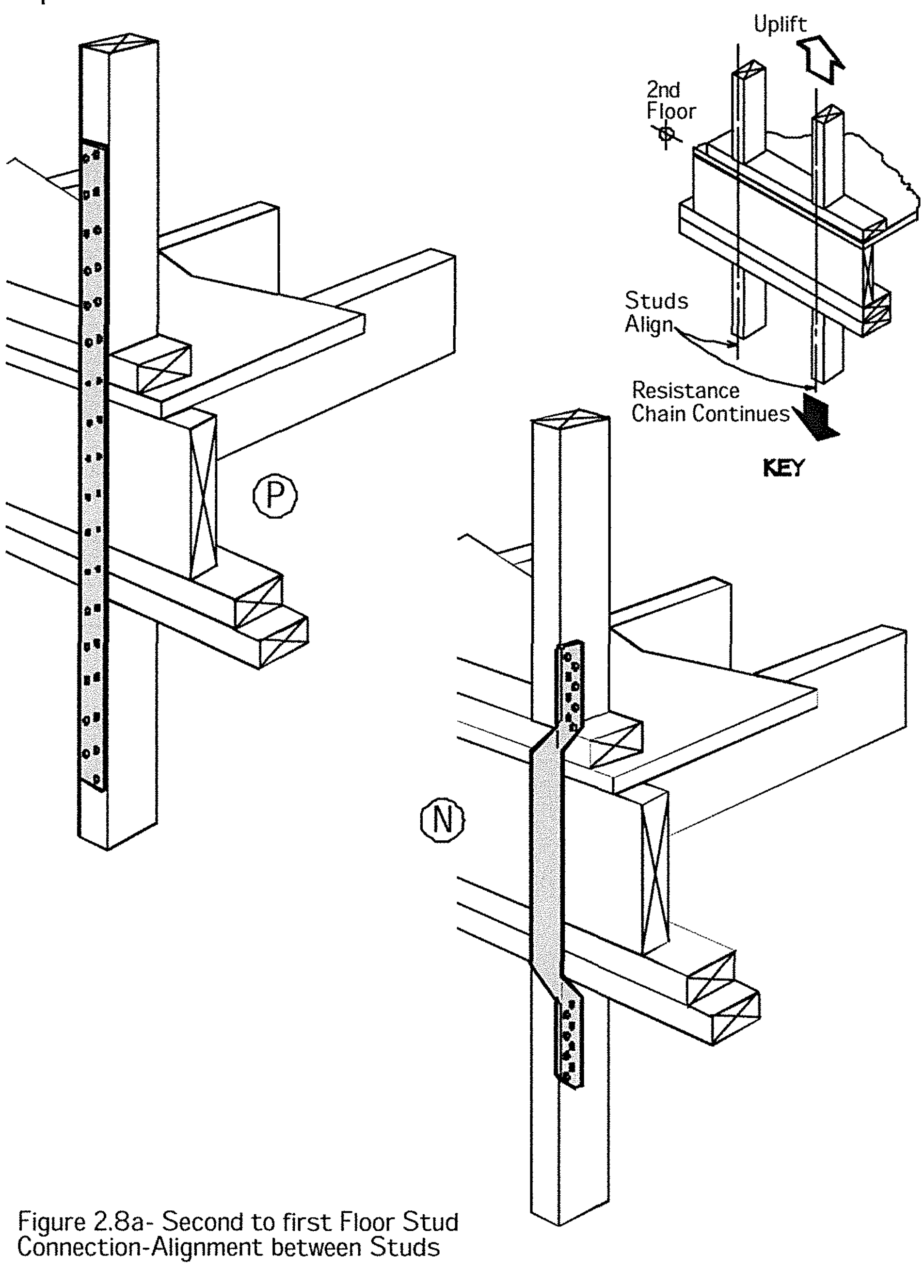


Figure 2.8a- Second to first Floor Stud Connection-Alignment between Studs

Figure 2.8b illustrates stud non-alignment above and below the second floor plane. Two connectors are now required and the splitting strength of the bandboard is now a factor. This will usually require a #1 grade Douglas Fir or Southern Pine bandboard for maximum - perpendicular to grain - tensile strength.

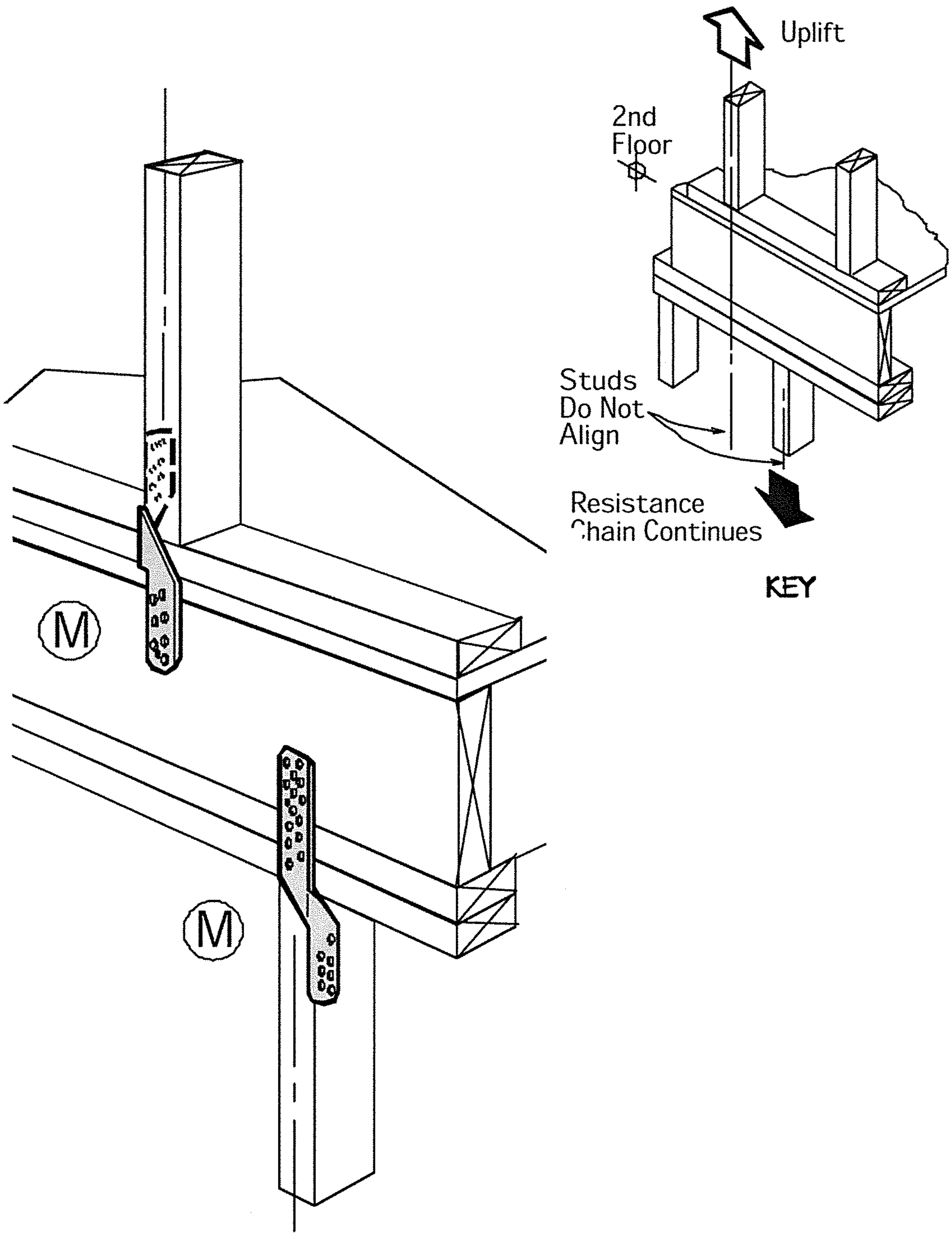


Figure 2.8b - Second to First Floor Stud Connection
Non-Alignment between Studs

Note: It is possible to double connectors between the roof and the wall when the required uplift capacity exceeds the allowable capacity of a single connector. Certain guidelines must be followed [2.4] to avoid splitting the rafter or truss. These guidelines are illustrated in Figure 2.8c. Case "A" would not allow doubling connectors, since splitting of the 1-1/2" wide truss or rafter is likely. Case "B" separates the two connectors by the width of the double top plate to avoid splitting when 1-1/2" width roof members are used. Case "C" allows doubling connectors but requires the roof member to be at least 2-1/2" wide. This condition is less likely to split the roof member.

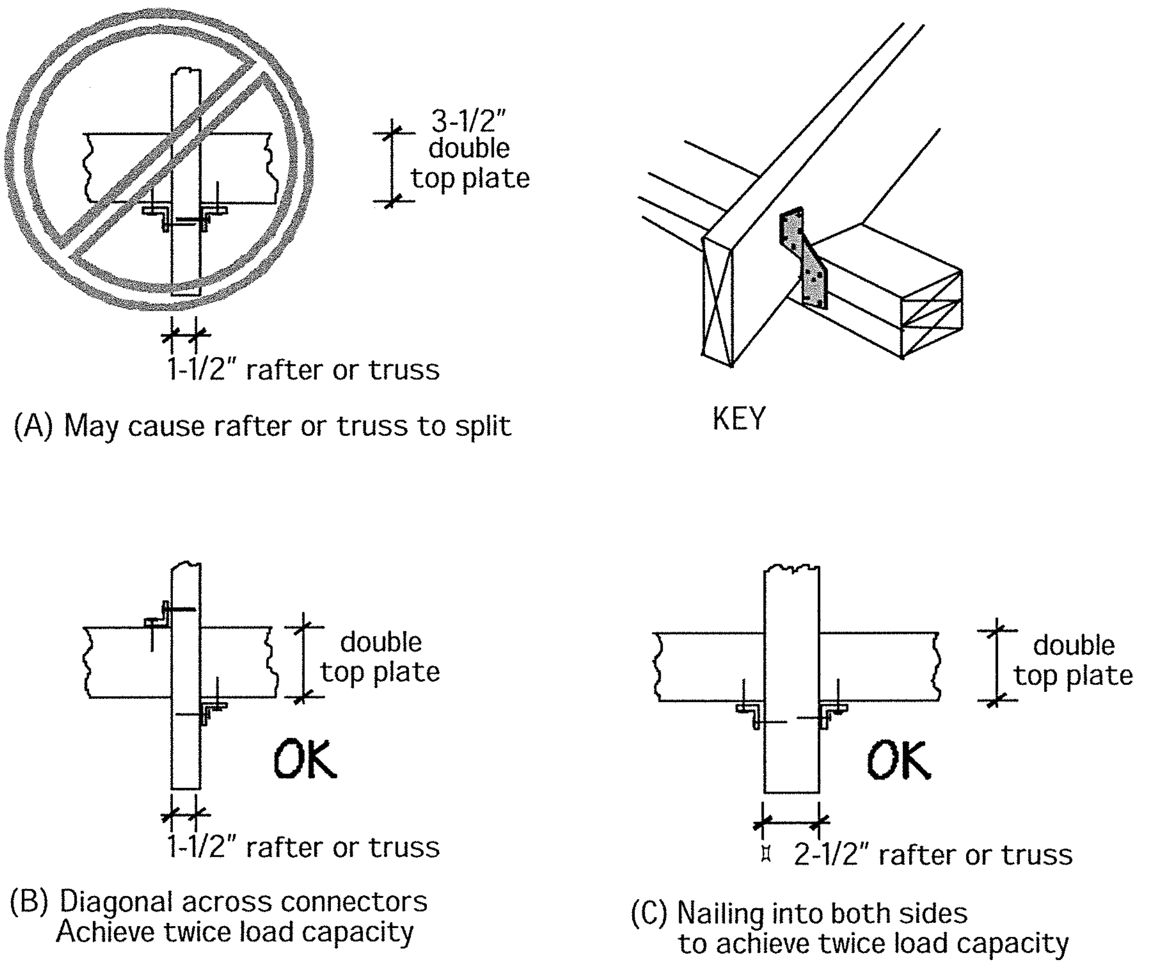


Figure 2.8c - Double Capacity of Connectors (Shown in Plan Views)

Walls Sheathed with Structural Panels in areas without Openings

The discussion has so far involved the skeleton framing of roof and wall with the exterior sheathing being a non-structural insulation board. There are two other choices:

1. Structural panels (plywood or OSB) as sheathing;
2. insulation board with calculable structural capacity.

Both of these options will now be discussed in detail.

The Structural Sheathing of the Wall Framing Elements

When a residence is totally sheathed with structural panels in a manner to assure vertical continuity, the need for metal connectors is significantly reduced. There will only be a need for a metal connector at the roof truss or rafter to the wall and the wall to the foundation. This is a very economical approach to the structural requirements of the tension chain, with only a minimal reduction in the insulation value of the wall. This issue was discussed in Chapter 1.

The essential criteria is for the structural panels to become a continuous plane from the top of the double top plate, across the second and first floor construction and down to the bottom of the mud sill, as shown in Figure 2.9. This figure has been slightly modified from Figure 10 of Chapter 7 in the *WMM*.

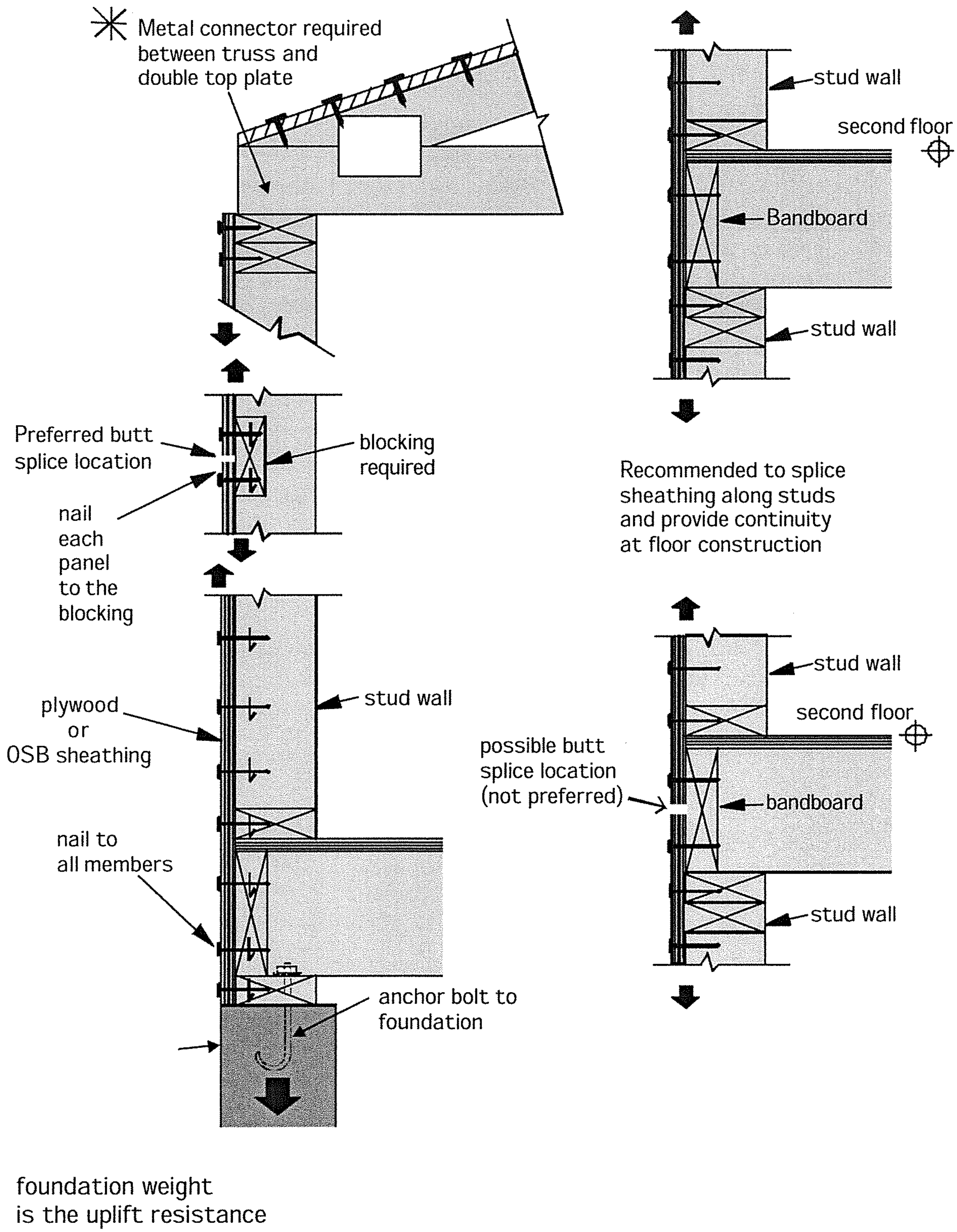


Figure 2.9- Total Use of Plywood or OSB Board to Resist Uplift

The first modification is that the 2x4 wood blocking at a typical splice has been rotated 90 degrees, so that the 3-1/2" width of the 2x4 is flat against the sheathing. This is done for several positive reasons:

(1) To provide more room for nailing, avoiding nails on a slant and splits in the wood;
(2) to provide for batt insulation to pass the blocking and merely be compressed in that region; and
(3) to provide room for electrical wiring to pass vertically through the wall.

The second modification deals with splices at the bandboard of the second floor level. Although it is structurally feasible for a splice to be made at mid-height of the bandboard, it is even stronger to place the splice within the length of the studs above and/or below the floor construction. This avoids the potential of splitting the bandboard perpendicular to the horizontal direction of its grain, and places the splice along the studs, which have their grain parallel to the uplift force.

Connection of the Roof Truss/Rafter to the Structural Wall Plane

The structural sheathing starts from the top of the double top plate and runs continuous for the full length of the wall. Connections between roof truss/rafter to the wall no longer depend on the position of the studs in relation to the roof framing. Thus, the metal connector options of Figures 2.5, 2.6a and 2.6b are all possible and the selection merely depends on the magnitude of the uplift force (R_{UP}) found in Chapter 4. Additional style of metal connectors for this purpose can be found on Figure 5 of Chapter 7 in the *WMM*. Every roof truss/rafter must be metal connected to the structural sheathed wall.

Connection of the Structural Wall Plane to the Foundation

The transfer of uplift force from the sheathing to the foundation is rather uniformly distributed along the mudsill by closely spaced nails into the mudsill. The mudsill is the means of transfer of this uniform uplift to the anchor bolts. The anchor bolts in turn have to transfer a concentrated uplift force into the concrete or grout of the foundation, which is the final link in the chain of resistance. The wind uplift is thus resisted by the weight of the foundation.

Two mudsill conditions that exist are:

1) When the floor joists bear on the wall subject to uplift; or
2) when the floor joists run parallel to the wall subject to uplift.

When the floor joists bear on the mudsill more dead load is present to help reduce the uplift on the mudsill. The joists also help to stiffen the mudsill and avoid bending and shear. The only failure mode likely is that of direct shear of the mudsill at the small round washer. This is illustrated in Figure 19c of Chapter 4 in the *WMM*.

When the floor construction runs parallel to the mudsill, the mudsill has to resist bending, shear and pullout. The weakest stress, usually shear, will result in the failure mode referenced in Figure 21b of Chapter 4 in the *WMM*. Square washers are recommended to reduce the tendency for the mudsill to shear and to spread the bearing stress over a larger area. It is recommended that 3 x 3 x 9/64 square washers be used with the anchor bolts at a spacing that satisfies both:

(1) The requirement of backfill pressure against a basement wall (Table 3 in Chapter 4 of the *WMM*); or
(2) the required wind uplift on the mudsill and anchor bolts.

There are somewhat problematic field conditions that can be alleviated by the use of the square washers that contain a diagonal slot. Figure 2.10 illustrates several anchor bolt conditions that have been seen far too often by this author.

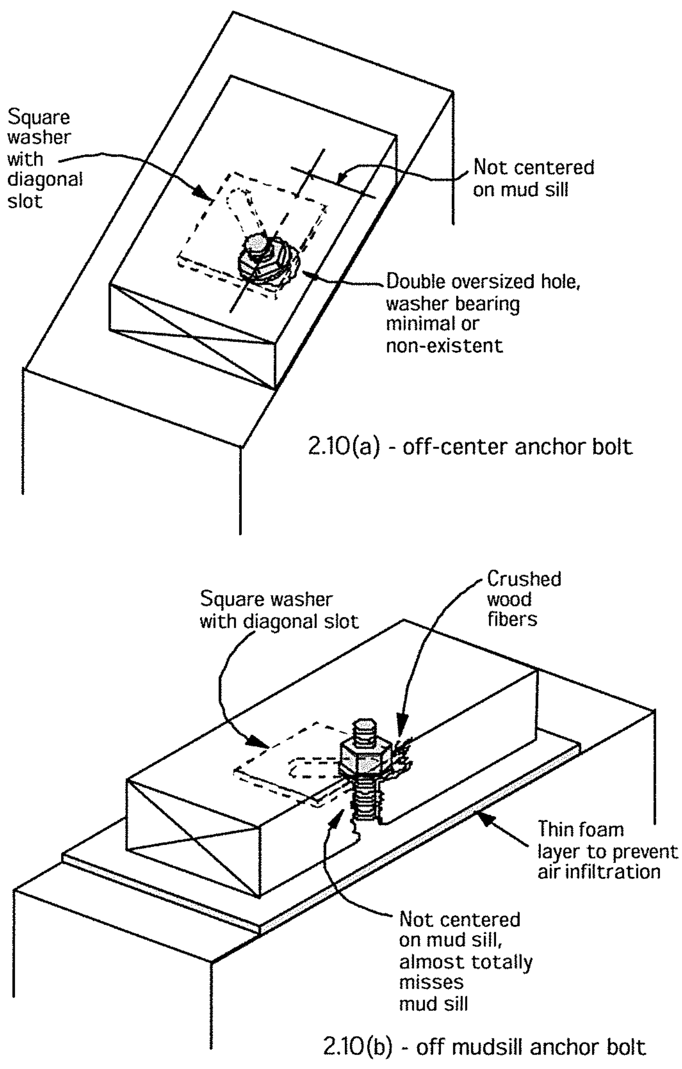


Figure 2.10 - Anchor Bolt Problems

Anchor bolts are many times placed in the concrete so they will not be centered on a mudsill or bottom plate. Figure 2.10a shows an off center anchor bolt in an oversized hole, where the round washer barely develops any bearing against the wood fibers. A square washer with a slotted hole not only adds bearing but covers most of the mudsill and avoids bending of the mudsill. Should the anchor bolt placement actually miss the mudsill or bottom plate, as shown in Figure 2.10b, the situation worsens. Now the washer squashes the wood fibers and will never produce a tight bearing without further damaging the wood. Again, a square washer will reduce the severity of this situation, but in this case it may extend over the edge of the wood. Note the illustration of thin foam between the concrete and the mudsill. When the top of the concrete is rough and uneven, this foam will try to fill the void and prevent air infiltration. Unfortunately, when the roughness is extensive daylight is visible between the wood and concrete.

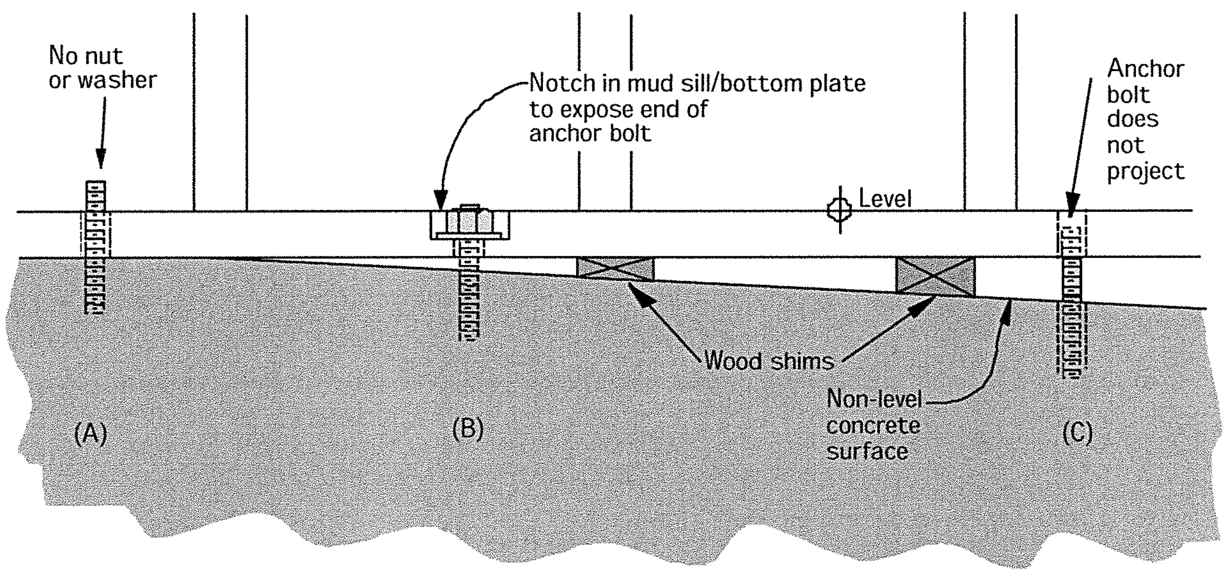


Figure 2.11 - Shims, Non-level Top of Foundation

Figure 2.11 illustrates a sloping (non-level) top of foundation wall where wood shims are used to return the stud wall for a garage or mudsill for a first floor to a straight horizontal plane. This is one of the more common, and most time consuming, conditions to correct. It creates an open link in the tension chain for wind uplift resistance. Plus, this condition is typically hidden behind the drywall and unknown to the owners. Figure 2.11 (C) illustrates that when the shim height is too great the anchor bolt cannot project above the wood to install a washer and nut. Figure 2.11 (B) indicates a notched mudsill or bottom plate to allow placement of the washer and nut over the anchor bolt. Here the shear capacity of the wood is severely reduced and so is the uplift resistance. Figure 2.11 (A) is an unconscionable omission of the nut and washer, even though sufficient anchor bolt length projects above the wood.

Metal straps that act as a belt embedded in the concrete or concrete block grouted cores may substitute for anchor bolts. They have sufficient structural capacity, as long as the strap ends are properly lapped. Figure 9 of Chapter 7 in the *WMM* illustrates these mudsill to foundation anchorage options. It is recommended to start with the anchor bolt-spacing requirements for the anticipated backfill condition and test whether this also satisfies the uplift requirement on the anchor bolt or strap.

Due to typographic errors in the dimensions of the square washer of Figure 9 in Chapter 7 of the *WMM*, and the need to clarify its appearance, it is reproduced here as Figure 2.12a.

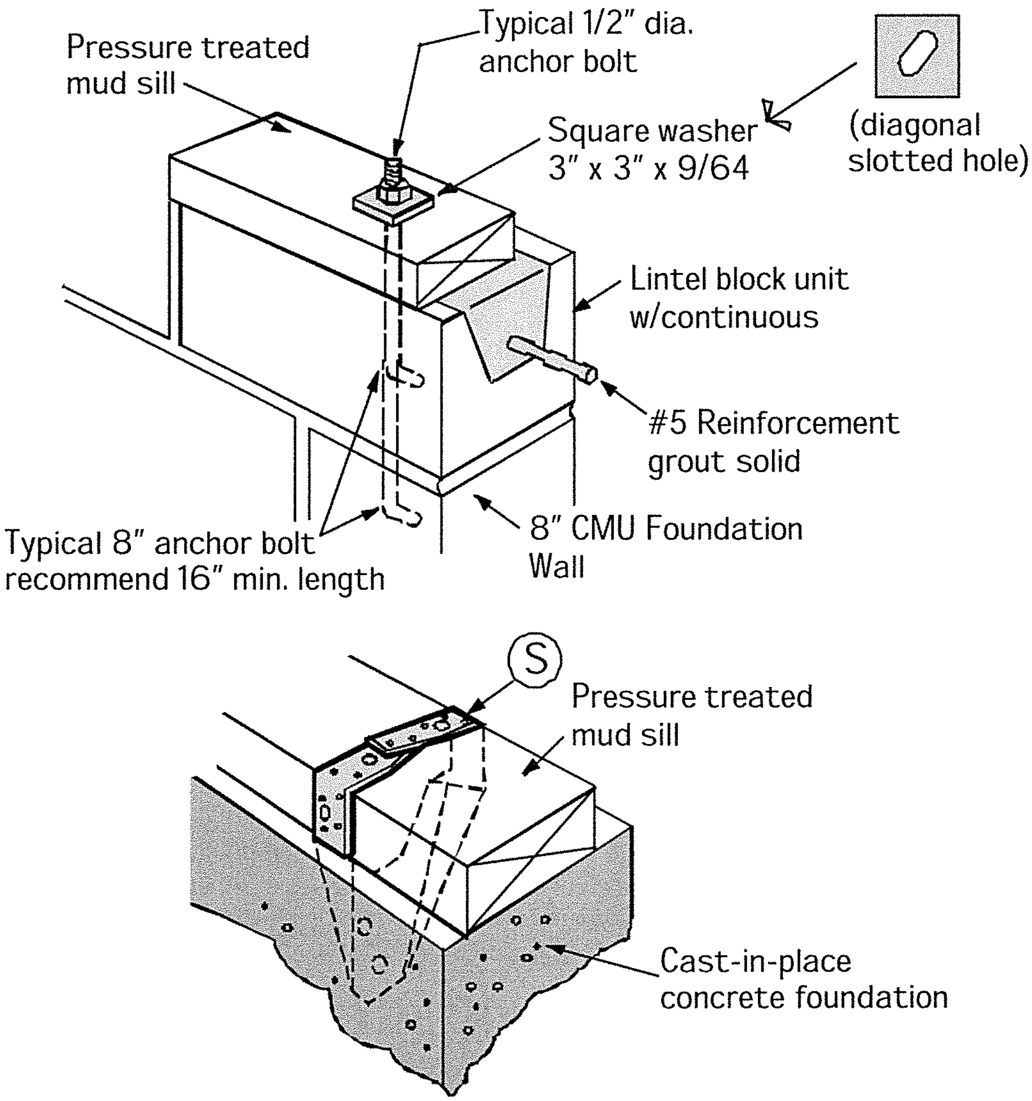


Figure 2.12a - Anchorage to Foundation-Options

Figure 2.12b illustrates an alternate mud sill anchor, using a split-strap connector. This anchor either spreads the straps into a "Y" shape based on the width of the mudsill, and wraps around the mudsill and is nailed to it; or the connector extends out of the hole drilled in the mudsill and the straps are bent down and nailed to it. The former is preferred for cast-in-place concrete, while the later is preferred for use in concrete block cores. **Incorrect use** of this split-strap connector is common, since it is usually bent at 90 degrees to the top of the concrete or block and then wrapped around the mudsill and nailed as illustrated. This misinstallation application reduces its allowable tension capacity and results in significant movement before it will resist any tension uplift.

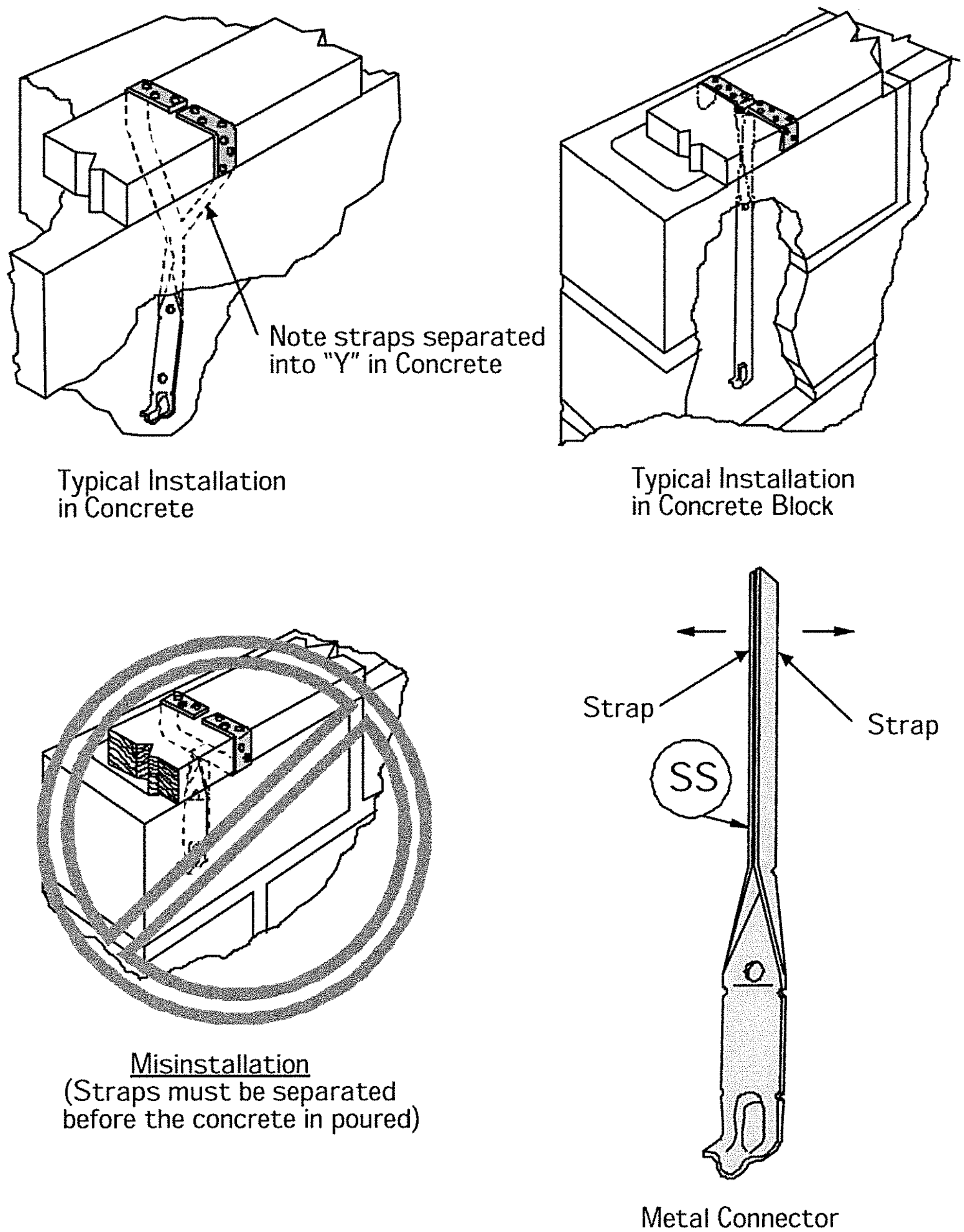


Figure 2.12b - Alternate Mud Sill Anchor

Walls Sheathed with Insulation Board with Calculable Structural Capacity – with or without Openings

Structural insulation board sheathing comes in various thicknesses from 0.078", 0.115", and 0.137", depending on the shear strength required. It comes in sheets 4'x 8' and the outside is covered with foil. The sheets can be lapped or wrapped around corners by bending. Nails or staples are commonly used to attach the sheathing to the wood skeleton. Thus, the entire house is sheathed with this material. Continuity is essential, just as it is for OSB or plywood structural panels. Too often this author has seen it stopped at floor planes and infill pieces inserted along the bandboard. This reduces the system to isolated pieces and destroys the tension chain of resistance. Staples or nails many times miss the studs behind, or are driven with a gun that forces them to gouge into the panel or even penetrate completely through the panel. Obviously the gun pressure must be calibrated for such a thin sheathing. Although the sheathing has its merits, it requires care in installation.

Exterior/Interior Walls Sheathed with Gypsum Board – with or without Openings

When fire ratings are required for apartment buildings, townhouses or condominiums a contractor may select exterior "X"grade Gypsum sheathing, fastened with nails, staples or screws. The Uniform Building Code (UBC) assigns allowable shear values for use as shear walls, but not for direct tension uplift. This topic will be discussed in Chapter 3.

Creative Tension Chain Ideas Used by Contractors

The natural creative ideas from contractors to minimize labor and material, while still achieving the required strength of the tension chain, are many. A few of these ideas catalogued under the specific categories used above follow.

The Structural Sheathing of the Wall Components

The creative input of one contractor has led to the idea illustrated in Figure 2.13 for the placement of structural sheathing while constructing the entire length of a wall elevation. The top structural sheet of OSB board is placed horizontal at the top of the wall, while it is still lying flat on the first floor plane. The entire wall becomes a stable wall for lifting and placement. Note that the top plate of the double top plate is included at this stage. The remainder of the sheathing is placed from grade, after the wall is positioned vertically and braced. This sheathing is placed with its grain oriented vertically. This approach avoids ladders and results in fewer accidents. This same approach can be done with plywood. Regardless of structural sheathing choice, the sheets must be cut resulting in some waste since the wall height of the sheathed exceeds 8 feet. Reference pages 37-38 and Figure 11 in Chapter 4 of the *WMM*. When finished, the structural sheathing covers the wall from the top of the double top plate to the bottom of the mudsill in a continuous tension plane. The blocking at panel free edges is not shown for clarity, but is then installed.

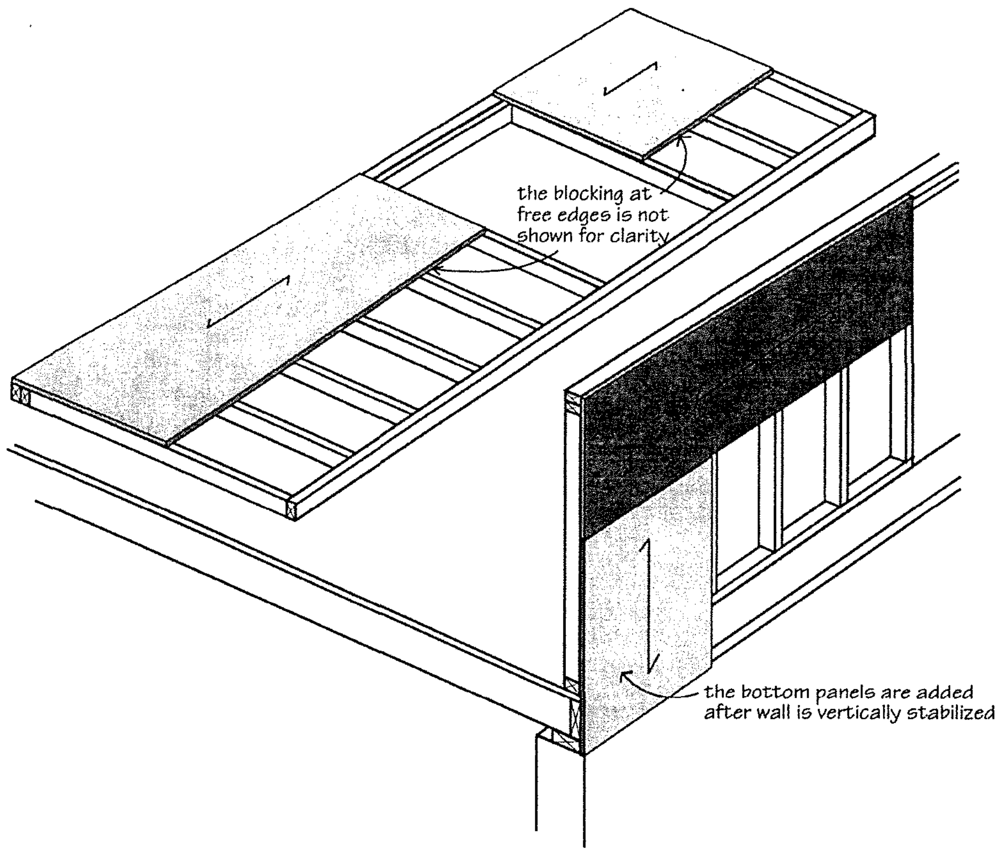


Figure 2.13 - Unique Approach to Sheathing Stud Wall

The Roof Truss/Rafter to the Structural Wall Plane

Ideas to eliminate metal connectors between roof truss/rafter and the structural wall plane can be unique. The extension of the structural sheathing to the top of "I" joist as detailed in Figure 2.14a and 2.14b, represents a one-story residence and a two-story residence respectively. The one story solution creates a cathedral ceiling and detailing elsewhere must prevent the horizontal outward movement associated with cathedral ceilings. A 2x6 wood blocking and a sufficient number of nails are required to resist the design uplift and tie the rafter to the wall sheathing. This method is preferred by some contractors to eliminate the time that is required to install the significant number of metal connectors when "I" joists will be used for the roof framing. Overhangs, if desired, can then be prefabricated and attached directly to the wall sheathing. The illustration shows a dotted overhang. The same approach to the tension chain could be accomplished with dimension lumber rafters. A squash type dimension lumbėr side plate would need to be appropriately nailed to the side of the rafter, again with its grain oriented vertically. This would allow nailing the structural panel into side grain rather than end grain of the rafter itself.

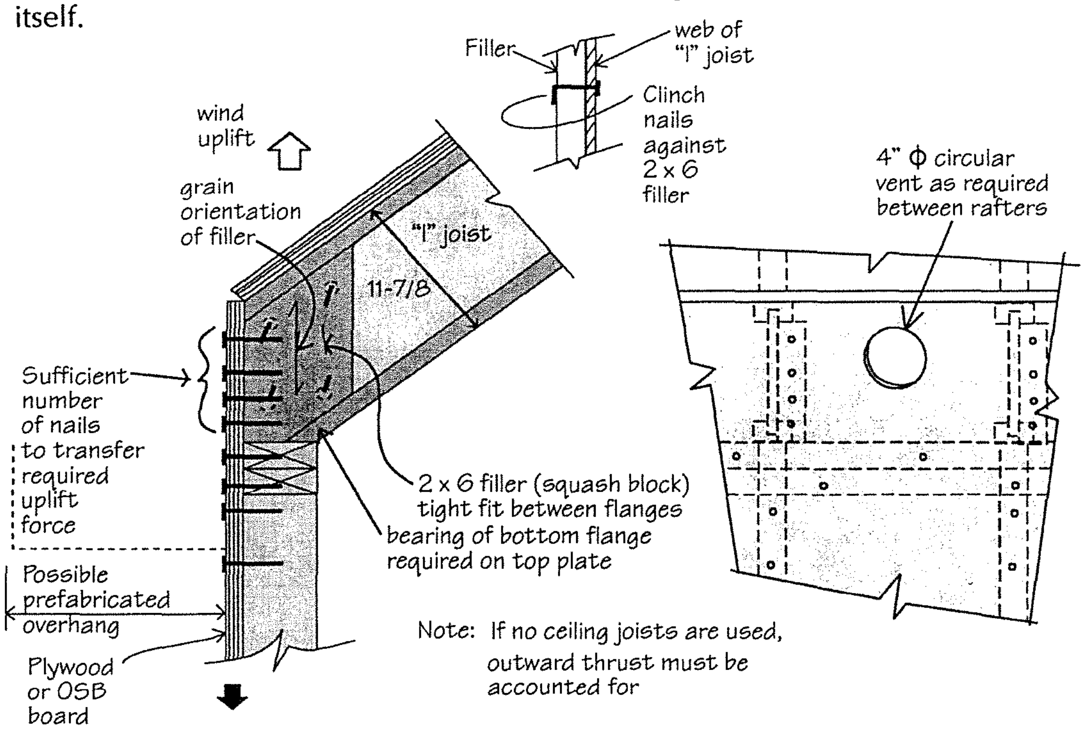


Figure 2.14a - Roof to Wall Detail
(cathedral ceiling)

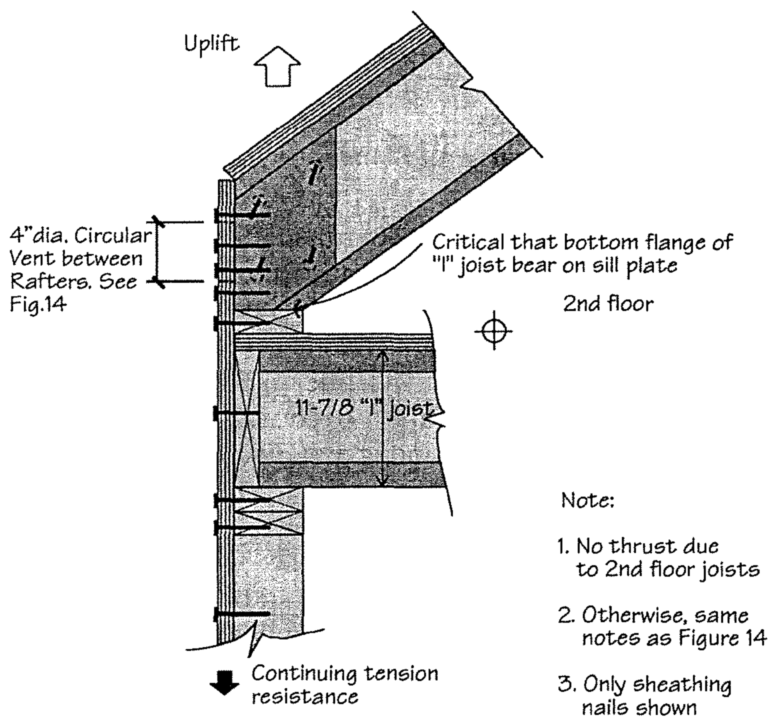


Figure 2.14b - Roof to Wall Detail
(2nd story)

A similar concept is possible with roof trusses that have extended verticals at their bearing ends, as shown in Figure 2.15. Again, a sufficient number of nails are required to resist the design uplift and tie the truss to the wall's structural panels.

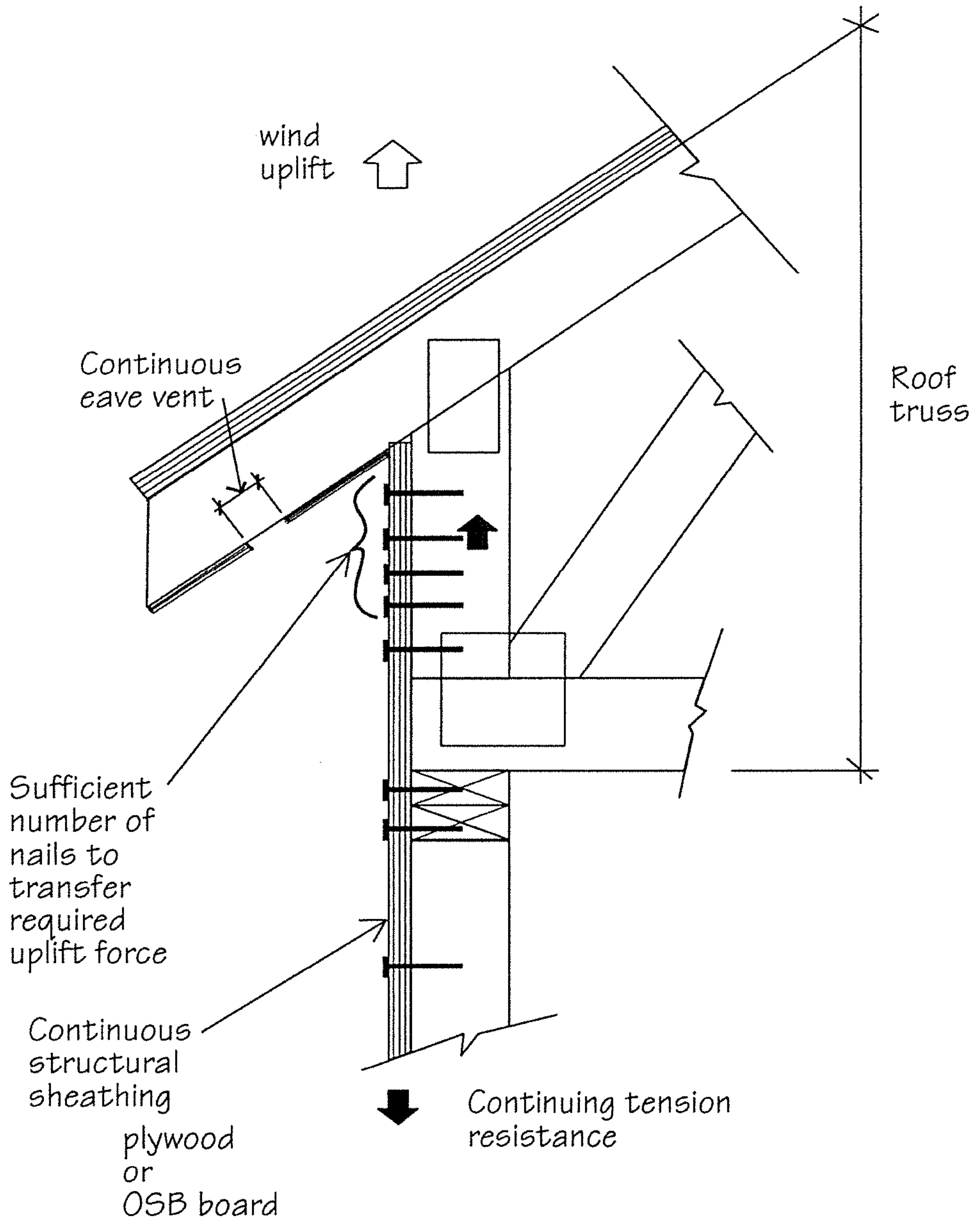


Figure 2.15 - Roof to Wall Detail

The use of roof trusses with an extended vertical is not very common in conventional residential construction, but has a particular function in custom homes as shown in Figure 2.16. It is possible to maintain a constant roof height for a home, while varying the interior spaces. For example, a taller (9 foot ceiling in this case) cathedral space in one location, while having an 8 foot flat ceiling in other spaces with a second story or an attic above, might utilize all three details shown in Figures 2.12a and 2.12b and 2.13. Again, this is only one approach to the variable ceiling height issue. Note that the "I" joist rafters are tied together at the ridge with solid "L" shaped plywood gussets to provide some continuity and moment resistance to the outward movement at the walls of the cathedral space.

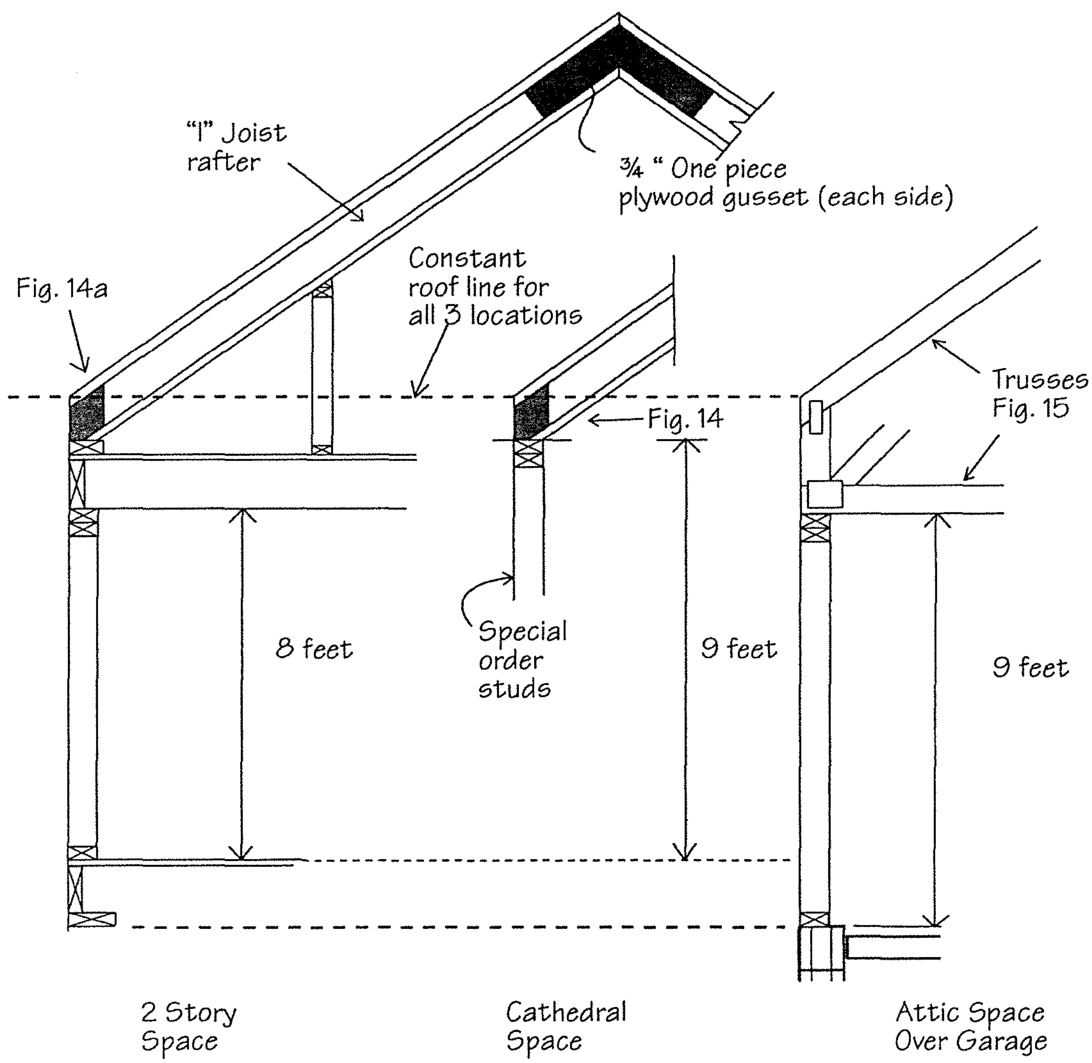


Figure 2.16 - Variable Interior Ceiling Heights

The Wall Sheathing to the Foundation

One contractor's approach to dealing with the large washers at the garage walls, was to place a first bottom plate on top of the foundation wall without recesses at the anchor bolt locations. He then used these as a template for a second bottom plate that then became the bottom plate for construction of the stud wall. Thus, the full strength of the 2x4 mudsill was available and there was no interference from the anchor bolts for the layout of the studs. Figure 2.17 illustrates this detail.

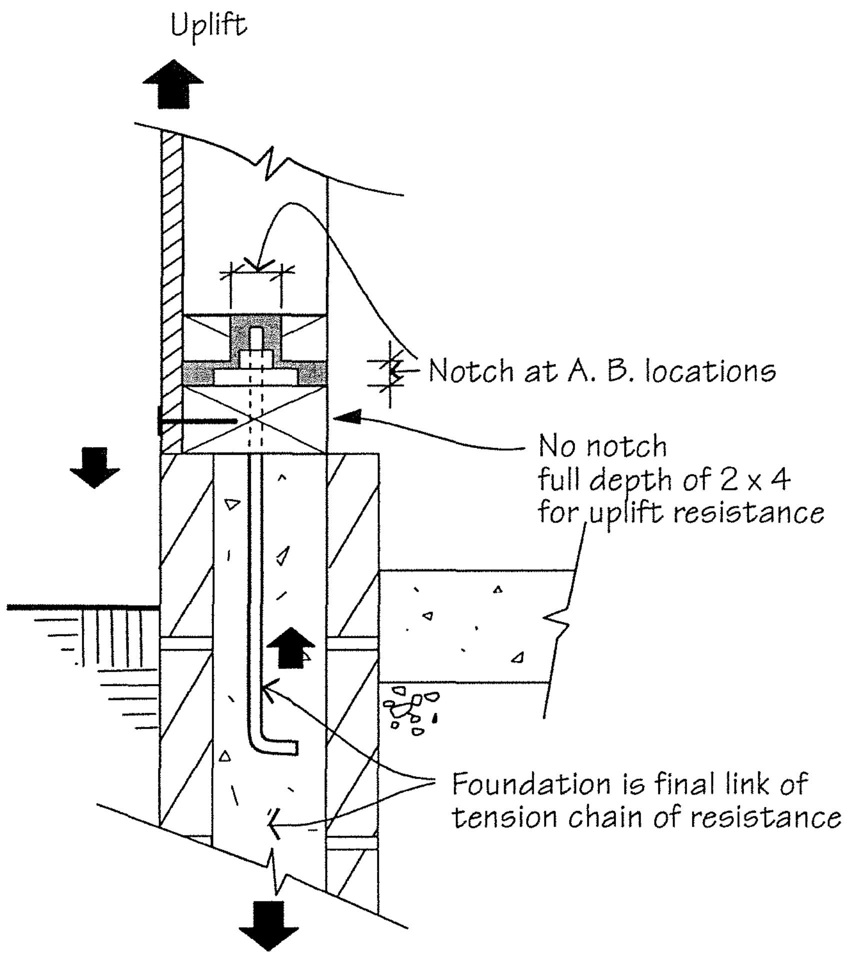


Figure 2.17 - Garage Stud Wall

Uplift Recommendations for Sections of Walls with Openings

The above discussion applied to exterior solid wall areas, where no doors or windows are present. Openings interrupt the transfer of uplift force through the sheathing. The uplift force suddenly concentrates at each side of every opening, transferring from the header over the opening. The larger the opening, the larger the tension uplift force to be transferred to the foundation. The span of the wood header (**H_L**) over the opening is the key to determining the magnitude of the concentrated force to be transferred to the studs and then to the foundation. It is clear that normally the garage door opening will be the largest concentrated force to be transferred to the foundation. Figure 2.18 illustrates typical openings.

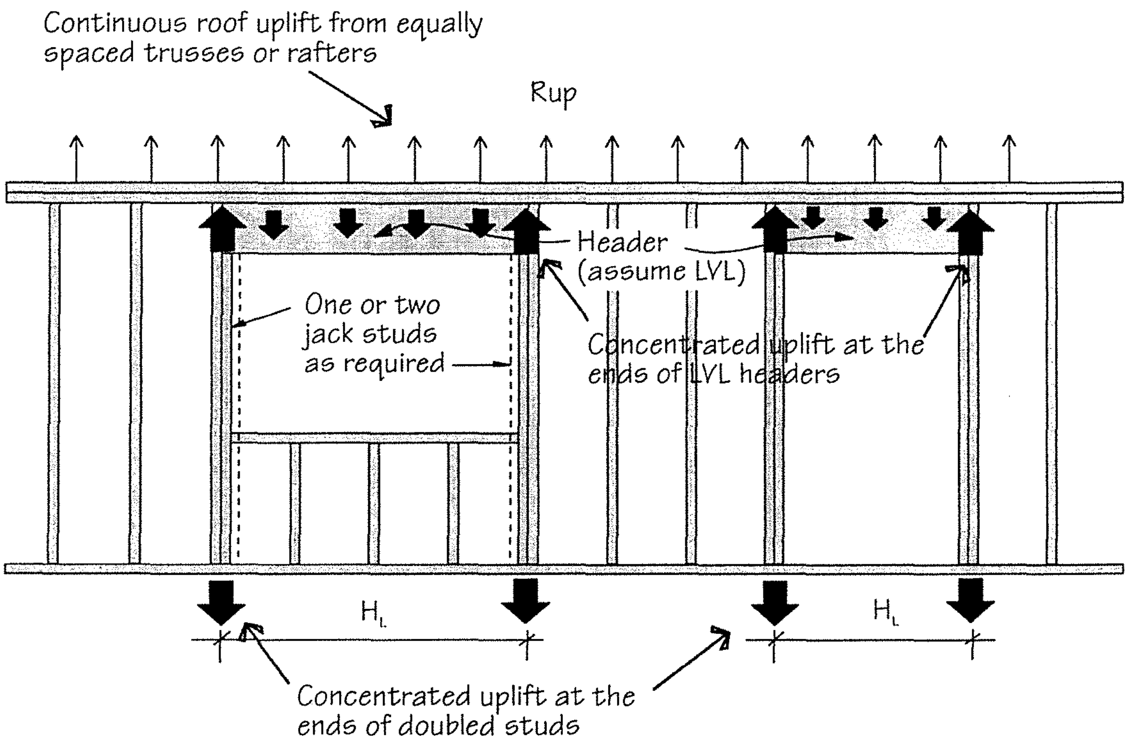


Figure 2.18 - Typical Openings in Exterior Wall

Figure 2.19 is a dissected view of the components that surround an opening, and the design wind suction reactions at each roof truss or rafter. The arrows show the concentration of forces transfering around the opening. The important connections are as follows:

1. The roof trusses or rafters to the header, bypassing the double top plate;
2. the header bearing ends to the studs; and
3. the studs to the foundation, bypassing the first floor construction.

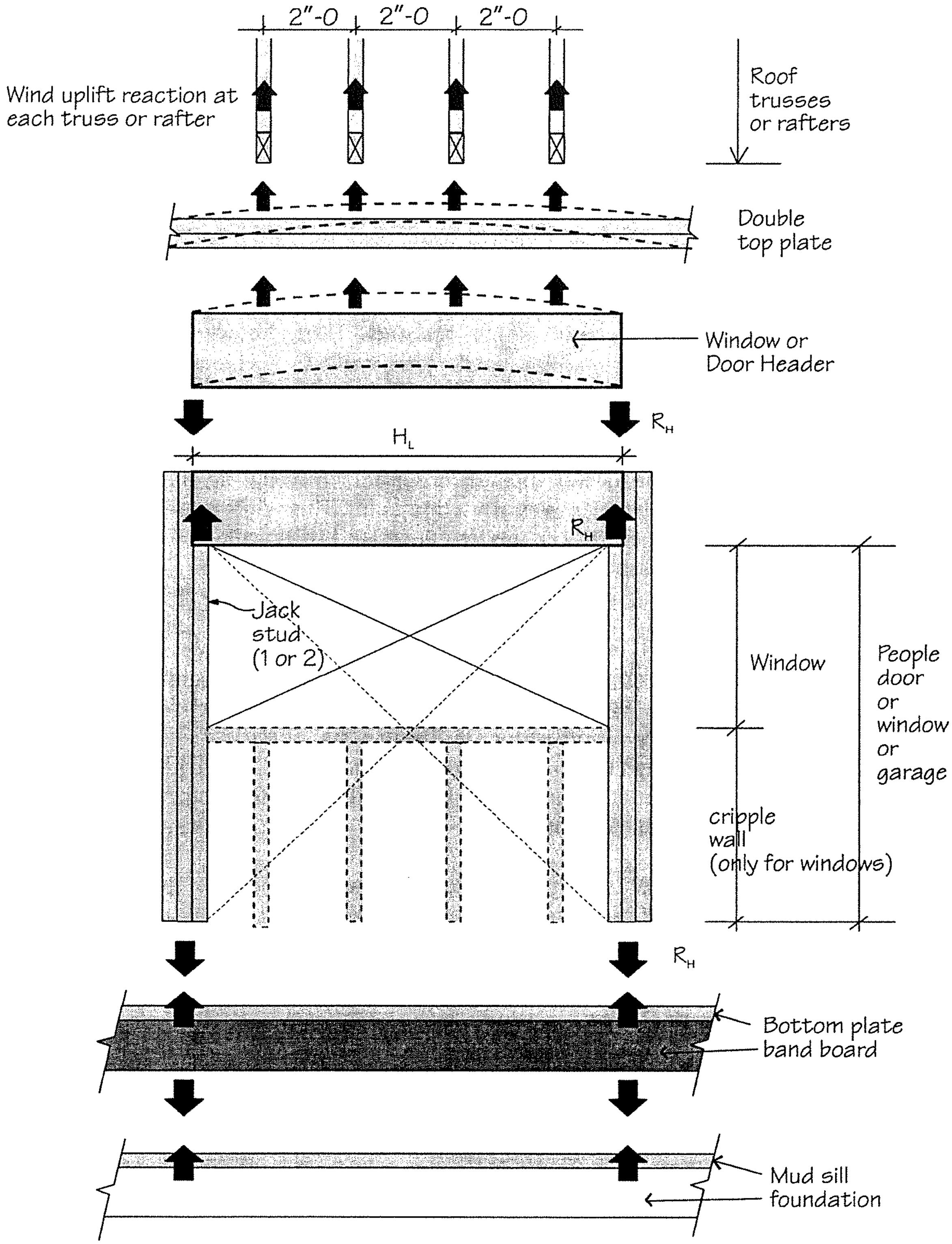


Figure 2.19 - Exploded View of Uplift at Opening in Exterior Wall

These three connections will be discussed separately.

A longer metal connector or a twist strap is required to reach the wood header and to transfer each roof truss reaction to the header. It is also possible for the structural sheathing to transfer this suction to the header, but that will depend on the magnitude of the truss uplift values.

Header to Studs at Ends of the Opening

The header tries to bend upward and lift off the jack studs. Thus, the header transfers an uplift end reaction ($\mathbf{R_H}$) to the jack and full-length studs, typically by means of metal straps. It is true that the structural wood sheathing may be able to assist or transfer all of the reaction from the header. It is not desirable to concentrate too much force into the sheathing and depend on a group of nails to be positioned just right. Two car garage door openings support the longest headers. If the header supports the bearing ends of roof trusses, the entire 16-foot header span is subjected to uplift. This will most likely be the longest single span around the exterior of the residence, and will produce the largest uplift force to be transferred to the foundation.

Studs to the Foundation

The uplift force ($\mathbf{R_H}$) is transferred from the studs to the foundation by means of hold-downs, such as found in Figure 14 of Chapter 7 in the *WMM*. These hold-downs use a rod that is drilled through the subfloor and bottom stud wall plate. The rod then goes past the bandboard and is drilled through the mudsill into the foundation. These rods are typically epoxied into the concrete foundation or grouted block cell, rather than placed during the foundation concrete or grout placement.

All the connectors will be larger or longer to accommodate more nails for garage door openings. The hold-downs will be a larger diameter rod embedded deeper into the foundation and more foundation weight will need to be engaged.

Covered Entry Porches or Patios

There is an infinite variety of possible entry-porch and patio forms and proportions that are linked to residences. A roof section is generally provided over them for weather protection. These roofs are many times open on three sides and are therefore supported at the open-end corners by posts.

Wind can move under as well as over these roofs creating positive pressure on the underside and negative pressure (suction) on the topside. The pressure pushes up on the underside of the open roof and the suction pulls up on the top surface of the roof. Thus, these two wind actions combine to produce considerable uplift on the roof.

The ASCE 7-95 wind provisions refer to this as an "open structure" and mathematically treats this condition differently than enclosed buildings to determine the uplift design force. It would be impossible to treat every porch or patio condition, but a typical porch will suffice to describe the approach to the tension chain of resistance to uplift. Figure 2.20 illustrates such an entry porch. The porch roof is attached directly to the roof of the residence, and is usually composed of rafters in combination with roof trusses as shown. The portion of the porch roof that projects is supported by beams that are in turn supported by corner posts that bear on a concrete porch that is supported by a foundation below grade. The projection of the roof beyond the house (**B_P**) is an important dimension. The structural wood posts are usually a pressure treated (wolmanized) 4x4 or 6x6, depending on strength or aesthetic considerations. This will avoid rotting of the wood due to exposure to moisture.

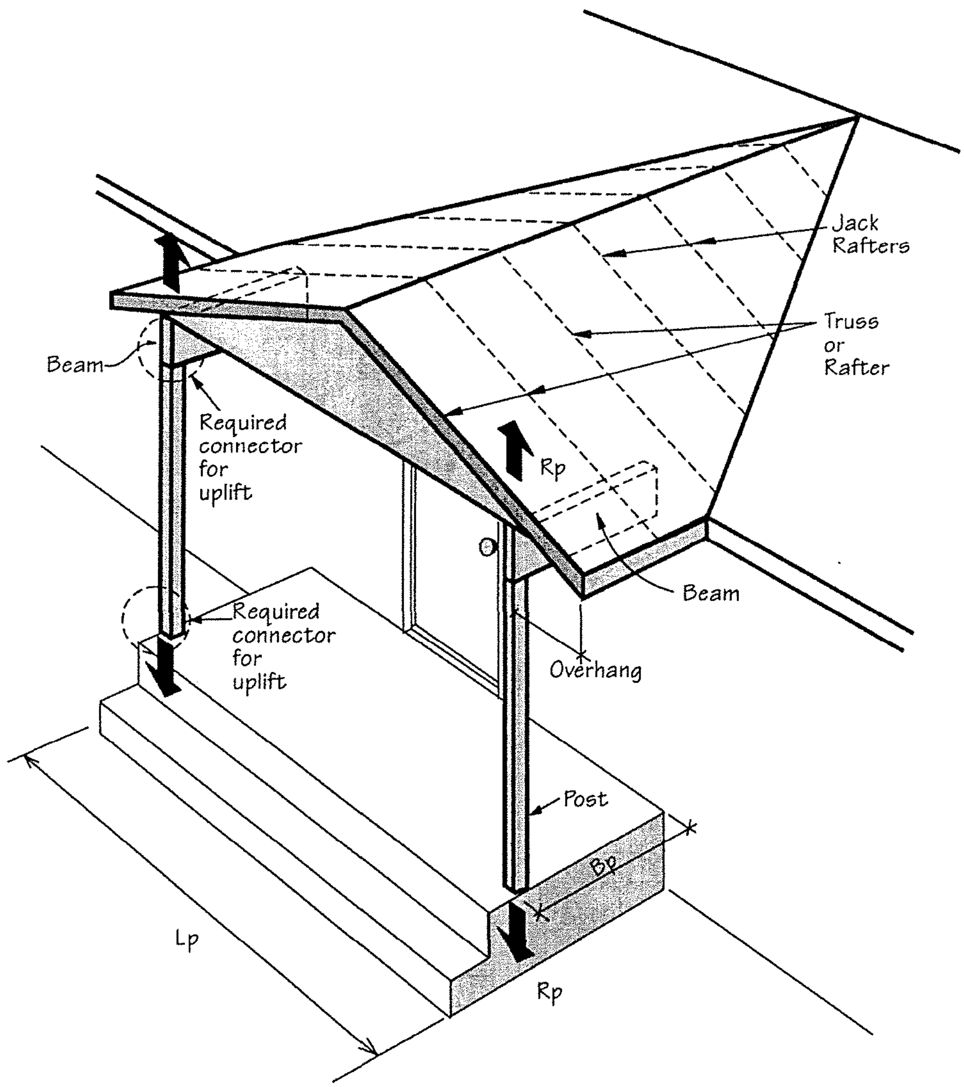


Figure 2.20 - Typical Porch and Support Posts

The load flow from wind uplift follows a typical pattern from roof to foundation, and the following connections compose the tension chain:

1. Roof trusses or rafters to the projecting beams
2. Beams to the wall of the residence and the corner post
3. Corner post to the porch concrete
4. Porch concrete to the foundation

Figure 2.21 separates all the porch components that require connections to complete the tension chain. These connections will be discussed separately.

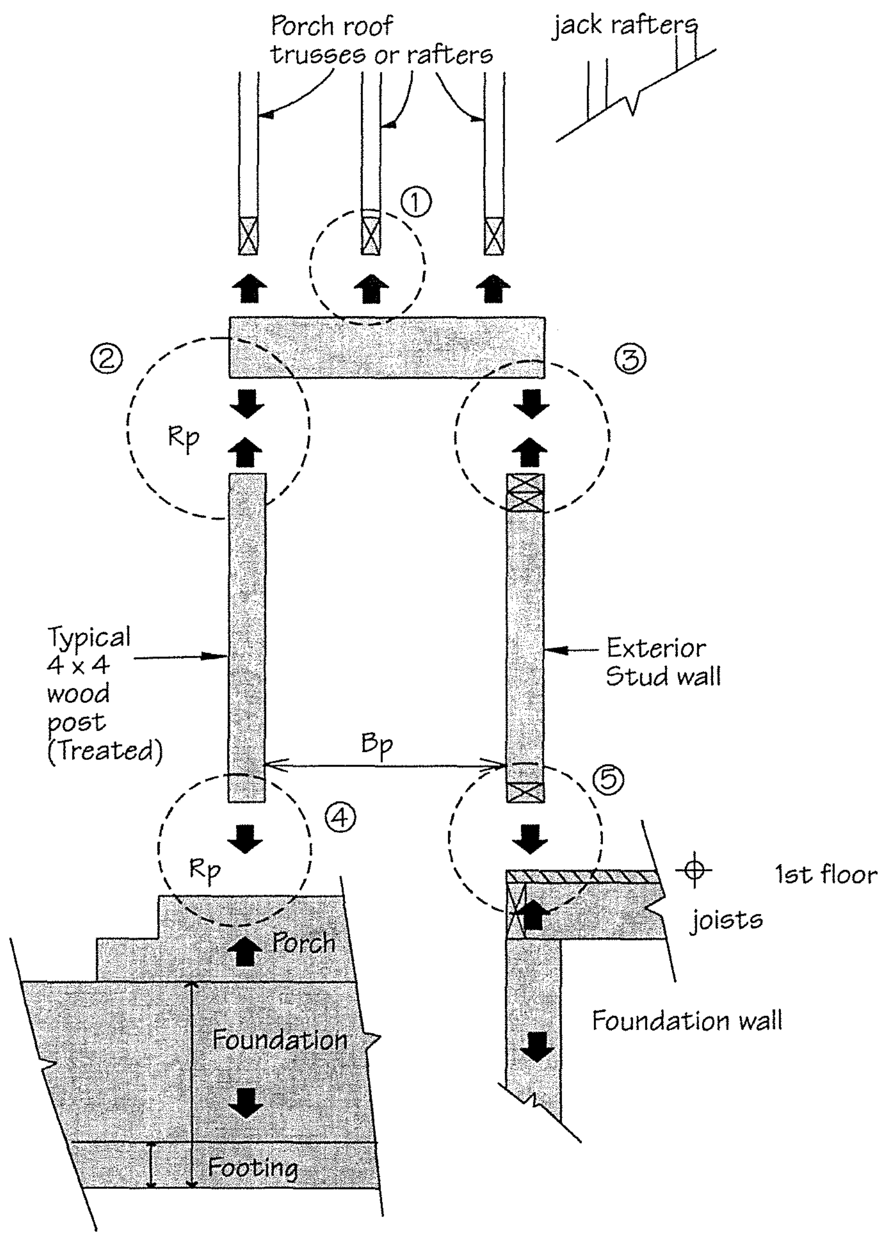


Figure 2.21 - Exploded Side View of Porch - Uplift Chain Defined

Roof Truss or Rafter to the projecting beam

Once the design wind uplift reaction is determined, metal connectors can be selected to tie the rafters or trusses to the wood beam. These connectors would be very much like those found in Figure 5 of Chapter 7 in the *WMM*.

Beam to the corner post and the wall of the residence

The uplift reaction at the ends of the beam is ($\mathbf{R_P}$). This is the design uplift force that must be transferred from the beam to the post. Many metal connector options exist, depending on the magnitude of ($\mathbf{R_P}$). Several examples are shown in Figure 2.22. Several depend on nails for load transfer, while some depend on bolts. Bolted connectors will have bolt head, nut and washers projecting from the post. If the post is clad with finish wood, shim material will be required. This architectural treatment will make the size of the column much bigger than structurally required.

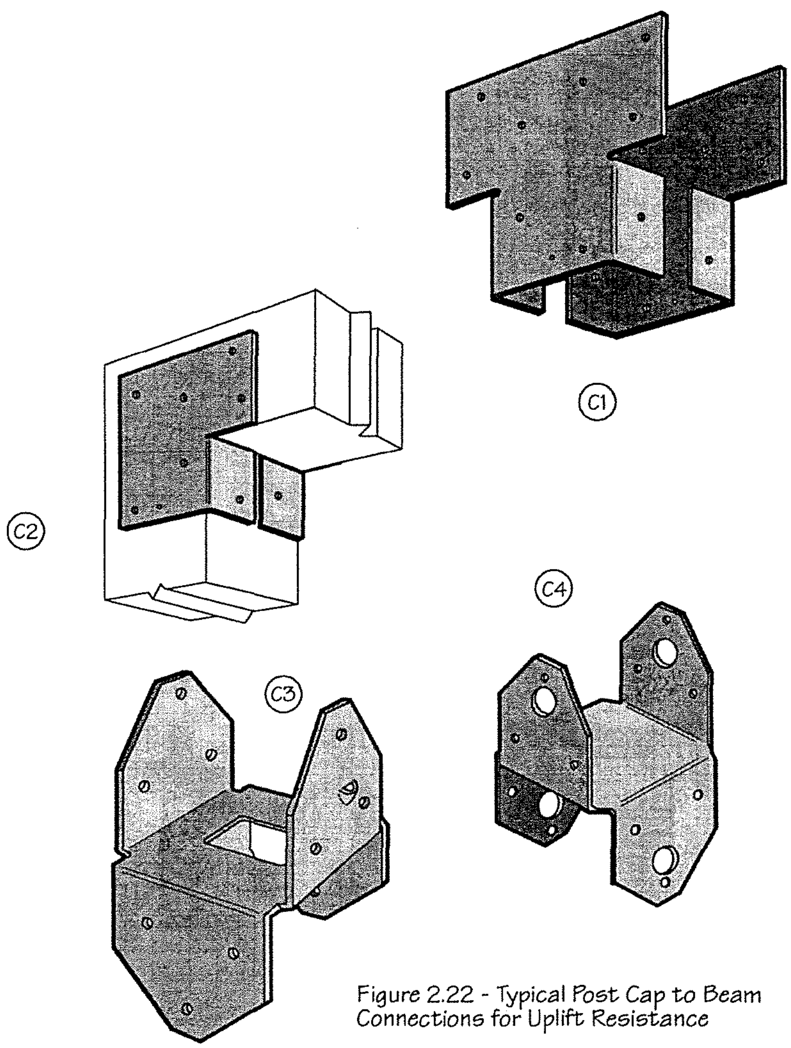


Figure 2.22 - Typical Post Cap to Beam Connections for Uplift Resistance

Corner post to the porch concrete

There is a variety of column post bases that are capable of transferring tension uplift to the porch concrete. Several such metal connectors are illustrated in Figure 2.23. Notice again, that either nails or bolts, or a combination of both are used to connect to the wood post. It is assumed that either the connector is set in the concrete as the porch is poured, or that an epoxy anchor is drilled and set after the concrete cures. Most contractors prefer the later detail, as setting the metal base requires exact placement or the post will not be straight.

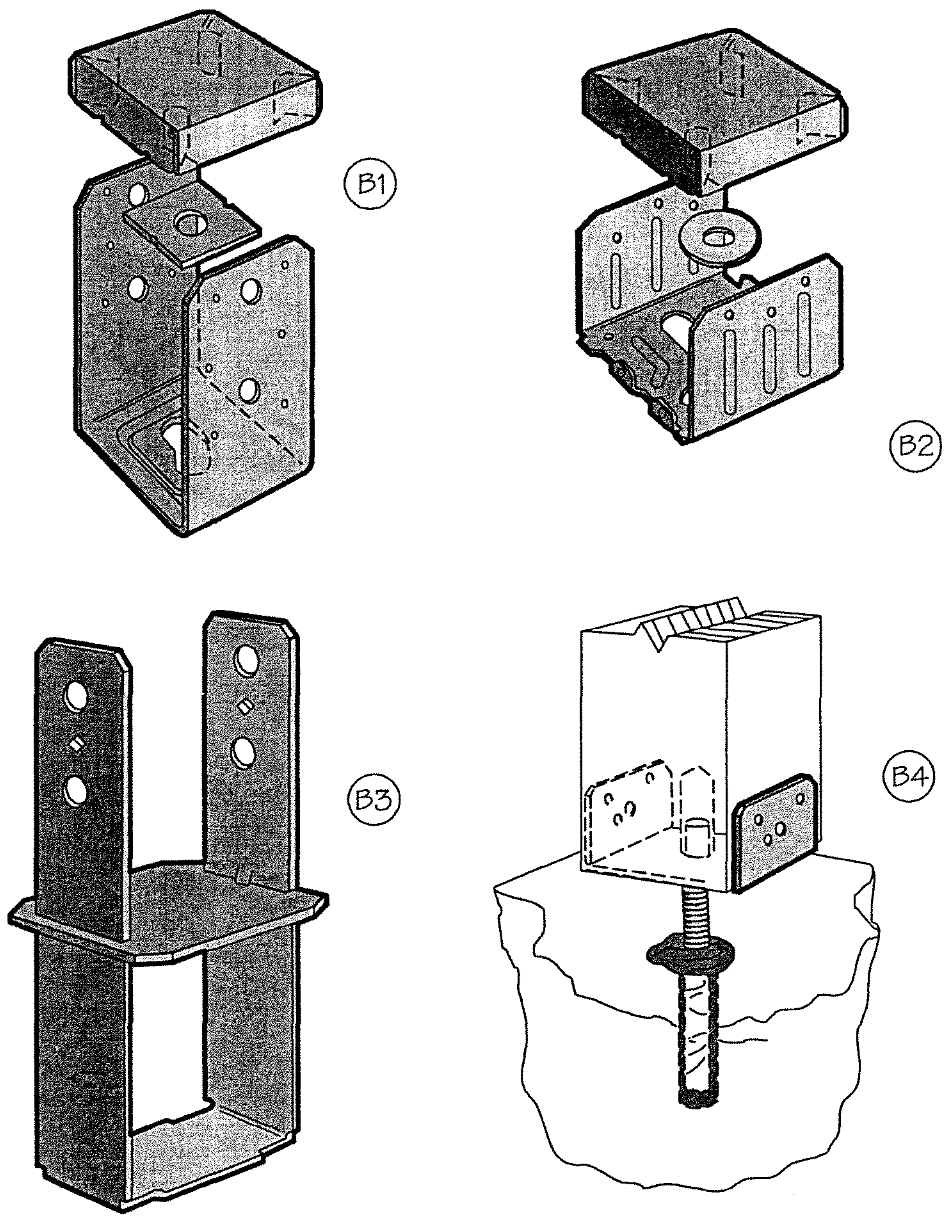


Figure 2.23 - Typical Post Base Connections for Uplift

Porch Concrete to the Foundation

It is most likely that sufficient weight of concrete exists in the porch construction and its reinforced connection to the foundation that the tension chain is complete.

Cantilevered Second Floor Planes

Cantilevered second floor planes are fairly common. Figure 2.24 (A) illustrates when the floor and walls above and below have their framing in alignment. Metal connectors can be used throughout to accommodate the tension chain of wind resistance. Structural sheathing is another alternative even if the framing aligns. Figure 2.24 (B) illustrates when the wall and floor framing does not align. Structural sheathing is the likely choice here; however, a metal connector is still required to tie the second floor to the lower wall plane.

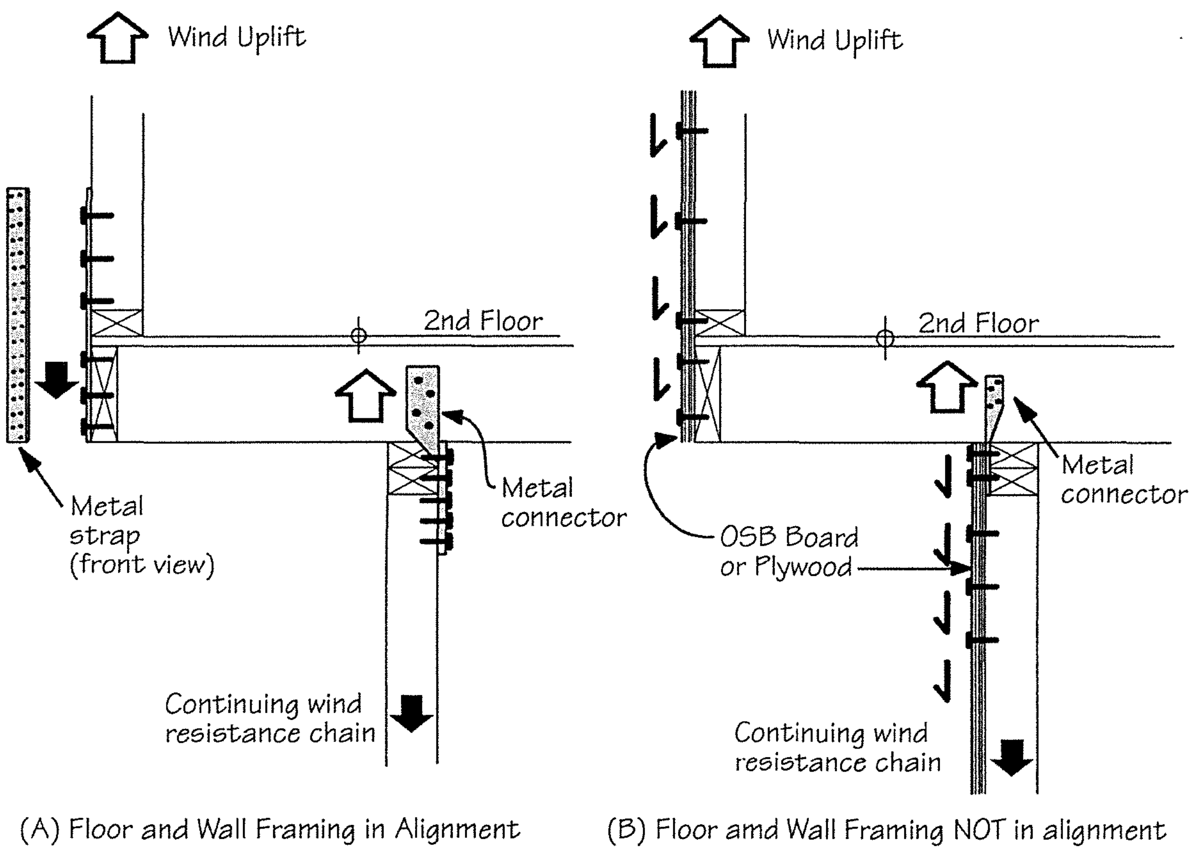


Figure 2.24 - Cantilevered Second Floors

Insulating Concrete Forms

There are numerous types of insulating forms used to create reinforced cast-in-place wall systems [2.5]. A few of these are illustrated in Figure 2.25. Regardless of the system employed, the basic concept is to use 2 layers of rigid insulation separated by ties that hold horizontal and vertical reinforcement in place and maintain their verticality while concrete is poured between the insulation layers. The end result is a box of insulated reinforced concrete walls that can have other finish materials applied to the insulation inside and out. The forms can be used for crawl spaces, full basements or even two story walls. They are becoming popular for two story condominiums, where they are used as separation firewalls between condo units. Ledger boards are expansion bolted, epoxy anchored or anchor bolted at the floor levels. Metal joist hangers attached to the ledger boards then support the wood floor level framing.

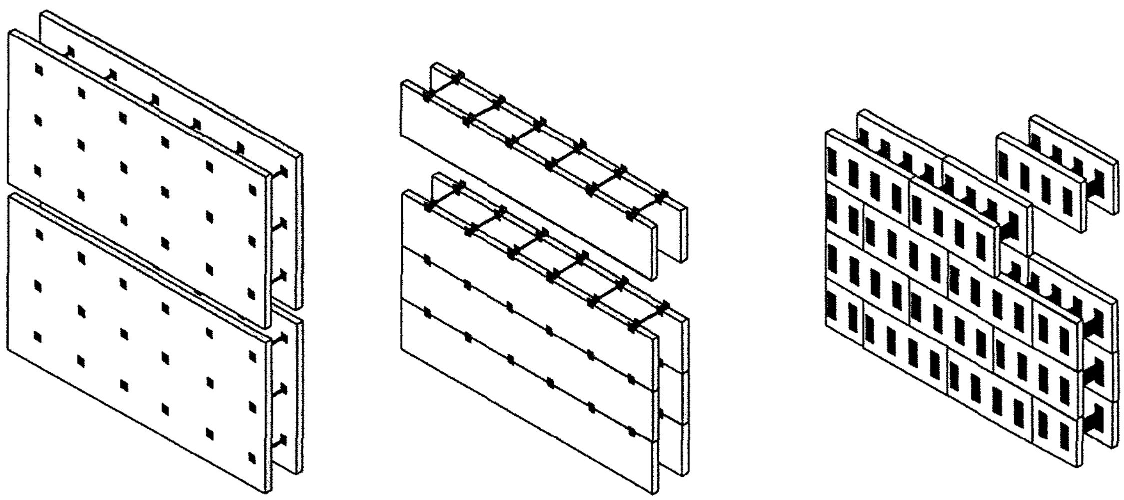
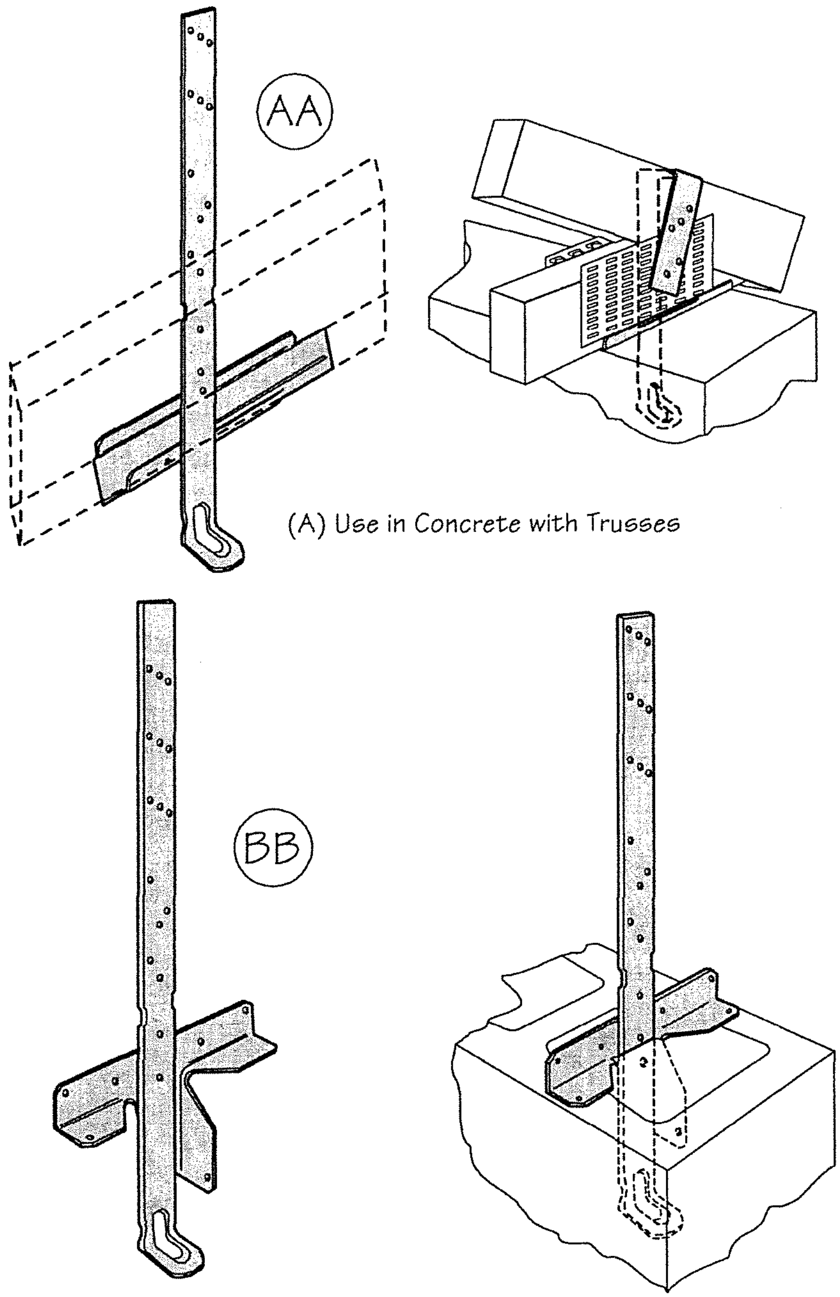


Figure 2.25 - Several Insulating Concrete Form Types

Significant weight is provided in these walls, whether for crawl, basement or two story applications. The uplift chain of resistance is easily provided by this system. Wood roof trusses or rafters are installed atop the two story walls to complete the roof enclosure of the wall system. This truss/rafter connection to the concrete wall is the only metal plate connection that must be provided to start the tension chain resistance from the roof. Reinforced concrete block walls can also be built to provide the walls of the box. Similar details are incorporated to tie the roof to the block wall. Both of these wall types and representative roof truss/rafter details are shown in Figure 2.26. A potential problem, when the rigid foam insulation comes in contact with the earth, is the potential for increased termite infestation. There is no definitive explanation for this surprising statement. The CABO Code now requires rigid foam insulation to start a minimum of 6 inches above grade. Insulating concrete forms can be used for foundation walls if non-combustible materials, such as steel, or treated lumber are used for the wood framing mudsills of the superstructure. This is not an issue when the insulating concrete forms extend from the footing to the roof. No access to wood is provided for the termites [2.6]. This issue is beyond the scope of this Manual and the references at the end of this chapter should be consulted.



(A) Use in Concrete with Trusses

(B) Use in Concrete Block with Trusses

Figure 2.26 - Typical Concrete/CMU Truss Connectors

Tension Chain Summary

It is clear that **all the exterior walls, plus the roof and foundation components of a residence participate in the tension chain of resistance**. This resistance can be totally accomplished with metal connectors or by a combination of roof and foundation metal connections linked to a wall that is structurally sheathed continuously from the top of the double top plate to the bottom of the mudsill. This sheathing can be accomplished with either Structural panels.

The **total metal connected tension chain of resistance** can incorporate the typical types of non-structural exterior 1-inch rigid insulation board. Yet the stability of the residence will require some structural sheathing to resist overturning and sliding.

The use of **continuous exterior OSB or Plywood can be used to develop the total tension chain of resistance**. Super batt insulation rolls inside the walls can make up for most of the lost R-value. However, it should be kept in mind that the real insulation loss is not at the solid wall locations, but at the seals around openings where infiltration of air is the main loss of R-value. It might be of interest to use 2x6 stud walls instead of 2x4 stud walls to accommodate more insulation, however this may not provide an R-value worth the increase in stud and insulation cost [2.7]. This issue is beyond the scope of this Manual and the references at the end of the chapter provide further reading.

Sufficient foundation weight is required to complete the tension chain of resistance and hold down the house. This weight is easily provided when basements are used, but less weight is available when crawl spaces or slabs on grade are used. Obviously, unique situations found mainly in custom homes require use of internal anchorage techniques not covered here. The input of a structural engineer is recommended when homes become that complex.

Selected References

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2.2 Connector Update, News for the Construction Community, *Simpson Strong-Tie Co., Inc.*, Vol.1, No.2, July 1989,pp.2-3.

2.3 National Design Specification for Wood Construction, ANSI/AFandPA NDS-1997, 1997 Edition, *American Forest and Paper Association, American Wood Council.*

2.4 Connector Update, News for the Construction Community, *Simpson Strong-Tie Co., Inc.*, Vol.1, No.2, July 1989,p.1.

2.5 Thompson, Donn, "Building with Insulating Concrete Forms", *Portland Cement Association, Concrete Technology Today*, Vol.19/No.2, July 1998, 1-3.

2.6 "Two-By Debate Continues", *The Journal of Light Construction*, June 1998, 16.

2.7 "CABO Limits Rigid Foam in Ground Contact", *The Journal of Light Construction*, August 1998,12.

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Chapter 3 - Wind Structural Planning Basics

The Stability System

Introduction

The stability issues of **Overturning and Sliding** comprise the last of the structural design issues for the lateral load resistance system. These topics will comprise the emphasis of this chapter. This is without a doubt the most complicated, but most important issue for the planner of a residential floor plan and its elevations. The Chapter will begin with basic concepts and then gradually apply these concepts in the structural planning of several residences of varying floor plan complexity.

Main Wind-Force Resistance System wind loads, as described in the "Wind Loads For Design" section of Chapter 2, are applied to the entire exterior planes of buildings for Overturning and Sliding resistance design. The term "**diaphragm**" for structural use of floor and roof planes and the term "**drag strut**" will be fully explained in this chapter. See Figure 3 in Chapter 5 of the *WMM*. The illustrations and text of Chapters 5 and 7 of the *WMM* will be used to supplement the information and ideas presented in this chapter.

The Stability System Defined

The stability of the total residence involves its resistance to ***overturning*** and ***sliding*** as a total building. The wind loads used for the stability design are those of the ***Main Wind-Force Resistance System***, described earlier. Wind pressures, both positive and negative, hit large tributary surface areas perpendicular to the wind direction. These surfaces transfer the wind pressures and suctions to the roof and floor planes and then to the vertical wall planes parallel to the wind direction. Now the horizontal components of the wind applied to the building are to be resisted; as shown in Figures 10 and 11 of Chapter 1 in the *WMM* along with all the figures and text of Chapter 5 in the *WMM*. Wind resistance for stability is generally an engineering review in two primary directions:

(a) Perpendicular to the long dimension of the building; and
(b) perpendicular to the short direction of the building.

These are illustrated in Figure 3.1.

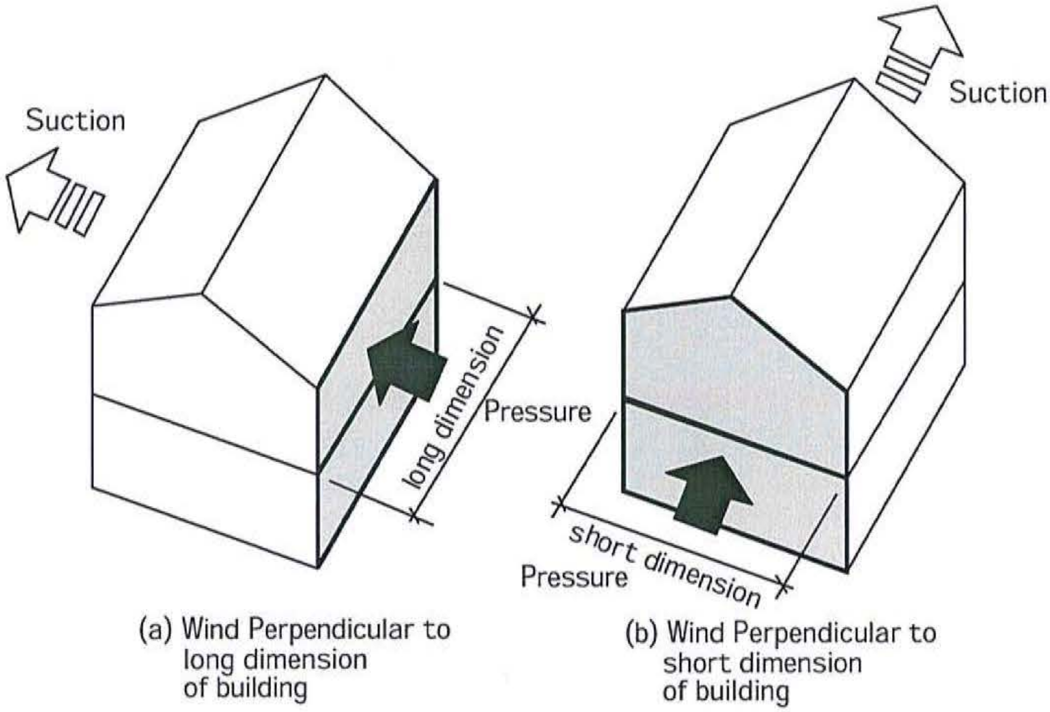


Figure 3.1 - Stability-Wind Directions for Structural Analysis

Just as for the issue of uplift and the tension chain of resistance, the building again must be thought of as composed of planes: wall, roof and floor planes. Since the primary wind force is now horizontal, specific planes must be selected to perform the resistance function.

Figure 3.2 has exploded a simple two-story rectangular residence. For the moment assume there are no openings in the exterior walls, thus all the planes are solid. A wind force perpendicular to the long dimension of the building means that the largest surface area is exposed to the wind. **The largest sliding and overturning forces generally occur when wind is applied perpendicular to the long dimension of the building.**

Returning to Figure 3.2, the windward walls and windward sloping roof plane receive the wind positive pressure, while the leeward walls and leeward sloping roof plane receive the negative pressure (suction). The wind forces against these wall plane and sloping roof plane surfaces, perpendicular to the wind direction, push and pull at the roof and floor planes, which are called "**diaphragms**".

Diaphragms behave like deep beams with their "web" being analogous to the floor plane or roof plane. Diaphragms therefore are assumed to behave like beams laid on their side, sending their wind generated reactions to the end walls, called "**shear walls**".

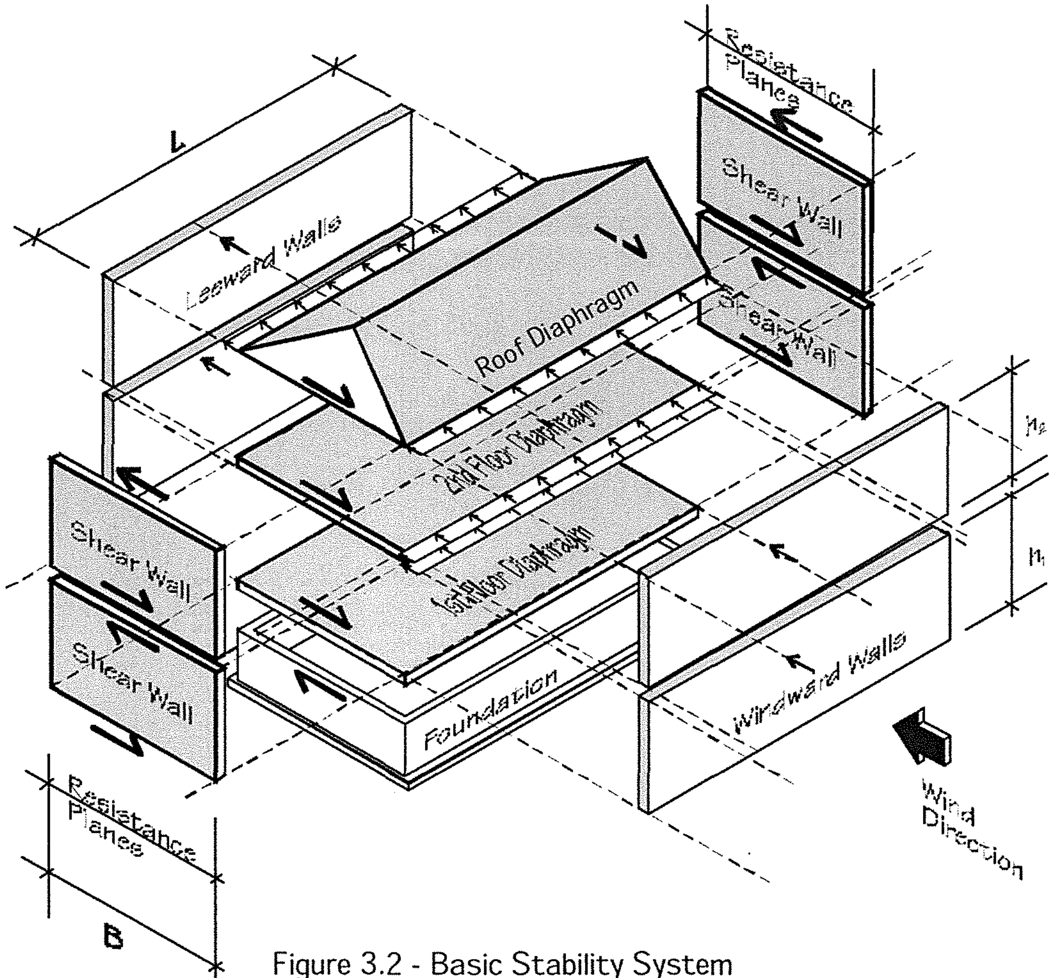


Figure 3.2 - Basic Stability System
Wind Perpendicular to Long Dimension (L)

The walls that are physically connected to the diaphragms receive their share of the reaction according to a distribution based on tributary area. Tributatry area is measured as half the distance to the other shear walls multiplied by half the height of the wall. Once the wind reaction is transferred to the shear walls, the connection of the shear wall to the diaphragm forces the reaction to move down the wall in shear. Thus the name "shear wall". The shear is resisted by the material of the shear wall and it continues down the wall to the foundation.

The shear walls become the stability system for horizontal wind resistance perpendicular to the long direction of the residence. These shear walls must resist the tendency to ***slide*** and the tendency to ***overturn.*** This is accomplished by proper connection and anchorage to the mass of the foundation below the shear walls. The sliding is resisted by foundation anchor bolts spread along the length of the bottom of the shear walls. The overturning is resisted by anchorage to the foundation wall at the ends of the shear walls. Again, reference should be made to Figures 3 and 4 of Chapter 5 in the *WMM*.

Figure 3.3 has exploded the same simple two-story rectangular residence shown in Figure 3.2. Again, for the moment assume there are no openings, and all the planes are solid. The wind force is now perpendicular to the short dimension of the building. This means that the smaller surface area is now exposed to the wind. **A smaller sliding and overturning force generally occurs when wind is applied perpendicular to the short dimension of the building**. Returning to Figure 3.3, the windward end walls and windward gable end wall receive the positive wind pressure, while the leeward walls and leeward gable end wall receive the negative wind pressure (suction). The wind forces against those surfaces push and pull against the roof and floor planes, again called the "**diaphragms**". These diaphragms are now extemely deep beams, sending their wind reactions to the side walls, which now become the "**shear walls**". These long shear walls transfer the smaller buildup of horizontal force in their plane in shear. They become the stability resistance system for wind perpendicular to the short direction of the residence. These long shear walls must resist the tendency to *slide* and the tendency to *overturn*. This is accomplished by connection and anchorage to the mass of the foundation. The **sliding** is resisted by the foundation wall anchor bolts spread along the length of the shear walls. The **overturning** is resisted by anchorage to the foundation wall at the ends of the shear walls. It is obvious, as solid very long walls, they generally have no trouble providing the required stability.

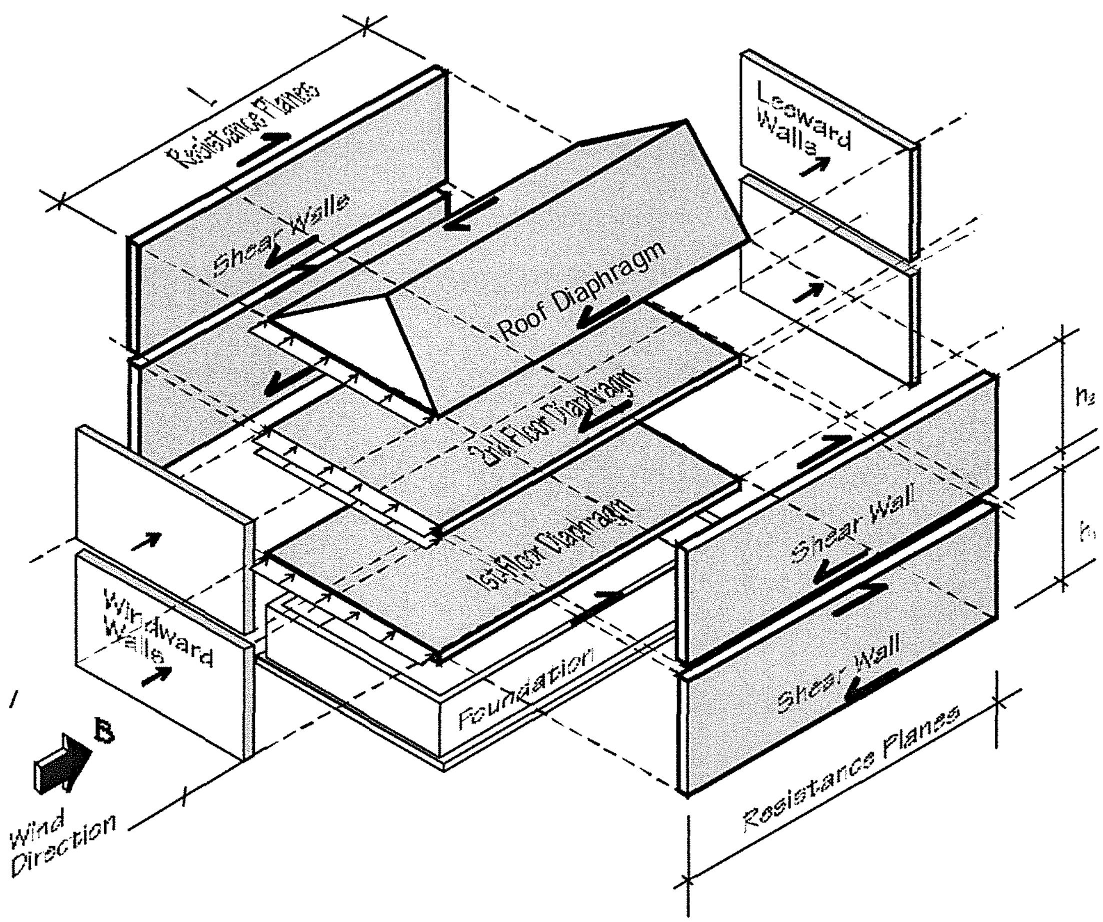


Figure 3.3 - Basic Stability System
Wind Perpendicular to Short Dimension (B)

This very basic example, when looked at in two mutually perpendicular directions, illustrates what commonly must be reviewed on every residence – most not as simple as the basic rectangle. However, it will serve to point out some very important structural planning concepts applicable to developing the stability system of any residence.

Stability Concepts Used in Structural Planning to Resist Sliding and Overturning

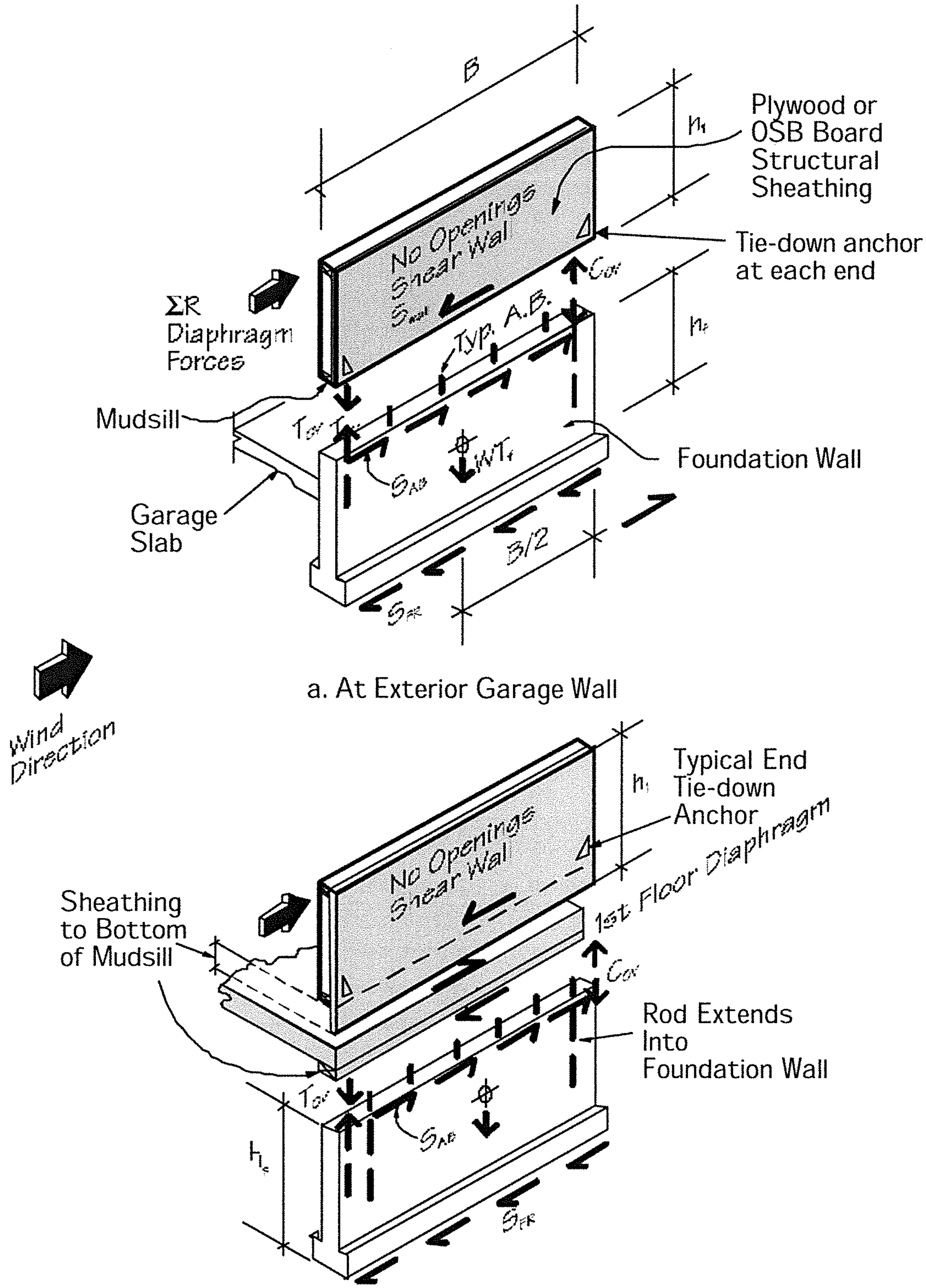
Foundation Walls

1. It is most desirable to have foundation walls under the superstructure walls selected as the shear walls for stabilizing the residence in each analysis direction.

   This concept is intended to provide sufficient foundation mass (weight) under the shear walls to be able to resist the overturning tie-down forces and use the foundation anchor bolts to resist the sliding. When the house proportions of length to width are less than 4, only exterior end walls are required for shear walls and therefore this concept is automatically provided.

   When the house proportions of length to width exceeds 4, an interior wall must be incorporated as a shear wall to reduce the proportions between shear walls back to 4 or less. Since an interior shear wall should also have a foundation below it, it is best to select a superstructure wall where a foundation change between a basement and a crawlspace takes place below a potential shear wall, or a basement and a slab-on-grade takes place below a potential shear wall. These concepts are illustrated in Figure 3.4.

   Figure 3.4(a) and detail Figures 9 and 11in Chapter 7 of the *WMM* shows the typical sliding and overturning that occurs between the shear wall and foundation at a garage condition. Figure 3.4(b) illustrates either a basement or crawl space condition, where a floor plane separates the shear wall from the foundation. Longer anchor rods are needed for the overturning tie-down anchors and again anchor bolts are used to resist the sliding, which is really the uniform shear along the base of the shear wall. Figures 9 and 14 in Chapter 7 of the *WMM* illustrate typical construction details.



a. At Exterior Garage Wall

b. At Exterior Foundation Wall-Basement or Crawl Space

Figure 3.4 - Concept 1-Foundation Wall Below Shear Wall

Garage Walls

2. Garage walls should always be used as tension chain walls to resist uplift forces and also as shear walls to provide the stability to resist overturning and sliding forces.

   Garage walls, except for the wall that contains the door opening, are an excellent choice for shear walls. Garage walls are particularly vulnerable in strong winds after the garage door has been breached. The tension chain to resist the increased uplift will require additional fastener requirements. These same walls will need to perform double duty and provide stability as shear walls for the increased horizontal wind forces. Luckily garage walls generally have foundations beneath the walls and one merely needs to check the foundation dead weight for its ability to complete the uplift tension chain resistance and the stability (sliding and overturning) resistance. So, **whenever possible garage walls should be sheathed with structural panels and used as shear walls**. Figures 6 and 7 in Chapter 1 of the *WMM* refer to the generic external and internal wind forces generated on an enclosed and open garage respectively, while Figure 8 in Chapter 1 of the *WMM* shows the distorted garage envelope relative to the wall elevation that contains the breached door. Figure 3.5a numerically illustrates the wind magnitudes for a closed garage door and then an open garage door. The wind pressures are based on Main Wind-Force Resisting system (MWFRS) values at 90-mph wind speed.

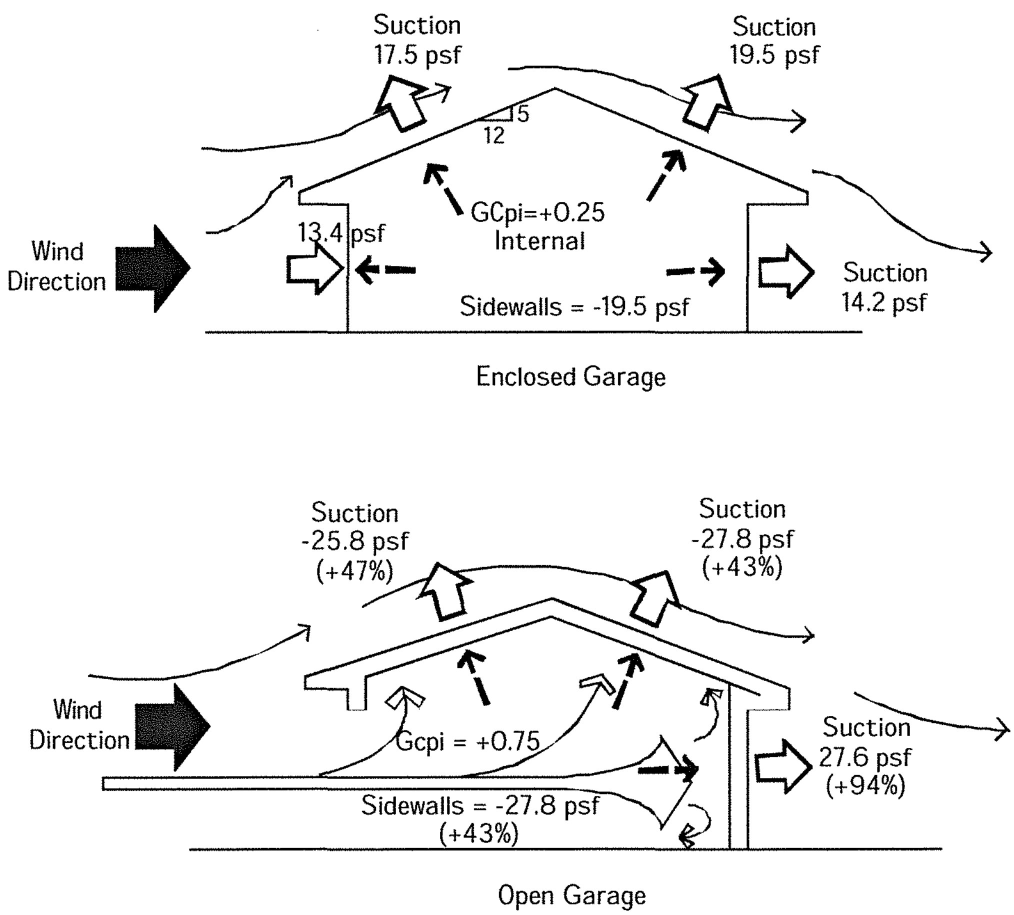


Figure 3.5a - Closed and Open Garage Wind Magnitudes

The garage is detached from the house and has dimensions 20' x 24' x 10' high and a 5 in 12 gable roof slope. The wind is coming towards the elevation that contains the garage door. It is clear when the garage door is open or breached that the wind enters the garage and produces an increased roof uplift resistance requirement of approximately 50%, and a leeward wall suction that increases stability requirements for sliding and overturning resistance by almost 100%.

Garage Doors

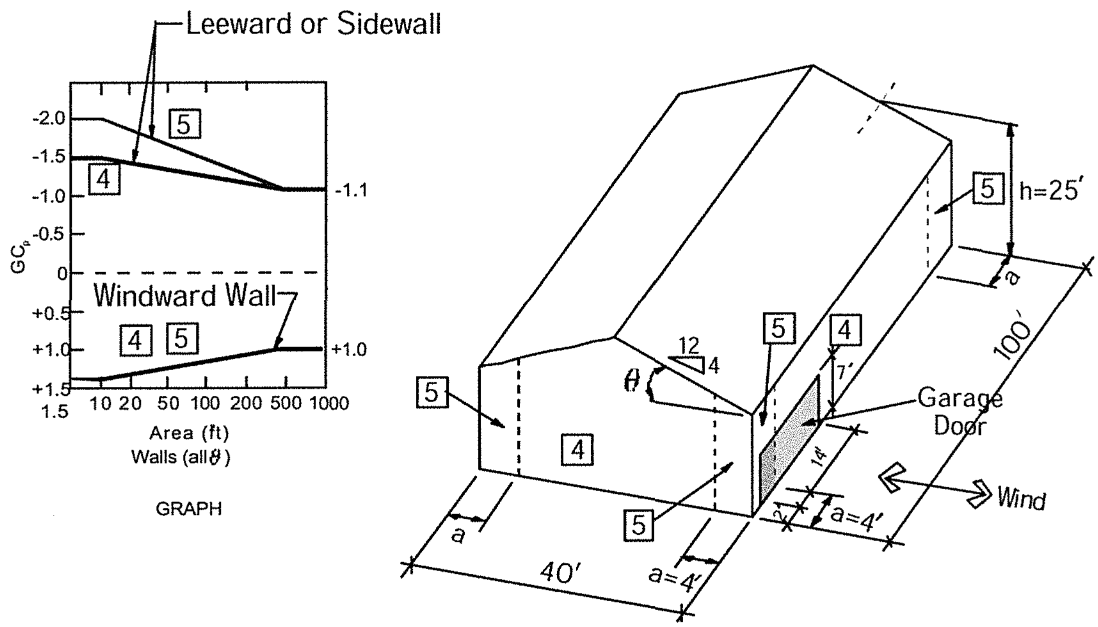
3. Garage Doors should be selected to resist component and cladding wind pressures based on the selected design wind speed. Perimeter framework and door anchors should also be designed for the selected wind speed.

   A. Garage door selection

      It has been stated above in item 2, that if a garage door is structurally damaged and permits a strong wind to enter the garage space, internal pressures subject the side walls and back wall to considerably higher pressures. Pressures against all internal surfaces will generally lead to the garage being lifted off its foundation, the roof being torn off or the walls being blown out. Further damage will occur to the residence if the garage is attached. Figure 8 in Chapter 1 of the *WMM* illustrates potential garage behavior.

      The selection of a garage door, sufficiently reinforced and tested, to resist a particular design wind speed is very important to the structural integrity of the garage and the attached residence. The area of an attached single garage door (7' x 9') or double garage door (7' x 16') is usually quite small in comparison to the total area of the elevation that contains the door. Components and Cladding wind pressures as defined in the ASCE 7-93 [3.1], and referenced by all the major codes, are thus used to account for localized higher wind pressures than typically used for Main Wind-Force Resisting systems.

      Figure 3.5b is taken from the ASCE 7-93 [3.2] to illustrate the variation in wind pressure near corners (region 5) and between corners (region 4) as discussed in detail at the beginning of Chapter 2. A sample two story residence is shown with dimensions and the garage door location. The door falls within both regions 4 and 5 as shown by the change in cross-hatching. Thus, 2' of the door falls in Region 5 and 14' falls in Region 4. Area averaging is used to arrive at a single negative and positive pressure value. A set of curves shown on the graph defines external pressure coefficients for both regions and for negative and positive pressure coefficients. Notice that the coefficients are largest for very small areas, such as garage doors. The 63 sq.ft. for a single garage door or the 112 sq.ft. for a double garage door have coefficients that are much greater than 1.0, indicating that the positive and negative pressures for Components and Cladding will be greater than Main Wind-Force Resisting System pressures.



Note a. The vertical scale denotes GC_p to be used, based on Exposure C.
Note b. The horizontal scale denotes the wind tributary area (A) in square feet.
Note c. External pressure coefficients for walls are permitted to be reduced by 10 percent where θ ≤ 10 degrees.
Note d. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
Note e. Each component shall be designed for maximum positive and negative pressures.
Note f. Notation:

a = 10 percent of minimum building width or 0.4*h*, whichever is smaller, but not less than either 4 percent of the minimum building width or 3 feet.
h = Mean roof height, in feet, except that the eave height is permitted to be utilized where θ ≤ 10 degrees.
θ = Roof slope from horizontal, in degrees.

Figure 3.5b - External Pressure Coefficients for Components and Cladding and Buildings with Mean Roof Height Less than or Equal to 60 Feet [3.2]

The "Door and Access Systems Manufacturers Association" (DASMA) has developed a *Garage Door Wind Load Guide* [3.3] that charts the Components and Cladding pressures on garage doors for wind speeds from 70 to 110 mph for all three model codes. The Guide covers one and two story structures with one or two car garages. Table 3.1 on the next page charts the positive and negative pressures in pounds per square foot based on the equations found in the 1996 BOCA Code [3.3] with accompanying notes, and use of Figure 3.5b.

Table 3.1

DASMA Garage Door Wind Load Guide

BOCA 1996 NBC

MEAN ROOF HEIGHT	DOOR SIZE	70 MPH	80 MPH	90 MPH	100 MPH	110 MPH
15 Feet Single Story	Single 9' x 7'	**14.7 psf*** -16.3	**19.2** -21.2	**24.2** -26.9	**29.9** -33.2	**36.2** -40.2
	Double 16" x 7'	**14.1** -15.3	**18.4** -20.0	**23.3** -25.3	**28.7** -31.3	**34.8** -37.8
25 Feet Double Story	Single 9' x 7'	**17.1** -18.9	**22.3** -24.7	**28.2** -31.3	**34.8** -38.6	**42.1** -46.7
	Double 16' x 7'	**16.4** -17.8	**21.4** -23.3	**27.1** -29.4	**33.4** -36.4	**40.4** -44.0
25 Feet Flat Roof	Commercial 10' x 10'	**15.1** -16.6	**19.8** -21.7	**25.0** -27.5	**30.9** -33.9	**37.4** -41.1

*Chart represents calculated "design" pressures (in psf, pounds per square foot) for the listed code standard representing common installations.

The local building authority may require testing to verify product performance. DASMA suggests that this product testing, if requires, be performed to ASTM-E330. Test conditions:

1. Product shall be tested to both negative and positive pressures. Product shall be installed to normal conditions (i.e. top rollers in track radius, proper counterbalancing, etc.).

2. Total test duration for each test direction shall be as follows:

   52 seconds (70 MPH), 45 seconds (80 MPH), 40 seconds (90 MPH), 36 seconds (100 MPH), and 33 seconds (110 MPH) at design pressure. Pressure equal to 1.5 times the design pressure to be included for 10 seconds in each test.

Product successfully passes the test if the door remains safely operable through full travel of the door track and recovers at least 75% of its maximum deflection. Sound engineering principles may be used to interlope or extrapolate test results to door sizes not specifically tested.

This guide is provided for reference purposes only. In all cases the local building authority is the sole and final determiner of the requirements and suitability of product.

Continued

Notes:
• Chart displays the design pressures calculated from the *1996 National Building Code* (NBC) for a typical single story residential house, a typical double story residential house, and a typical commercial building.
• The pressure requirements decrease for larger sizes and increase for smaller sizes.
• Negative pressures assume door has 2 feet of width in building's end zone.
• Garage doors evaluated as *Components and Cladding, Exposure C.*
• Garage doors evaluated as attached to enclosed buildings with an *Importance Factor of 1.0*.

Owners, contractors, architects and engineers can select a garage door from a manufacturer based on the values contained in Table 3.1 and required compliance with ASTM-E330 test results supplied by the manufacturer. As wind design speeds increase from 70 to 110 mph, so does the type and gauge of door reinforcement and hardware, plus the size and number of fasteners between door tracks and perimeter dimension lumber. Figures 3.5c and 3.5d illustrate typical details for a generic 90-mph rated door. Shown in these two figures are (1) the parts of the door where panels hinge; (2) the lightgage metal tracks and rollers; and (3) the connections between tracks and the wood framing. Also shown are the panel rib stiffeners.

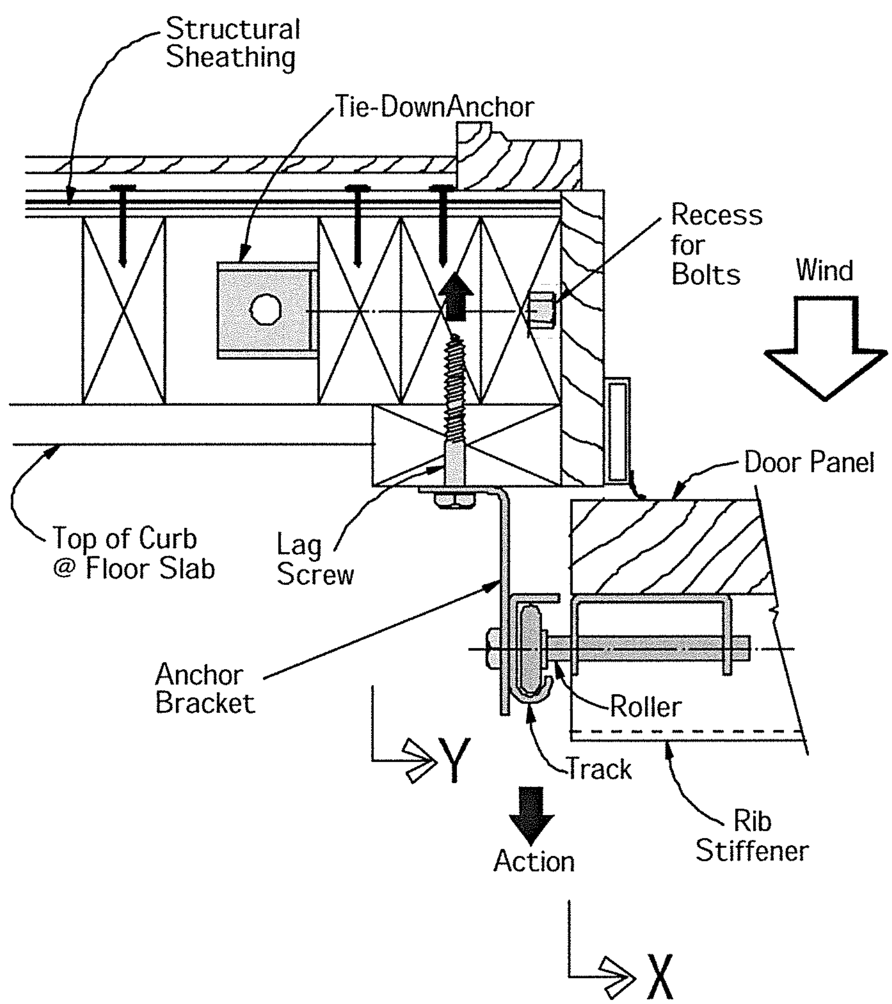


Figure 3.5c - Generic Garage Door Jamb Detail

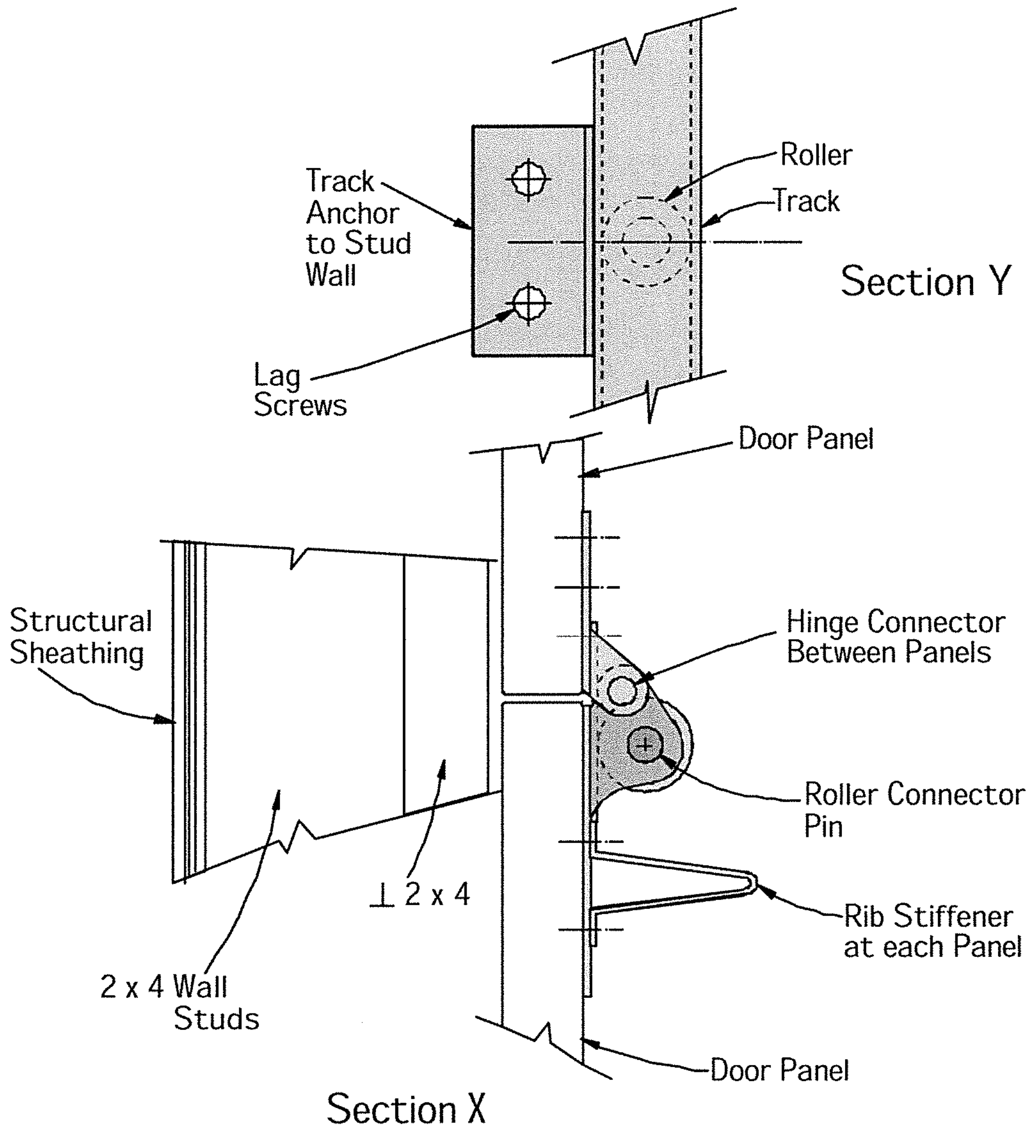


Figure 3.5d - Generic Garage Door Sections for Figure 3.5c

B. Door anchorage and wood frame requirements

Figures 3.5c and 3.5d basically familiarize the user with door terminology and how door components relate and fit together. It is thus possible to discuss how wind loads flow and transfer between the lightgage metal parts and the opening wood framing.

Wind load transfers between door components as shown in Figure 3.5e in the exploded perspective.

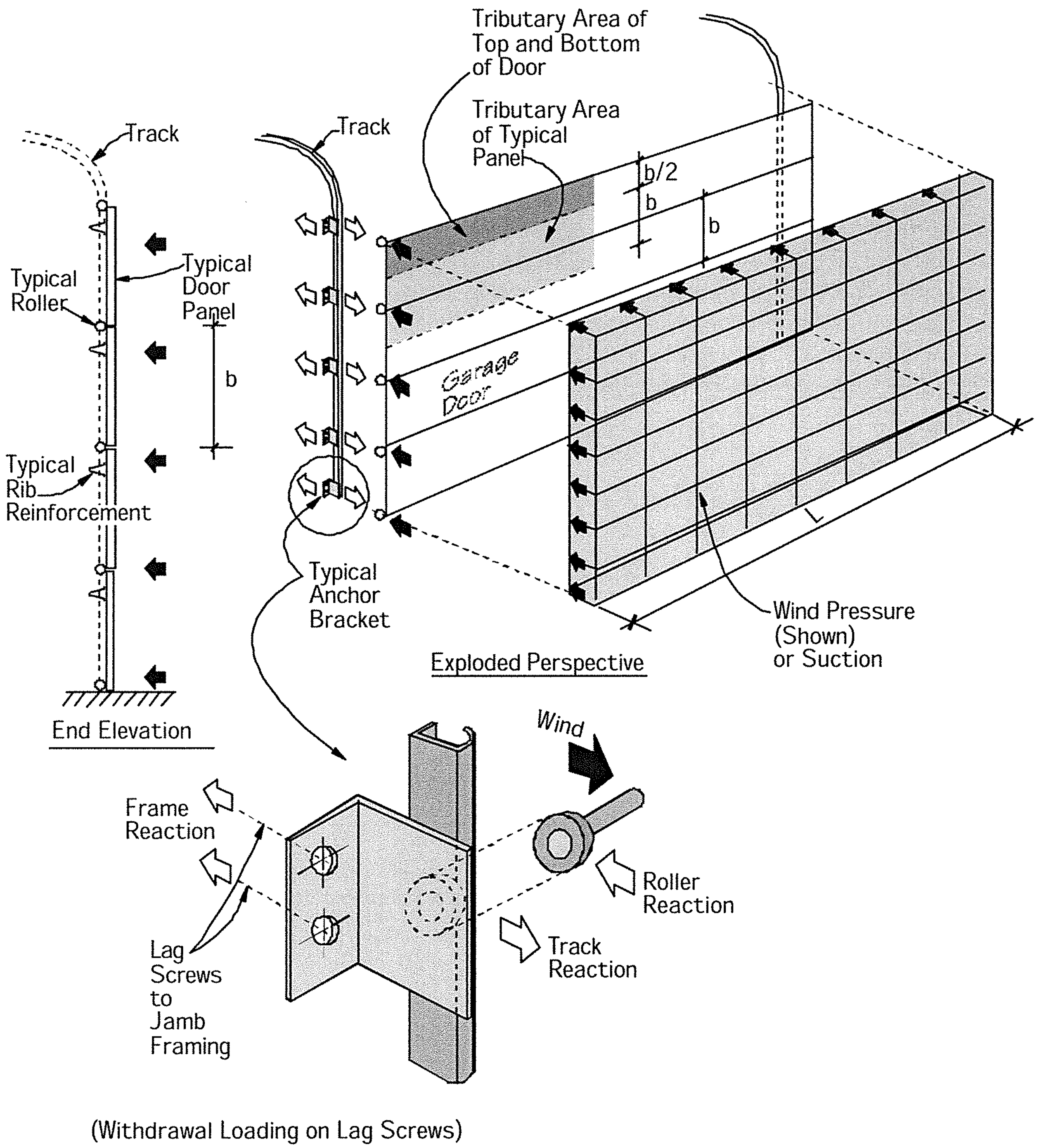


Figure 3.5e - Garage Door Load Transfer to Wood Frame

Wind positive pressure (or negative pressure: suction) pushes or pulls on the door panels. Due to the rib reinforcement behind each panel the wind load is transferred to the rollers that bear within the vertical metal tracks. These tracks are anchored to the wood frame (jamb) by means of anchor brackets that are lag screwed to the wood studs adjacent to the track. The enlarged view of a single bracket, track and roller illustrates the load transfer when positive wind pressure is applied to the door. The lag screws will be in *withdrawal* from the wood when positive pressure is against the door panels. Since lag bolts (or lag screws as another name) are threaded the NDS provides allowable withdrawal load capacities per inch of threaded length [3.4].

The lag screws will be in compression when a negative wind pressure is applied to the door panels. This is when the largest loading is on the wood studs that act as the jamb on each side of the door. The doubled or tripled studs will be subjected to bending stress and will deflect outward. The strength and stiffness of the wood studs at the door jamb are important to the strength of the whole door. A numeric example will be detailed in Chapter 5.

A tie-down anchor into the foundation is shown adjacent to the jamb. This is a recommended location for tie-downs. If the garage door is breached by the wind, direct uplift will necessitate a tie-down. Should the length of the wall either side of the garage door opening be sufficient to act as a shear wall, the tie-down will also serve the purpose of overturning stability - besides providing uplift resistance as part of the tension chain.

Drag Struts

4. Drag struts are a useful engineering device to transfer wind load between shear walls. Specifically, shear walls that are not aligned between levels or shear walls that do not extend the full width of the floor or roof diaphragm.

   This is an appropriate time to discuss the definition and use of **drag struts,** as shown in Figure 3.6a. Assume a wind direction from right to left as shown, producing wind positive pressure and wind negative pressure (suction) on the windward and leeward walls respectively. The roof diaphragm receives wind from half the height of the wall, producing illustrative values of 100 plf on the windward side of the diaphragm edge and 200 plf on the leeward side of the diaphragm edge for a total of 300 plf. This example uses two exterior shear walls and one interior shear wall with foundation walls below the three shear walls. The exterior shear walls "A" and "C" run the full width of the residence and receive the wind reaction directly from the diaphragm along their entire length. It is the interior shear wall "B" that does not receive the wind from the diaphragm directly, since it does not start from the windward side of the roof diaphragm and end at the leeward side of the roof diaphragm. It is most helpful to incorporate a beam in the plane of the roof that will stiffen the diaphragm and force a distribution of wind to the shear wall "B". **The**

function of a drag strut is to assist in directing and transferring wind load to a shear wall.

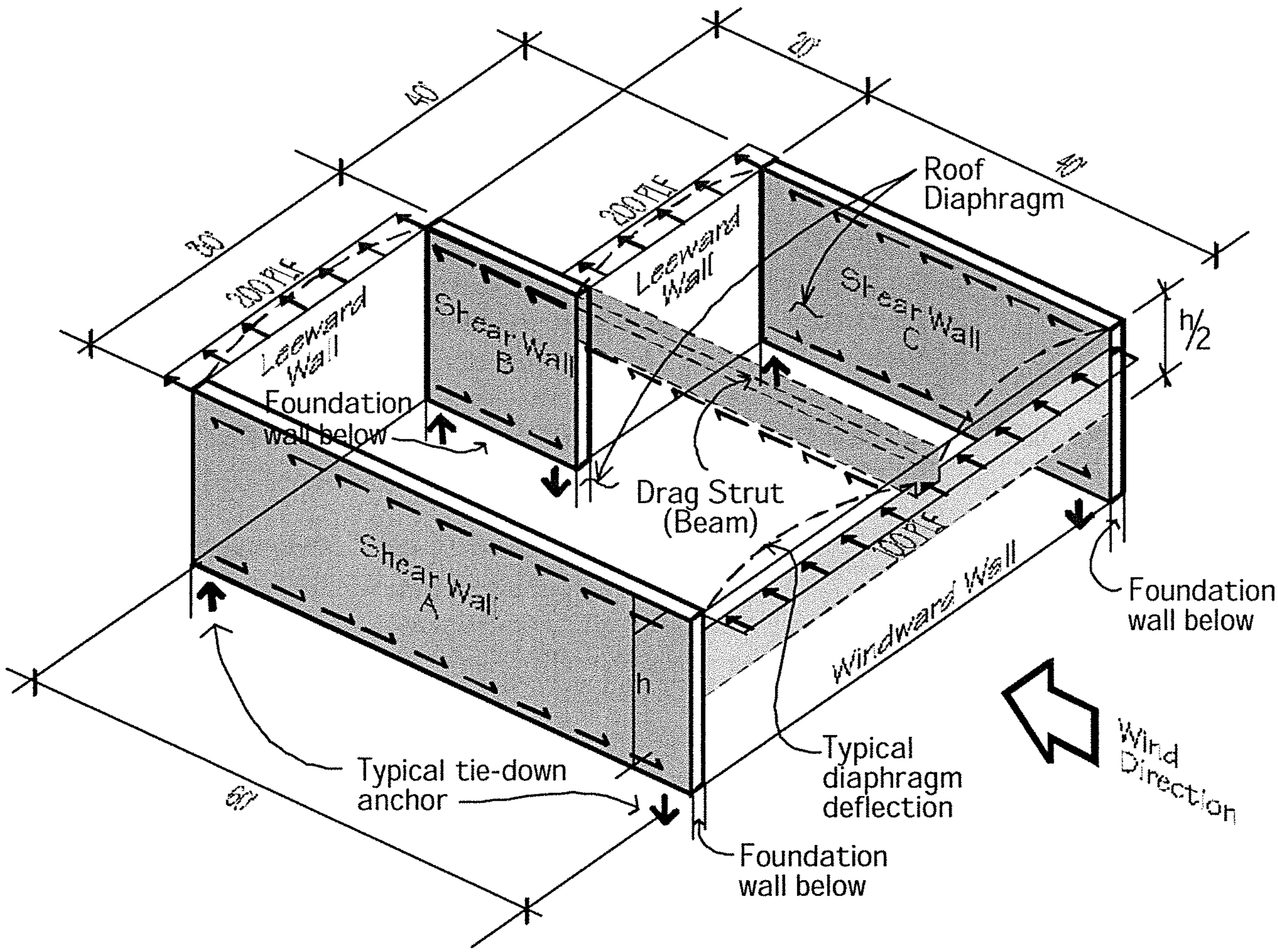


Figure 3.6a - Diaphragm Load Transfer in a One Story Residence

To mathematically illustrate the distribution of shear from a diaphragm to the shear walls, the magnitudes of windward and leeward wind on the diaphragm will be used. **Wood diaphragms are considered flexible diaphragms and thus are assumed to distribute load to the shear walls based on tributary widths.** Tributary widths are defined as half the distance between adjacent shear walls. Starting with the exterior shear wall "A", the total wind reaction at the wall will be a tributary width of half the distance to shear wall "B" multiplied by the total horizontal diaphragm wind load at the roof, i.e.30'/2 x (100plf + 200 plf) = 4500 pounds. Since shear wall "A" is 60 feet long, this means 4500 Lbs/60' = 75 plf of shear per lineal foot that must be transferred from the diaphragm and spread along the shear wall. Connections could be closely spaced nails or more widely spaced metal connectors. Exterior shear wall "C" similarly receives a wind reaction from the diaphragm of 40'/2 x 300 plf = 6000 pounds. Since the shear wall is 40 feet long, this means 6000Lbs/40' = 150 plf of shear per lineal foot that must be transferred from the diaphragm and spread along the shear wall. The interior shear wall "B" receives its reaction from a tributary width equal to half the distance to the shear wall "A" plus half the distance to the shear wall "C".

Figure 3.6b is an enlarged view of the wall plane that contains shear wall "B", the drag strut and the foundation wall below the shear wall.

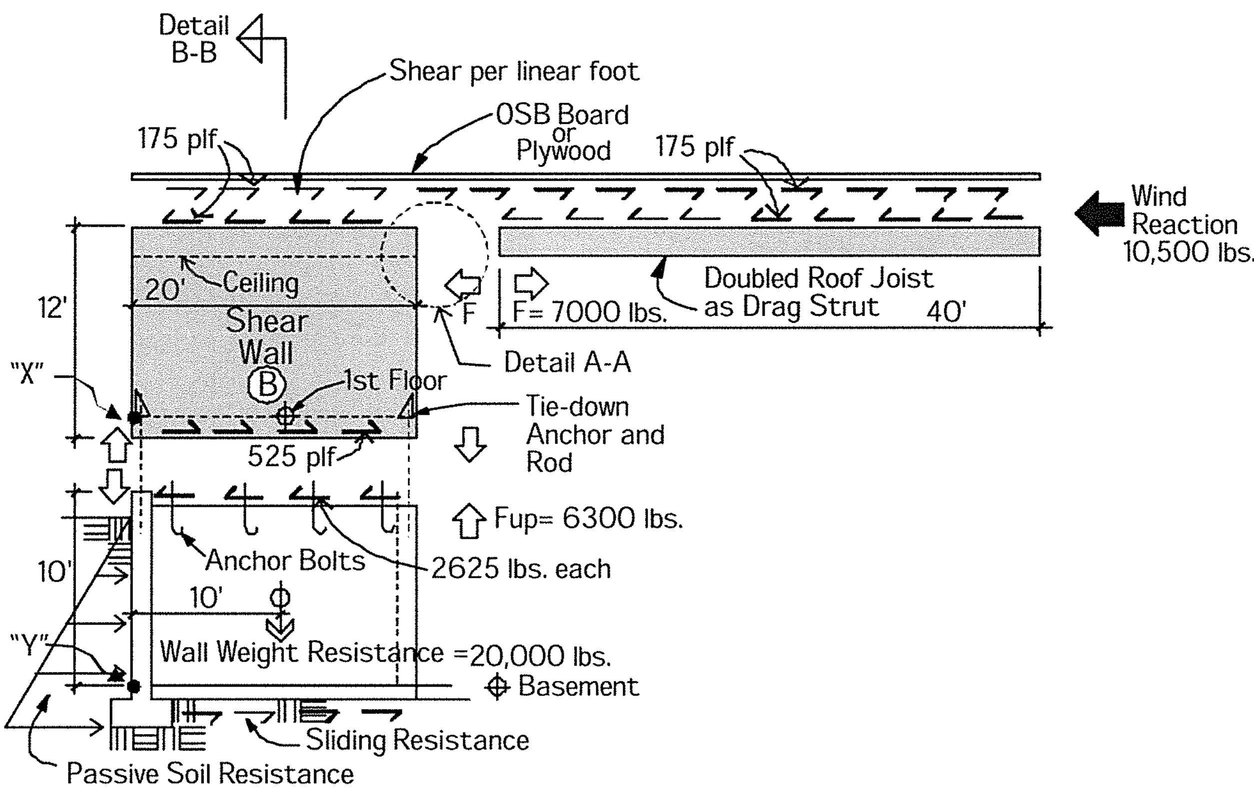


Figure 3.6b - Drag Strut Force Distribution To Shear Wall and Foundation

The roof sheathing is the diaphragm that receives the uniform horizontal wind force of 300 plf. The shear wall "B" must carry the total wind reaction of (30'/2 + 40'/2) x 300 plf = 10,500 pounds. The shear wall "B" plus the drag strut length must transfer 10,500 Lbs/60' = 175 plf of shear along the total length of 60 feet. The drag strut delivers 175plf x 40' = 7000 pounds to the shear wall and it must be connected to the shear wall to transfer that reaction. The shear to be transferred and spread along the length of the shear wall "B" is 10,500 lbs/20' = 525 plf. This is considerably more shear per foot than the exterior shear walls must receive and transfer from the diaphragm. **Interior shear walls generally carry more shear per foot because of their increased tributary width**.

The foundation wall that exists under shear wall "B" receives the 525plf and resists that shear by the anchor bolts. Assume 4 anchor bolts exist, thus each anchor bolt must resist 10,500 lbs./4 = 2625 lbs. The *sliding* of the foundation due to the 10,500 pounds must be resisted by friction between the concrete and the soil plus the passive soil block shown. The *overturning* tendency of the shear wall produces an uplift force at the end of the shear wall. This force is determined by taking moments about point "X". The uplift value is (12' x 10500 lbs.)/20' = 6300 pounds of tension, conservatively ignoring the shear wall weight and any roof dead load.

This example has the roof framing spanning parallel with the shear walls and very little roof gravity dead load is transferred to the shear walls. The tie-down anchor, near the end of the shear wall, must transfer the tension to the foundation wall. The weight of the concrete foundation must in turn resist the tie-down force from overturning the concrete shear wall. The basement wall tends to rotate (overturn) about the point "Y". Summing moments about point "Y" produces 10' x 20000 lbs. – 20' x 6300 lbs. = -6280 ft-lbs. The negative answer proves there is sufficient dead load in the foundation to resist the overturning and the uplift. Various details are available to connect a drag strut to a shear wall.

Figure 3.6c illustrates two possible approaches when dimension lumber framing is used. The upper detail occurs when the drag strut beam extends continuous across the width of the roof and actually rests on top of the shear wall. The lower illustration is a detail when the drag strut beam stops, or is discontinuous, at the shear wall. It becomes essential to tie the two elements together and provide a tension resistance capability. Note the placement of an angle to support the gravity load carried by the reaction at the end of the drag strut beam. The tension straps resist the tension pull.

Conditions where foundations *do not* exist under shear walls, and/or first floor shear walls *do not* exist under second floor shear walls, are very complicated and uneconomical, although sometimes unavoidable. These conditions commonly occur at interior walls. Figure 3.7a illustrates both conditions occurring on the same two-story residence, and shows the need for drag struts at each level to complete the transfer of load between roof and floor diaphragms to the offset shear walls at each level. These shear walls lie in the same overall building plane, but not directly under one another. The horizontal wind reactions to each shear wall are calculated based on tributary area just as for the previous example, resulting in the concentrated wind reaction arrows shown at each level. The shear walls do not extend across the full width of the building at each level and it is required that drag struts force the wind reactions over to each of the shear walls.

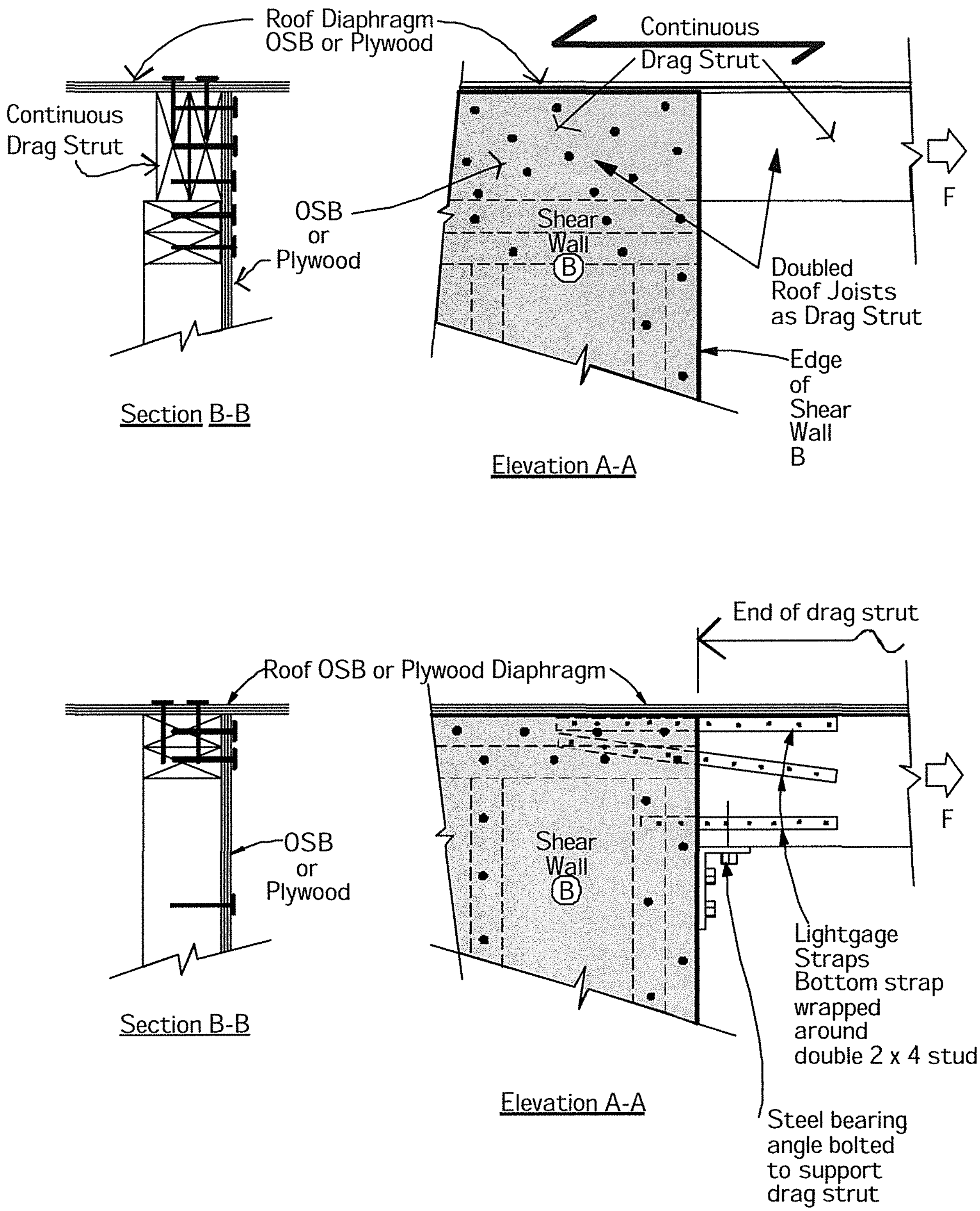


Figure 3.6c-Alternate Drag Strut Connections to Shear Wall

Conditions where foundations *do not* exist under shear walls, and/or first floor shear walls *do not* exist under second floor shear walls, are very complicated and uneconomical, although sometimes unavoidable. These conditions commonly occur at interior walls. Figure 3.7a illustrates both conditions occurring on the same two-story residence, and shows the need for drag struts at each level to complete the transfer of load between roof and floor diaphragms to the offset shear walls at each level. These shear walls lie in the same overall building plane, but not directly under one another. The horizontal wind reactions to each shear wall are calculated based on tributary area just as for the previous example, resulting in the concentrated wind reaction arrows shown at each level. The shear walls do not extend across the full width of the building at each level and it is required that drag struts force the wind reactions over to each of the shear walls.

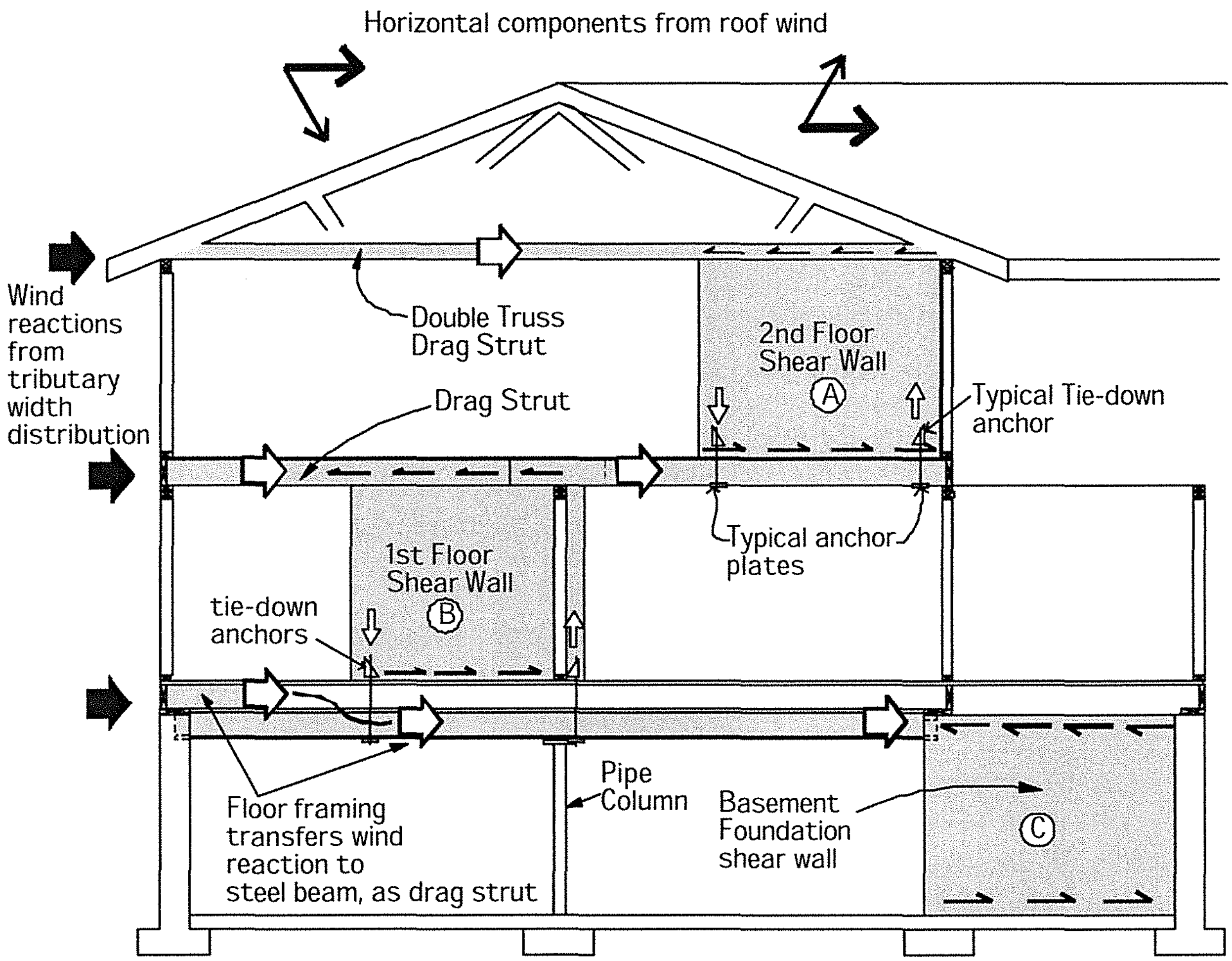


Figure 3.7a - Section at Interior Offset Shear Walls (within same plane)

The bottom chords of doubled trusses are used as the drag strut beam at the roof level to transfer the wind reaction from the roof plane diaphragm to the top of the second floor shear wall labeled "A". Connections capable of making the horizontal shear transfer will require nails, lag screws or metal connectors.

The shear force at the bottom of the second floor shear wall adds to the second floor level horizontal wind reaction and must transfer the entire reaction uniformly along the second floor drag strut beam, which is a doubled "I" joist. The second floor drag strut joists sends its total shear force to the top of the first floor shear wall, labeled "B". The drag strut must connect to the shear wall to transfer its total shear force and the shear wall "B" must transfer the total sum of all the wind reactions above it uniformly across its length. Thus, the shear force transferred to the first floor shear wall is larger than the shear force transferred to the second floor shear wall. Connections capable of making these horizontal shear transfers include nails, lag screws or metal connectors.

The horizontal shear force at the bottom of shear wall "B" must add to the wind reaction, and again transfer along the first floor doubled "I" joist drag strut uniformly in shear. This time a steel beam is attached to the doubled "I" joist and takes over as the drag strut to transfer the sum of all the wind reactions at the roof, second floor and first floor to the foundation wall "C". This scenario completes the horizontal sliding resistance.

Figure 3.7b is a section through all of the shear walls and drag struts of Figure 3.7a with all the horizontal shear connections satisfied by the use of nails. This is followed by Figure 3.7c, which is a side elevation of shear walls "A" and "B" to help clarify the connections shown in the section of Figure 3.7b.

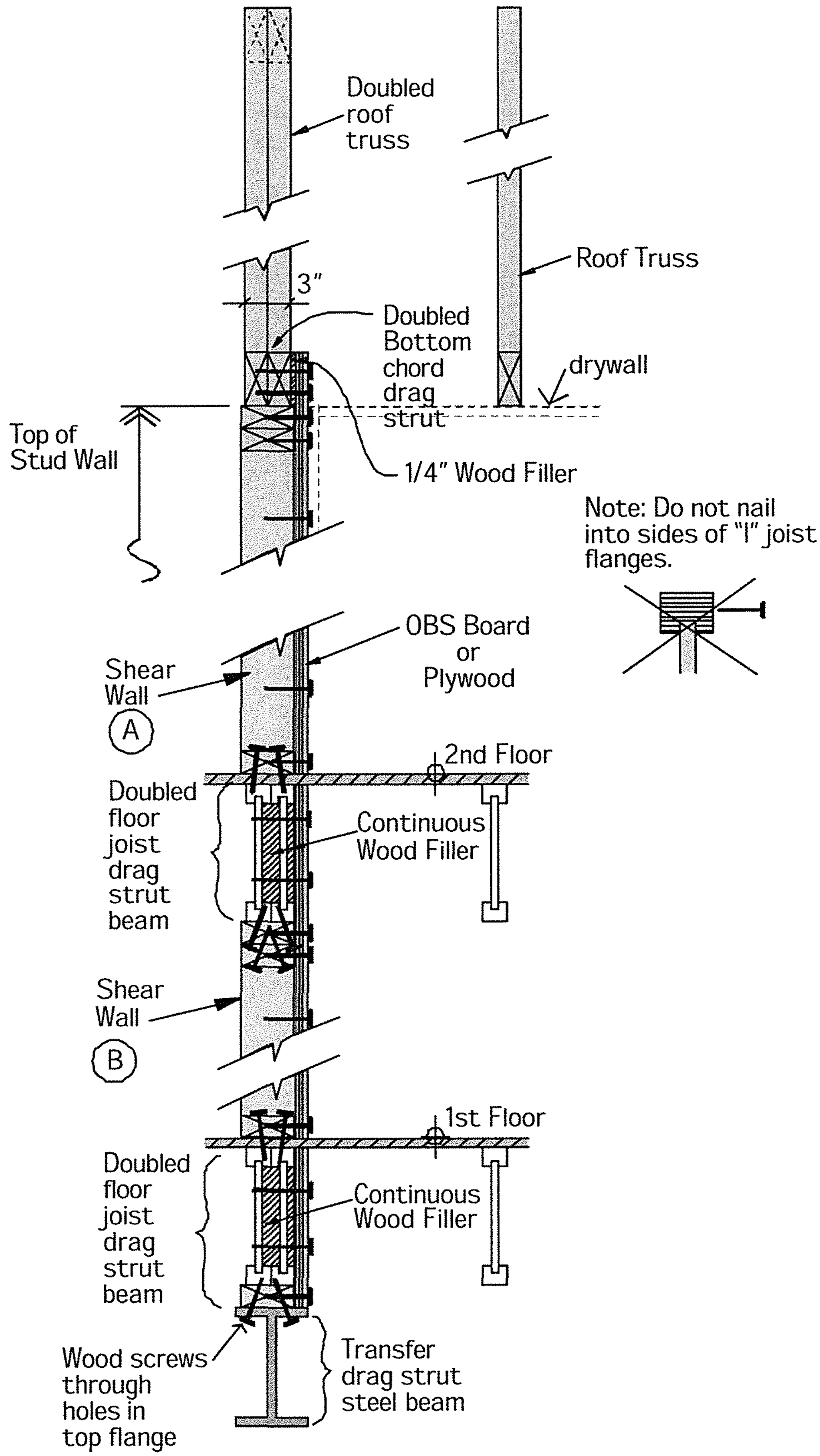


Figure 3.7b - Section Through Shear Walls

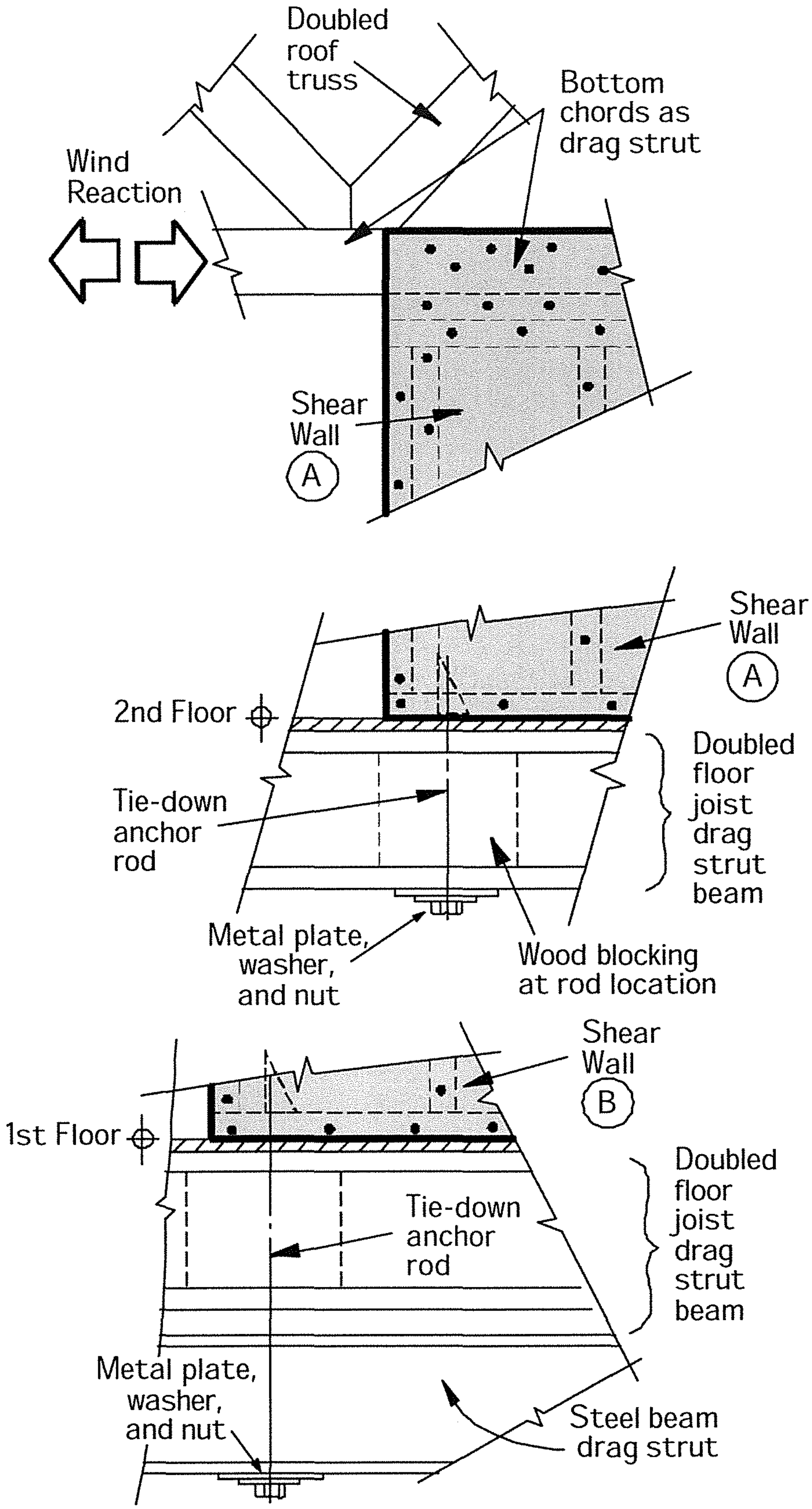


Figure 3.7c Side Elevation of Shear Wall:

Each shear wall in Figure 3.7a also requires overturning tie-down anchors at the ends of each wall. These anchors resemble Figure 14 and Figure 16 of Chapter 7 in the *WMM*. The doubled joists and the steel beam are thus used in bending to accommodate the concentrated tie-down forces, combined with the axial tension or compression force for drag strut behavior. This double duty accounts for why the trusses and joists are doubled.

Figure 3.8 is a perspective of just the first floor construction and foundation; between the steel beam drag strut to the in-plane concrete foundation walls that are used as shear walls as shown in Figure 3.7a. The sequence of load transfer to the steel beam is described in detail above for Figures 3.7a, 3.7b and 3.7c. The horizontal shear transfer and the added components of wind load related to the first floor shear wall down to the concrete foundation can be summarized as described below:

(1) Place two floor-framing joists directly under the shear wall "B";

(2) Connect shear wall "B" to the doubled floor joists below with lag screws, spikes or metal connectors to transfer the horizontal wind reactions similar to that shown in Figure 3.7b for nails only;

(3) Place a steel beam below the doubled joists to span between the in-plane foundation walls that run parallel to the shear wall. This step is not always necessary, particularly if the doubled wood joists are short enough in span to reach the foundation walls and adequate in strength to act as the drag strut and tie-down support;

(4) Connect the double joists (if a steel beam is used) to the steel beam, most likely with screws as shown in Figure 3.7b;

(5) Weld steel channels to the steel beam at each parallel foundation wall, and then use expansion bolts to tie the steel channels to the foundation wall;

(6) If the span between walls is too long, a steel pipe column and footing will also be required to reduce the steel beam span, as shown in Figure 3.8.

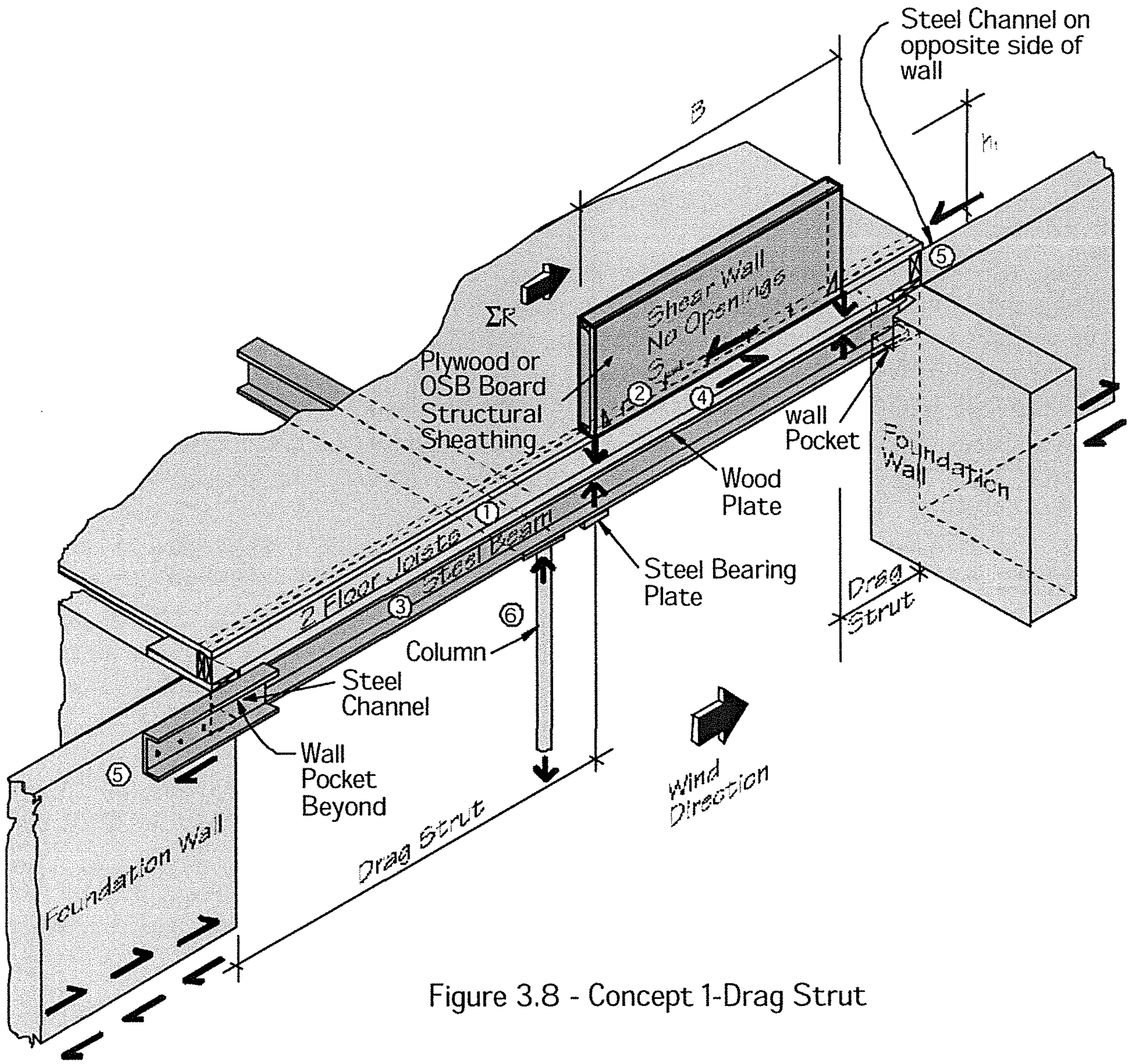


Figure 3.8 - Concept 1-Drag Strut

Shear Wall Design

5. Select the traditional shear wall analysis method or the perforated shear wall empirical method.

   A. Traditional Shear Wall Analysis Method

   Traditionally, only wall sections fully sheathed with structural panels were used as shear walls. Sections of wall containing door and window openings were excluded from consideration. Figure 3.9a illustrates numerous types of wall openings and their influence on shear wall appropriateness, based on the "traditional" method. Very small openings were ignored, if they did not interfere with the continuity of the shear wall; thus the entire wall could be used as a shear wall. Vertical, long and narrow openings in a wall split the potential shear wall into two separate but smaller shear wall segments. Large openings, such as doors, large windows, or obviously garage doors also split the wall into solid segments between these openings.

   Structural panels above and below openings are not considered to contribute to the overall strength and stiffness of the wall. Solid planes of walls less than 4'-0" long are questionable for use as shear walls, since the overturning anchorage force becomes very large and economical to tie the shear wall to the foundation.

   Refer to Figure 9 in Chapter 5 of the *WMM* for examples of relative tie-down forces dependent on shear wall proportions based on the traditional method. Note that if the shear wall becomes too narrow it will become more flexible and horizontally move and rotate much to easily. The traditional method uses tie-downs at the ends of each solid wall segment to resist overturning, as shown in Figure 3.9a.

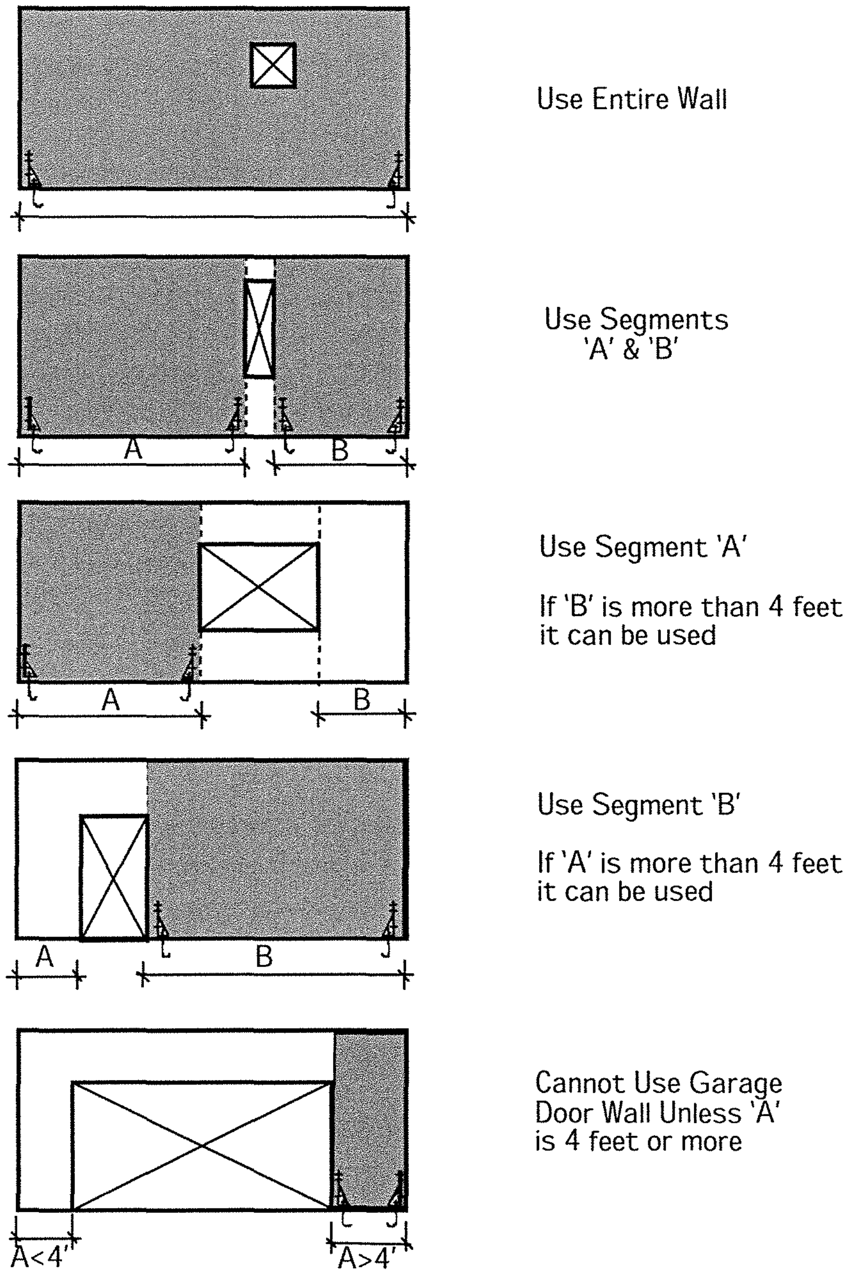
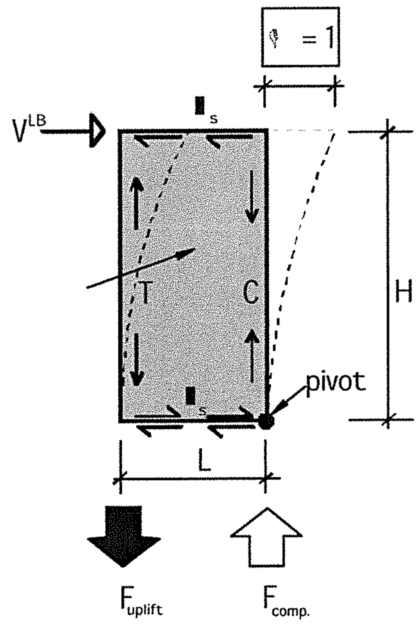


Figure 3.9a - Traditional Approach to Shear Walls with Openings

Chapter 5 of the *WMM* gives a complete overview of "traditional" shear wall behavior and the necessary connections to resist sliding and overturning. The equilibrium of a shear wall and the resulting equations are of general use in the examples that will appear in Chapter 5. Figure 3.9b illustrates the forces applied to a shear wall, and develops the shear and overturning equations. It also illustrates, as did Chapter 5 in the *WMM*, that a square wall segment is most appropriate as a shear wall, since it cuts the overturning force in half and increases the wall stiffness by six times.



$$V^{LB} = \blacksquare_s^{plf} \sphericalangle L^{ft} \quad \text{or} \quad \boxed{\blacksquare_s^{plf} = \frac{V^{LB}}{L^{ft}}}$$

sum moments about pivot:

$$\blacklozenge M = 0$$

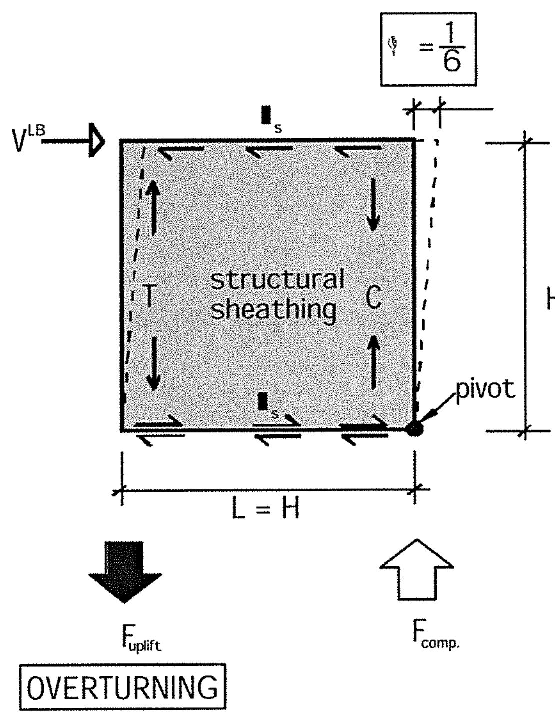
$$V * H - F_{uplift} * L$$

$$V * H = F_{uplift} * L$$

ॐ $$\boxed{F_{uplift} = \frac{V * H}{L}}$$

if $L = 4'\text{-}0$ & $H = 8\text{-}0'$

$$\boxed{F_{uplift} = 2V}$$



sum moments about pivot:

$$\blacklozenge M = 0$$

$$V * H = F_{uplift} * H$$

$$\boxed{F_{uplift} = V}$$

if $L = 8\text{-}'0$ & $H = 8'\text{-}0$

Panel 6 times as stiff
as the 4′ panel

Figure 3.9b - Equilibrium of a Shear Wall

B. Perforated Shear Wall Empirical Method

Considerable research has been dedicated to achieving more economy in shear wall design. Design optimization studies incorporating the structural panels above and below openings found that these normally ignored structural panels are a significant contributor to the overall wall shear strength and stiffness. This approach also required tie-downs only at the corners or ends of a shear wall elevation. A significant number of static and cyclic tests have verified the empirical equations of the perforated shear wall method [3.5].

Figure 3.9c illustrates the perforated empirical method, when compared to the shear wall examples in Figure 3.9a.

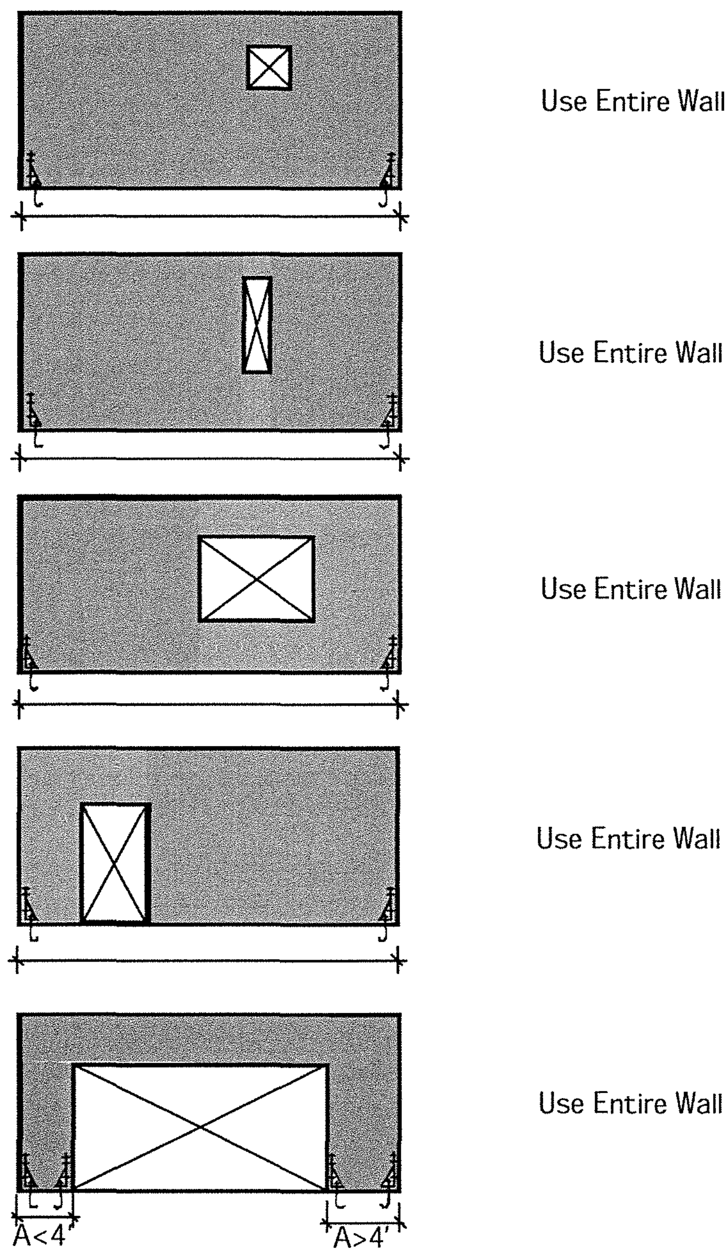


Figure 3.9c - Perforated Shear Wall Method

Note the significant reduction in anchor tie-downs, and the incorporation of the whole wall for shear wall performance. This method now appears in the *Standard Building Code* [3.6] and the *Wood Frame Construction Manual for One and Two Family Dwellings* [3.7]. The example in Chapter 5 will use both methods for comparison purposes.

The perforated shear wall design approach, based on finding the *required shear wall length*, uses the following steps:

1. Find the required length of shear wall sheathed with structural panels for the full height of the wall. This is based on the traditional method;

2. Use the empirical equations of the perforated shear wall method to find the **length adjustment factor** (R_β) to reduce the strength of the traditional fully sheathed shear wall (found in step "1") and compensate for the presence of openings by increasing the shear wall length.

The perforated shear wall design approach, based on finding *the allowable shear strength* of a perforated wall length, uses the following steps:

1. Determine the allowable shear (plf) based on the structural panel thickness and nailing size and spacing from Table B.28 in Appendix **B**. Determine the total allowable shear load (lbs.) by multiplying the allowable shear by the length of fully sheathed segments of a given building exterior wall elevation. This is based on the traditional method;

2. Use the maximum opening height and the ratio of fully sheathed walls to the total length of wall with openings to find a **shear capacity adjustment factor**. Multiply the adjustment factor by the allowable shear found in step one and multiplied by the sum of solid structurally paneled wall segments to determine the reduced allowable shear strength (lbs.) of the perforated wall. This will account for the presence of openings and permit tie-down anchors to be placed only at the ends of the wall.

Notes:

- When designing for a given wind load, shear walls by the perforated shear wall method will be longer, but generally have a reduced number of overturning tie-downs compared to the traditional method's solid-sheathed shear wall segments.

- Shear walls designed by the perforated shear wall method instead of the traditional method will have lower ultimate shear capacity and less stiffness.

- Plywood or OSB sheathing is appropriate as the structural panels used on the exterior of the wall. Interior drywall can add to the allowable shear capacity of the wall.

- The perforated shear wall method uses only two tie-down anchors located at each end of the elevation, rather than the traditional method that uses tie-downs at the ends of each solid wall segment. This method will generally prove to be more economical.

Shear Walls Containing Garage Doors

6. Provide more than 4 feet of solid shear wall on either side of an elevation that contains a garage door opening.

The unique case of a wall containing a garage door should be reviewed here. Two car garage doors are usually 16 feet long, and front garage wall widths are generally 20 feet long. It is clear that for the 20 foot front garage wall, only 2'-0" remains for symmetry of the elevation with respect to the door. These 2'-0" sections are extremely flexible, and will produce very large tie-down anchorage forces, if used as shear walls. Since garage foundation walls are approximately 4'-0" deep or less, to bear below the frost level of a given geographic location, there is usually insufficient foundation wall and footing weight to resist a large tie-down force.

Figure 3.10 illustrates a typical plan view of a garage that is attached to a residence.

A. Traditional Approach

Consider first that the entire garage projects in front of the house, as indicated by the black walls extending past the rear of the garage. For a wind direction "A" it would be advisable to use wall 2 as one shear wall and purposely increase the width of the garage to approximately 26 feet, rather than the typical 20 feet. This permits offsetting the garage door to the right as shown, so that a large segment of wall 4 is created that can work as a stiff shear wall. This creates a non-symmetrical front elevation of the garage. The additional space in the garage can be used for storage or a work shop area. This is an economical approach to creating a structural paneled shear wall at location 4. This approach is valid even if the house exterior wall is partially set back from the front of the garage as shown by position 4B. When the house front elevation is in alignment with the garage wall, the house wall 4A can become the shear wall. This direct alignment allows a return to a 20-foot wide garage elevation, although the additional garage storage space is then eliminated.

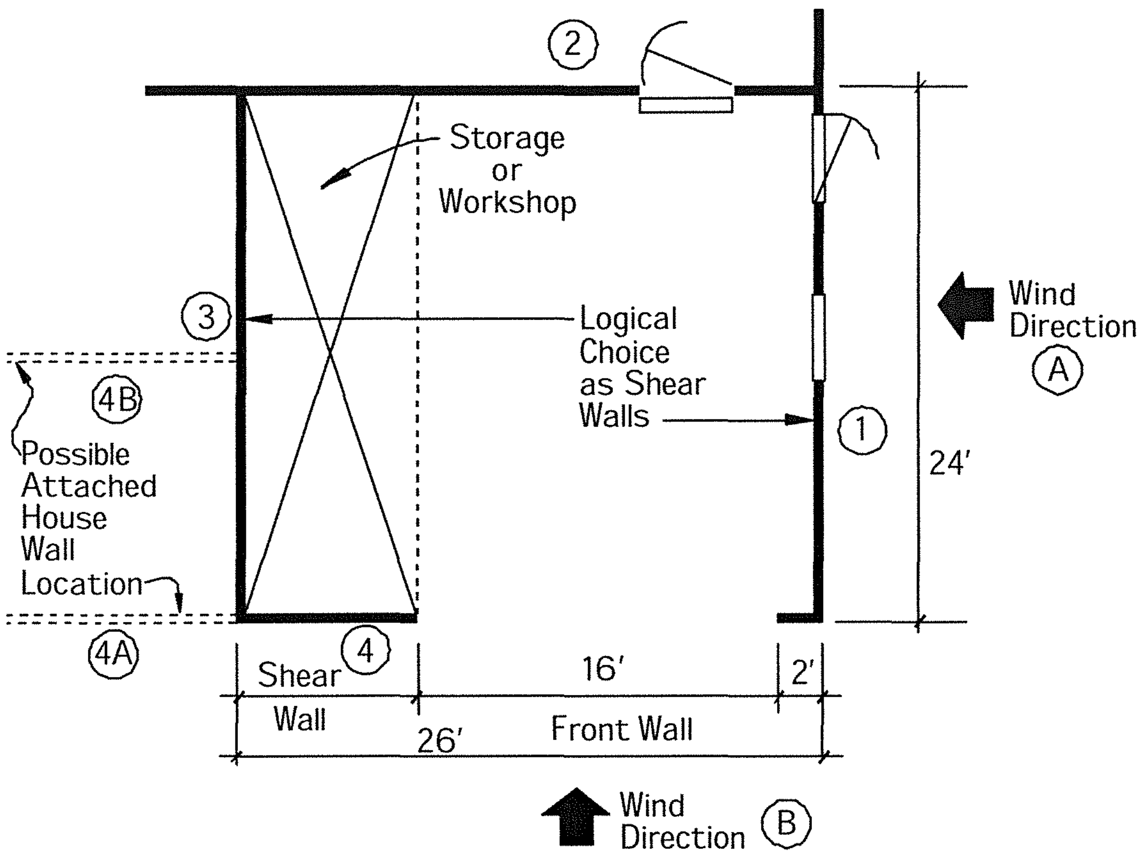


Figure 3.10 - Improved Garage Front Wall Shear Resistance

Sometimes it is easier to achieve lateral stability at the front elevation of a garage by means of a steel frame around the garage door. This approach would avoid using the 2'-0" wall segments on each side of the door as shear walls, and instead uses the walls to hide the columns of the steel frame. Figure 3.11 illustrates this concept for a situation when the house is set back from the front of the garage and there is no need for additional garage space. Either wide flange shapes or rectangular tubes are used for the steel frame. The steel beam acts as the garage door header beam. If the beam supports brick above the opening then a tube should be used to avoid twisting of the beam. The column base detail is shown for a tubular solution that fits within the 3-1/2" stud dimension. It is common for a structural engineer to assist an architect when steel frames are required.

B. Perforated Wall Method

Even if the empirical method is used, the solid structural paneling either side of the garage door should be at east 4'-0", when the garage wall is not in alignment with the house wall. This wall dimension is necessary to achieve any acceptable movement control in the plane of the wall for wind blowing in direction A. If the minimum 4'-0" paneled wall is achieved on each side, then its possible that the entire wall plane can be used, as shown in Figure 3.9c. Acceptable sway control might then be achieved.

If the small door opening has structural paneling above shear wall 2, then the entire wall may be treated as a perforated shear wall. Anchored at each end of the elevation.

Wall 1 is likely to still perform as a shear wall despite the two window openings by the perforated wall method, when wind blows in the direction B. Structural paneling would be needed above and below the openings.

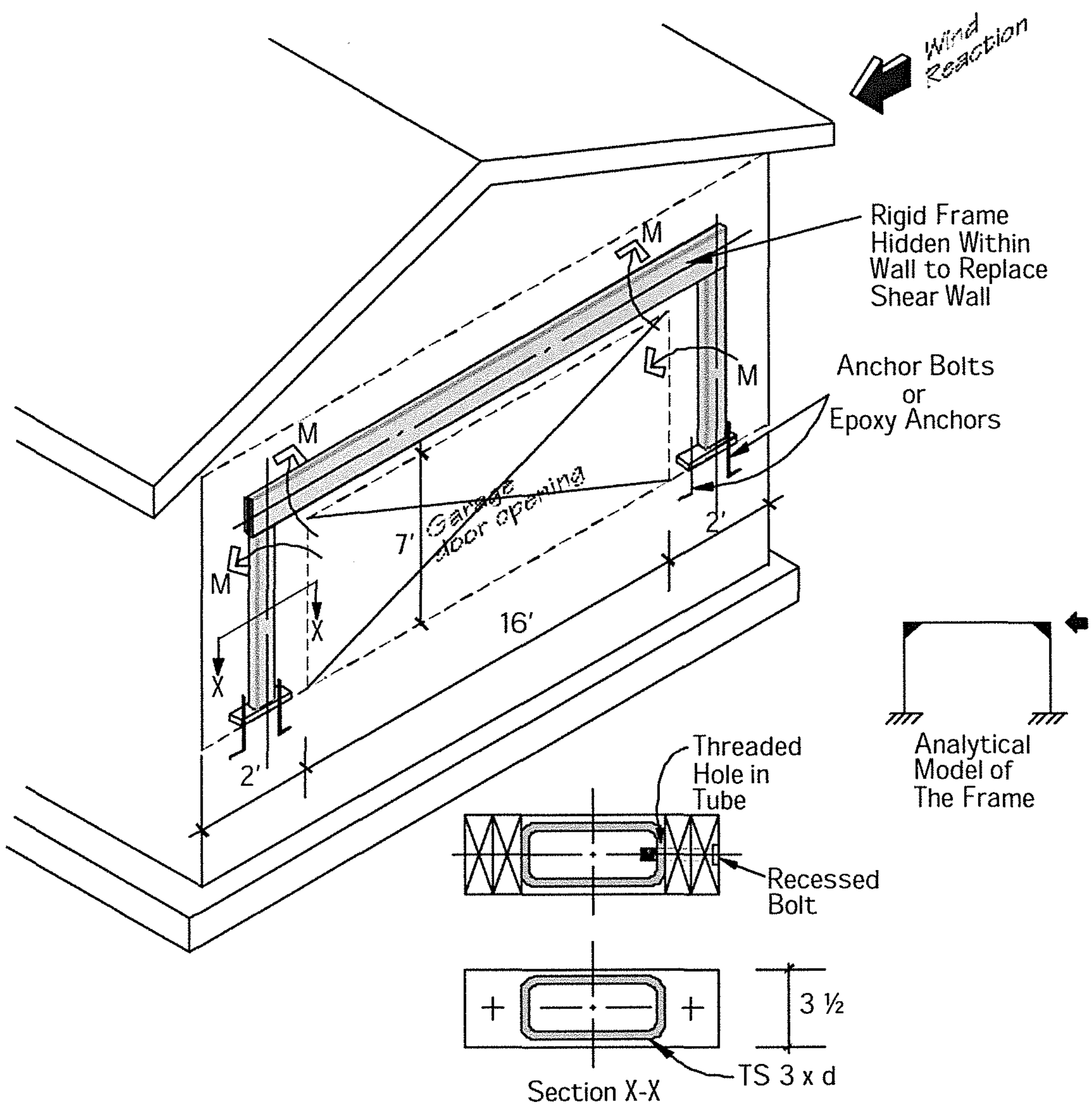
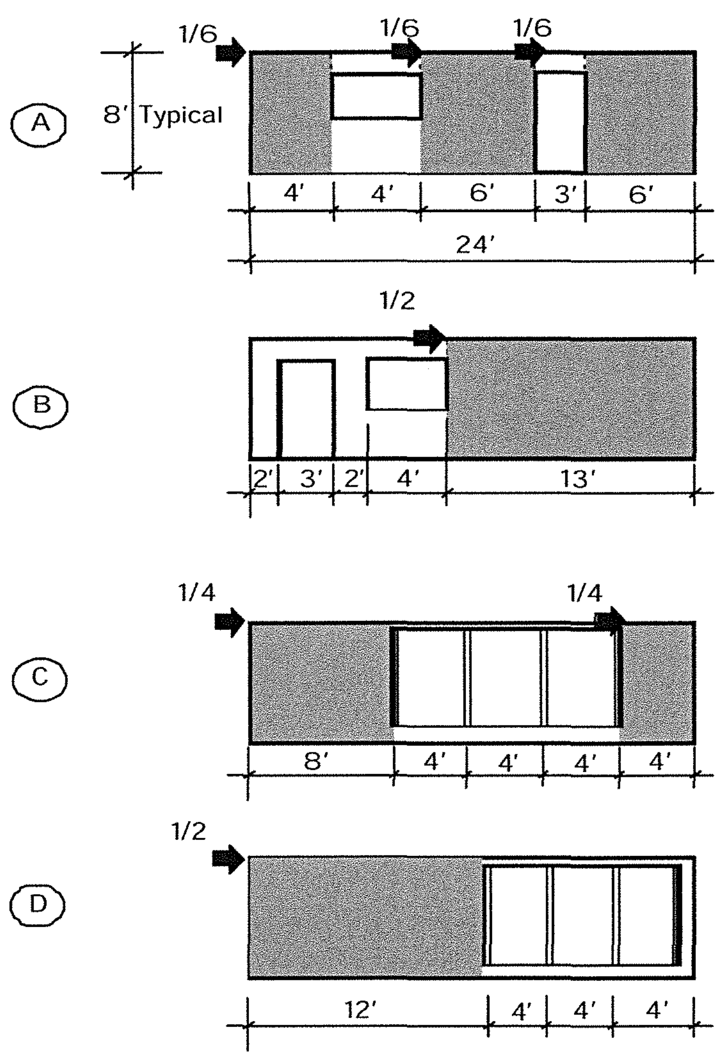


Figure 3.11 - Steel Frame Hidden Within Garage Wall

Placement of Doors and Windows in Shear Walls

7. Place doors and windows to maximize potential shear wall length.

Regardless of shear wall design method selected in item #4, the shear walls must have sufficient stiffness; that is, they control the amount of sideways movement of the residence subjected to horizontal wind load forces. The longer the shear walls the stiffer the shear walls, and the smaller will be their horizontal movement. Also, longer shear walls are less likely to overturn. Thus, the required tie-down force will be smaller and the anchorage detail will be more economical. It may not always be possible to shift the placement of windows and doors. However, if it is possible without disrupting the design of the room behind the wall, it can greatly influence the shear wall structural stiffness and the required tie-down force magnitude. This concept was presented in Chapter 5 and illustrated in Figure 9 of the *WMM*. Figure 3.12 continues the same relative stiffness and relative tie-down force proportions as found in the WMM, but with an emphasis on the position of window and door openings. Figure 3.12 specifically relates to the traditional shear wall method.



	Relative Stiffness	Relative Tie-down Force L	M	R
A	7	.333	.22	.22
B	14			0.3
C	6.4	0.25		0.5
D	11.8	0.33		

Figure 3.12 - Elevation Planning

Wall "A" in Figure 3.12 is an elevation with a door and window placement that has been designed for one elevation of a residence. Notice that the 24-foot long wall ends up with two openings that break the wall into 3 short segments that could be used as shear walls. If it were possible to rearrange the two openings off to one side, as shown in elevation "B", a significant increase in stiffness would occur. This would reduce the movement of the residence subjected to a horizontal wind reaction by 100%. This requires a design trade-off or a compromise in the visual expression to be made for the sake of creating a stiffer and more wind resistant home without increasing the cost. A similar example is the elevation of wall "C", where three large windows are adjacent to one another, resulting in two potential shear wall segments. Again, if it is feasible to move the trio of windows to one end of the wall, as shown in Elevation "D", the stiffness again doubles. Thus, the sideways movement under wind load would be cut in half. Tie-downs would be used at the ends of each full height structurally paneled segment. This improves the shear wall stiffness overall.

Use of the Perforated Shear Wall Method may allow elevations "A" and "C" not to have to change, but they will be more flexible. Tie-downs will be located at the ends of the elevation only.

Avoid Wall Offsets

8. Avoid short offsets over the length of exterior walls.

Offsets, as shown in the partial foundation plan in Figure 3.13a, may be considered an architectural feature for the interest of the superstructure elevations above. The elevations will add shadow lines and visual interest, plus it will avoid making a wall seem overly long.

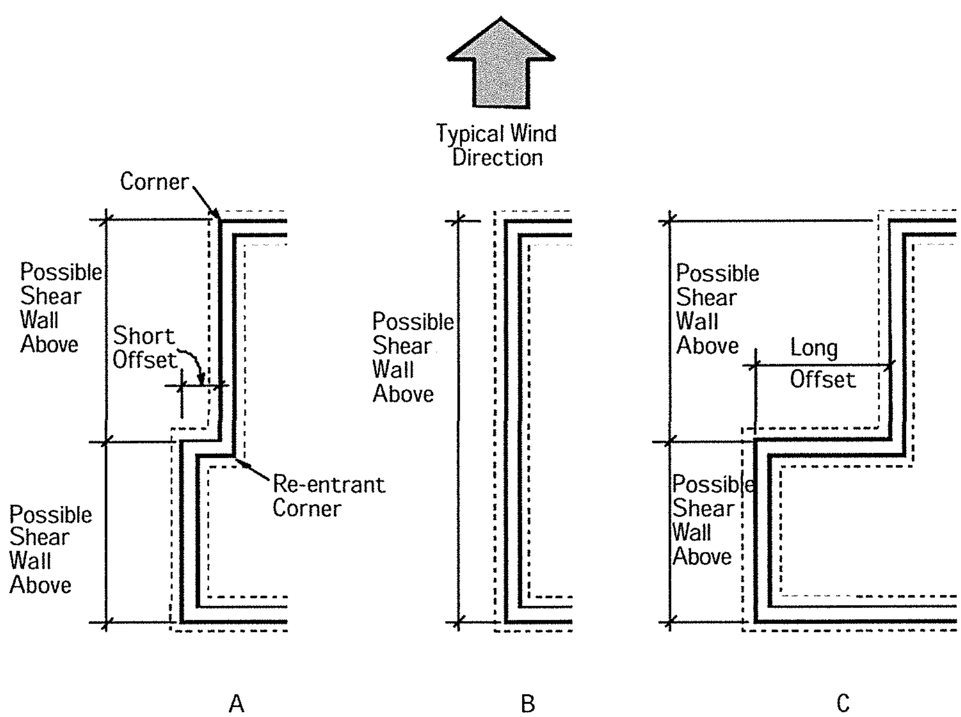


Figure 3.13 - Partial Foundation Plan Views

These minor offsets, usually in the 1 to 3 foot range, do little to increase the size of an interior room and result in a foundation that is more expensive to install. Most contractors will agree that every exterior corner or re-entrant corner is an added cost. This information leads to the following conclusion: **offsets of one to three feet mean very little to interior space, but do complicate and make the foundation more costly**. Offsets should be at least four feet or more to influence interior space and make the added cost of the foundation worth considering, as shown in Figure 3.13c.

Structurally, short or long offsets will break a shear wall into two separate segments and reduce its overall stiffness as previously shown for wall openings in Figures 3.9a and 3.12. Since short offsets add cost for very little gain, avoid them to maximize wall length and thus maximize strength and stiffness as a potential shear wall as shown in Figure 3.13b. Architecturally the straight wall has less visual interest but structurally produces the best potential shear walls for stiffness and overturning resistance. Somewhere there must be a balance struck between design and structures, particularly as it will influence cost. Drag struts, although adding cost, may be used to force short offset walls to transfer shear as though both wall segments were in the same plane. This technique is less likely to be structurally effective for large offsets.

Unusual situations can occur in custom house designs where cathedral ceilings and two story spaces are part of the design. One such complicated design is shown in Figure 3.14. It involves a two-story residence with a cathedral ceiling for a two-story space, adjacent to two floors with a flat ceiling and roof trusses. The length of the house and the magnitude of the wind forces requires using an interior shear wall along with two exterior walls as shear walls. The interior shear wall is on one side of an open sided stair, that separates the cathedral space from the two-story spaces. Only the crosshatched wood stud wall runs from the roof down to the basement floor. The length of the shear wall is insufficient to transfer the build-up of wind shear at the basement level.

The solution is to engage a drag strut from the top of the basement wall to the garage foundation wall, which is in perfect alignment. Thus, the garage foundation wall is made to share the wind shear with the stair wall. The exterior foundation wall of the basement is offset by 16 inches, and would require a diagonal drag strut and significantly more engineering. Although it is easier to introduce a second drag strut and balance the shear, the offset makes this impractical. This is a complicated solution, which requires an engineering review.

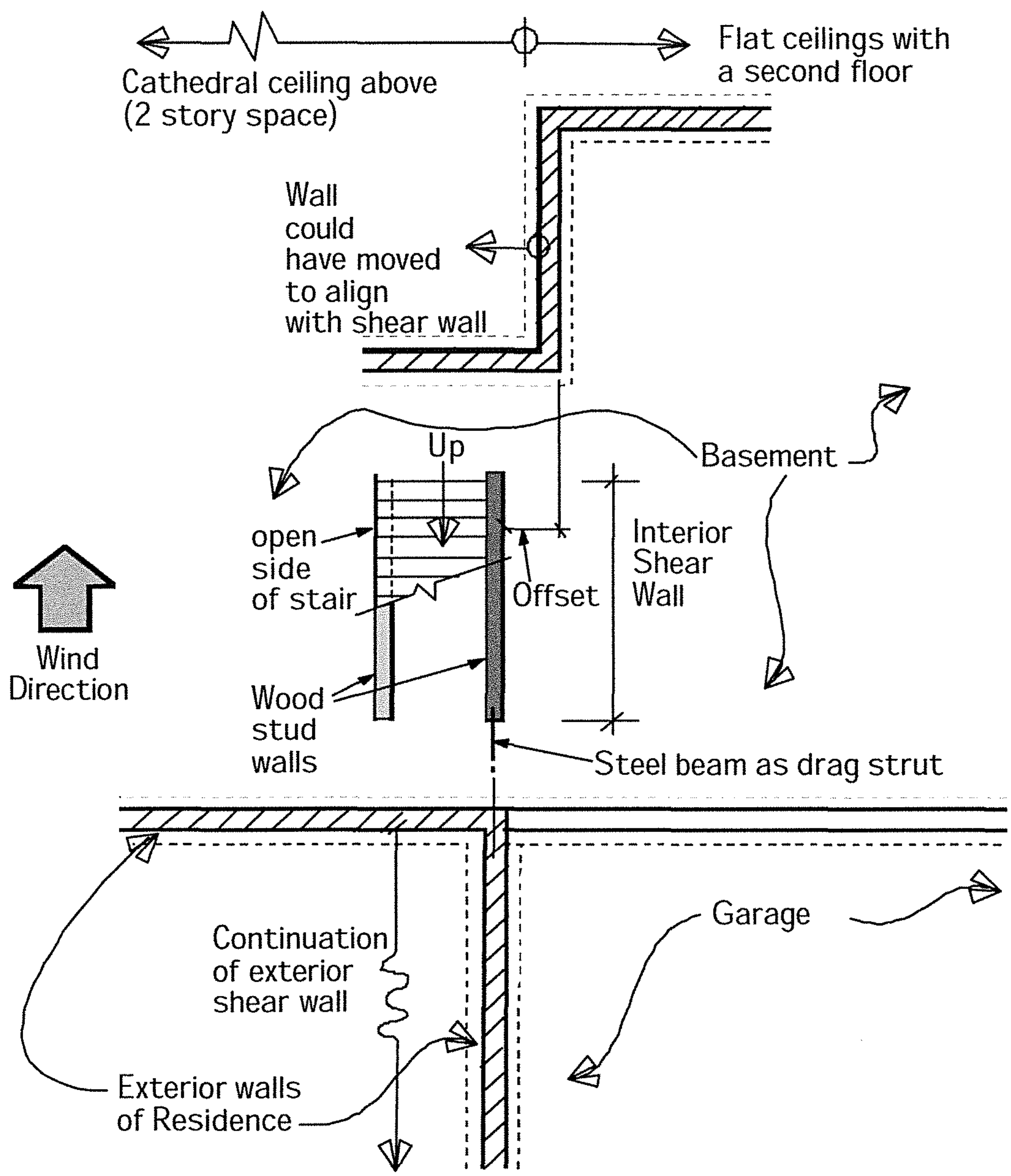


Figure 3.14 - Partial Foundation Plan

Figure 3.15 illustrates another type of offset situation. A recessed side entry to a residence also breaks the potential exterior shear wall length into two pieces, even if a straight foundation wall exists below. Add to this the introduction of a masonry fireplace bounded by large windows on both sides. Now the rear exterior portion of wall is unlikely to be of any shear wall value. This leaves the front portion of wall as the only exterior shear wall of use.

It is clear from this discussion that not all walls can be shear walls, and that such issues as openings or recesses, or even offsets will lessen the shear wall length and therefore its strength and stiffness.

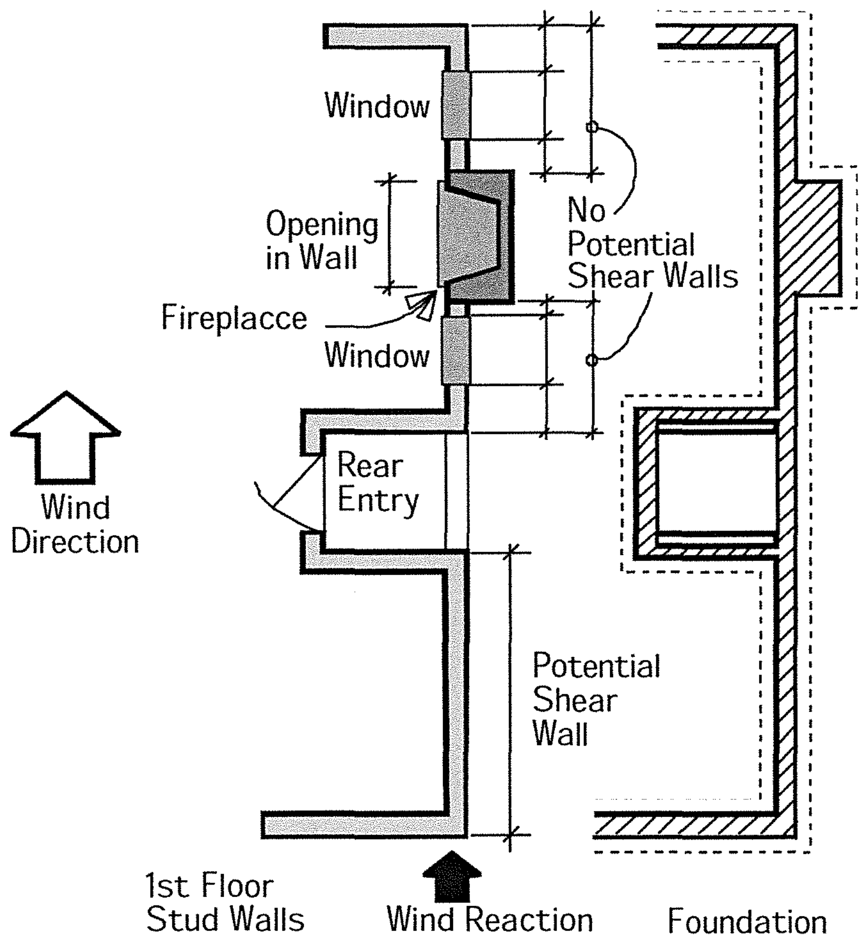


Figure 3.15 - Partial Plans with Offsets

Shear Walls that are Bearing Walls

9. Exterior walls that are bearing walls are the best choice for shear walls.

The added dead load of floors and/or roofs framing into exterior walls improves their resistance against overturning and reduces or eliminates the anchorage requirements at the ends of the shear wall. A reduced anchor size and a reduction in the number of bolts lessen the cost of the shear wall. Figure 3.16 illustrates a portion of an exterior bearing wall of a two-story residence. The hip roof is framed with rafters that bear on the exterior shear wall. The first and second floor framing joists bear on the exterior shear wall. Only a central portion of the total wall length can be utilized as a shear wall due to openings that are not shown. Drag strut members will be used to transfer the wind reaction to the shear wall. The entire dead load adds resistance to the overturning tendency of the shear wall.

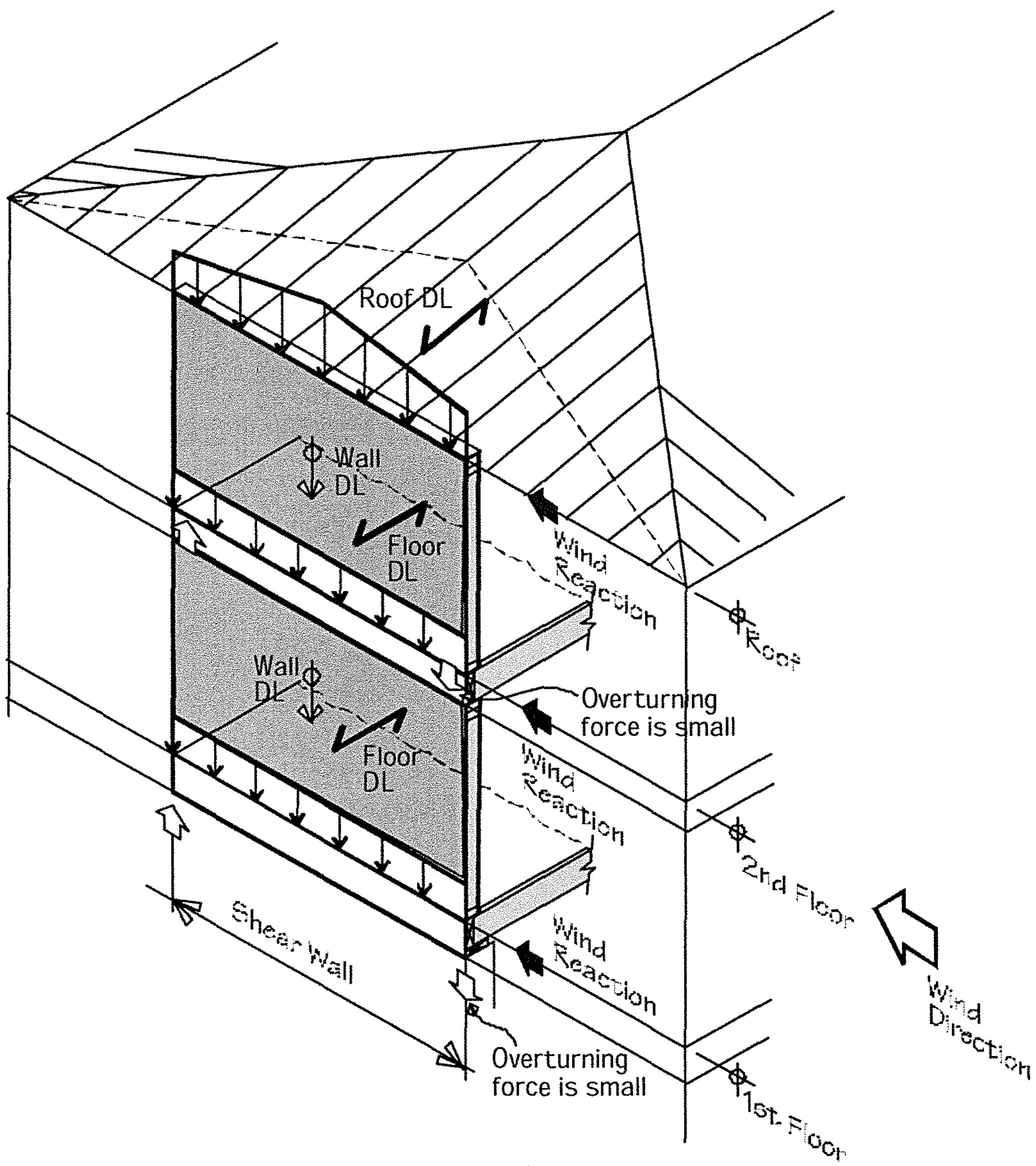


Figure 3.16 - Bearing Wall Used as Shear Wall

Figure 3.17 illustrates a non-bearing wall used as a shear wall. Only a small portion of the roof dead load from the short rafters is added to the weight of the wall itself. The floor joists run parallel to the shear wall and do not add any dead load to the walls. Non-bearing walls will most likely require a greater anchorage at the ends of the shear wall. This requires more uplift resistance from the foundation concrete, longer anchorage rods and more bolts into the ends of the shear wall – ultimately more cost.

It is logical that the floor and roof framing should span parallel to the long direction of the residence, since the long direction receives a greater amount of wind and places larger wind reactions on the short direction shear walls, these shear walls will be loadbearing. However, it is most common that floor and roof framing spans perpendicular to the long direction of the residence, which makes the long exterior walls bearing walls.

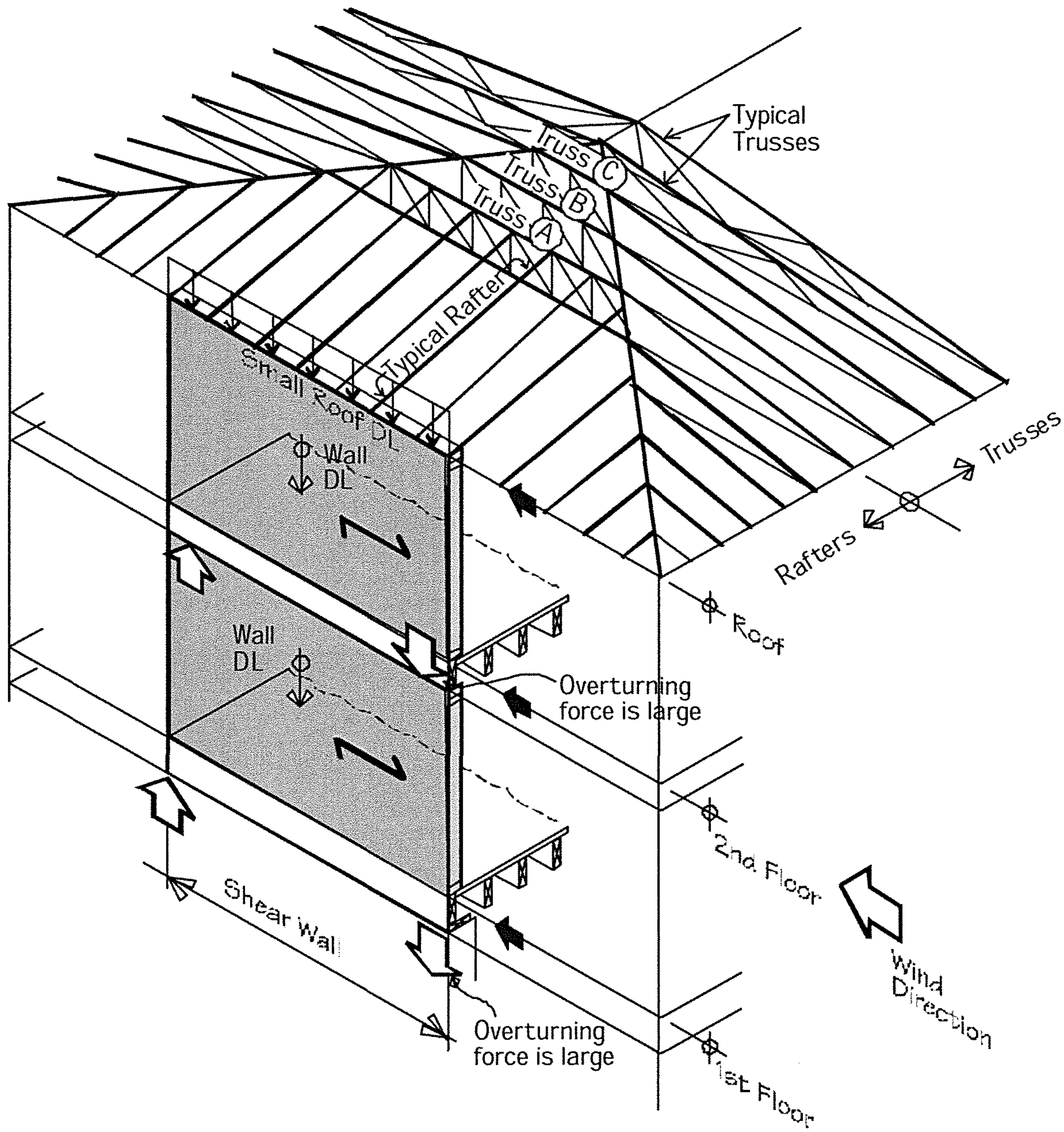


Figure 3.17 - Non-bearing Wall as a Shear Wall

End Wall Design

10. End wall Design for Gable and Hip Roofs.

Figure 3.18a compares the end wall behavior for a single story gable roof (B) building and a single story hip roof (A) building.

Gable Roof

End stud walls of a gable roof, that create an interior cathedral ceiling, will be very flexible in platform framing. The double top plate location is where the weakness occurs. The failure mechanism at the double top plate when subjected to a wind load is shown in location "B" of Figure 3.18a.

Figure 3 in Chapter 6 of the *WMM* illustrates A potential solution utilizing 4x4 posts to stiffen the wall. This solution resembles balloon framing, as the 4x4's extend continuous from the floor to the roof plane. Another solution is to introduce a 4-foot offset in plan near the highest portion of the gable. This will significantly stiffen the wall. Cathedral spaces are generally used for family rooms or living rooms. A design consideration is to introduce a fireplace in either of these types of spaces. Should either of these rooms contain an end wall, place the fireplace within the end wall. This technique will also stiffen the wall.

Figure 21 in Chapter 7 of the *WMM* illustrates a detail to stiffen the gable end wall at the double top plate when flat bottom roof trusses are used and a horizontal ceiling plane exists.

Hip Roof

A hip roof, utilizing rafters to create an interior cathedral space, presents a structural improvement to the end wall problem since the jack rafters provide a restraint at the wall's double top plate. The stud wall will bend when subjected to a wind load, rather than rotate without bending at the double top plate mechanism. This is illustrated in "A" of Figure 3.18a.

A hip roof is the structurally preferred roof form for wind resistance. This is true since it avoids any additional bracing to stabilize the end walls. Gable frame end walls can be made to perform adequately if bracing, such as that shown in Figure 3.18b. The sheathed hip roof form also acts as a 3-dimensional shell and creates a stiff roof shape for wind resistance.

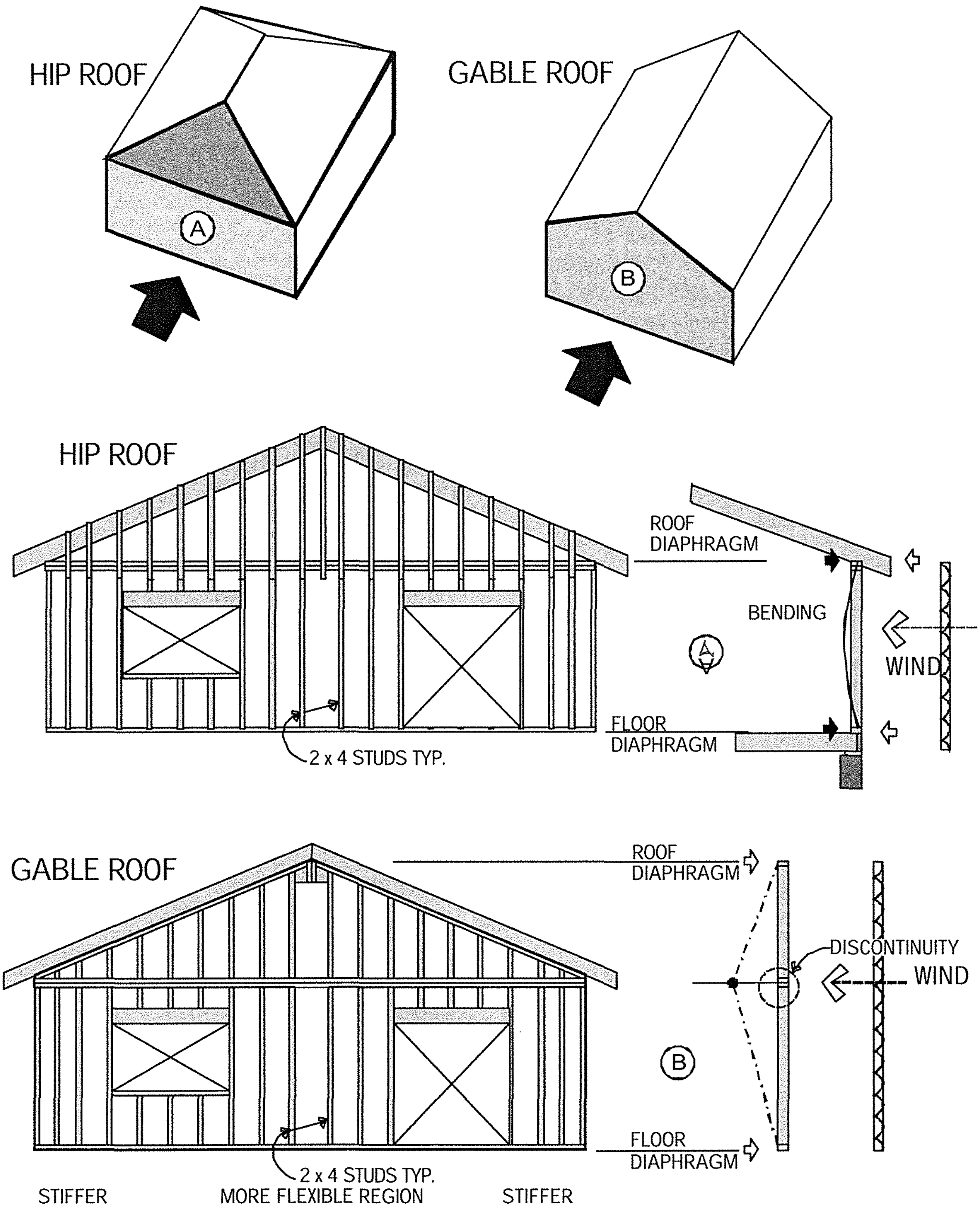


Figure 3.18a - Stability-Wind on Gable and Hip Roof End Walls

Two story spaces, that include an exterior end wall, present much the same problem of stability as shown in Figure 3.18a for gable roofs. Figure 3.18b illustrates this situation. Solutions are similar to those described above: offsets in plan, 4x4 posts, or if design warrants a fireplace.

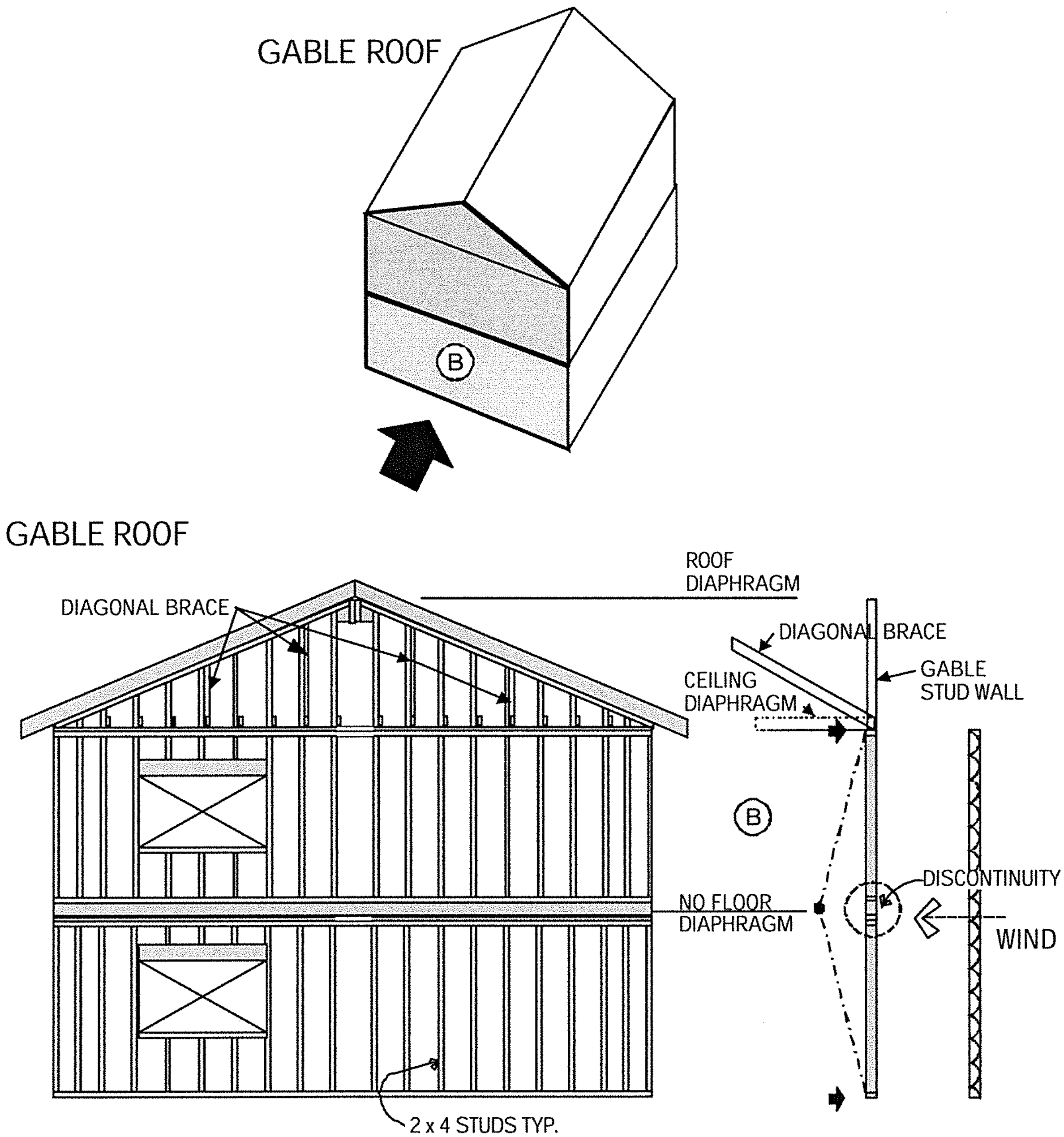


Figure 3.18b - Stability - Wind on Two-Story Space End Wall

Stability Applications

It is valuable to apply the structural planning concepts developed in this chapter to several example problems to gain proficiency in the use of the concepts. This will help to solidify the approach to selection of diaphragm proportions, shear wall locations, tie-down locations and the need for drag struts.

Structural Planning Applications

Example 1: Figure 3.19 shows a first floor plan, roof plan and foundation plan for a basic one-story rectangular house, given the following:

Roof: 3 in 12 slope hip roof form, composed of trusses spaced 24 inches apart. The hips are a combination of flat top trusses and jack rafters.

Garage: Detached.

Foundation Type: Cast-in-place Basement; 8 inch thick walls; a central steel girder spans the length of the basement and is supported by three steel pipe columns spaced 20 feet apart.

Wood Stud Walls: Dimension lumber 2 x 4's at 16 inches on center.

Structural Planning Solution:

A. Wind Direction Perpendicular to the Long dimension of the Building

1. Begin by assessing the proportions of the overall rectangle. Do they exceed L/W of 4? In this case 80/32 = 2.5 and does not exceed 4. Thus the proportions of the diaphragm are within acceptable code prescribed limits. Knowing this, it is possible to proceed with the perimeter walls of the residence to achieve total lateral stability, without the need to employ interior walls. Drag struts may still be required within the shear walls, dependent on window and door locations.

2. Use imaginary planes as shown in Figure 3.20 to help visualize the important structural planes for wind resistance *perpendicular to the long dimension of the building.* Pass the planes through the exterior stud wall and foundation wall, the first floor and the sloping roof planes. This helps to define the roof diaphragms **D** and **E**, the floor diaphragm **C** and the exterior walls that lie in the planes **A** and **B**. The wind reactions are the dark arrows located at the intersection of the roof, floor and exterior wall planes.

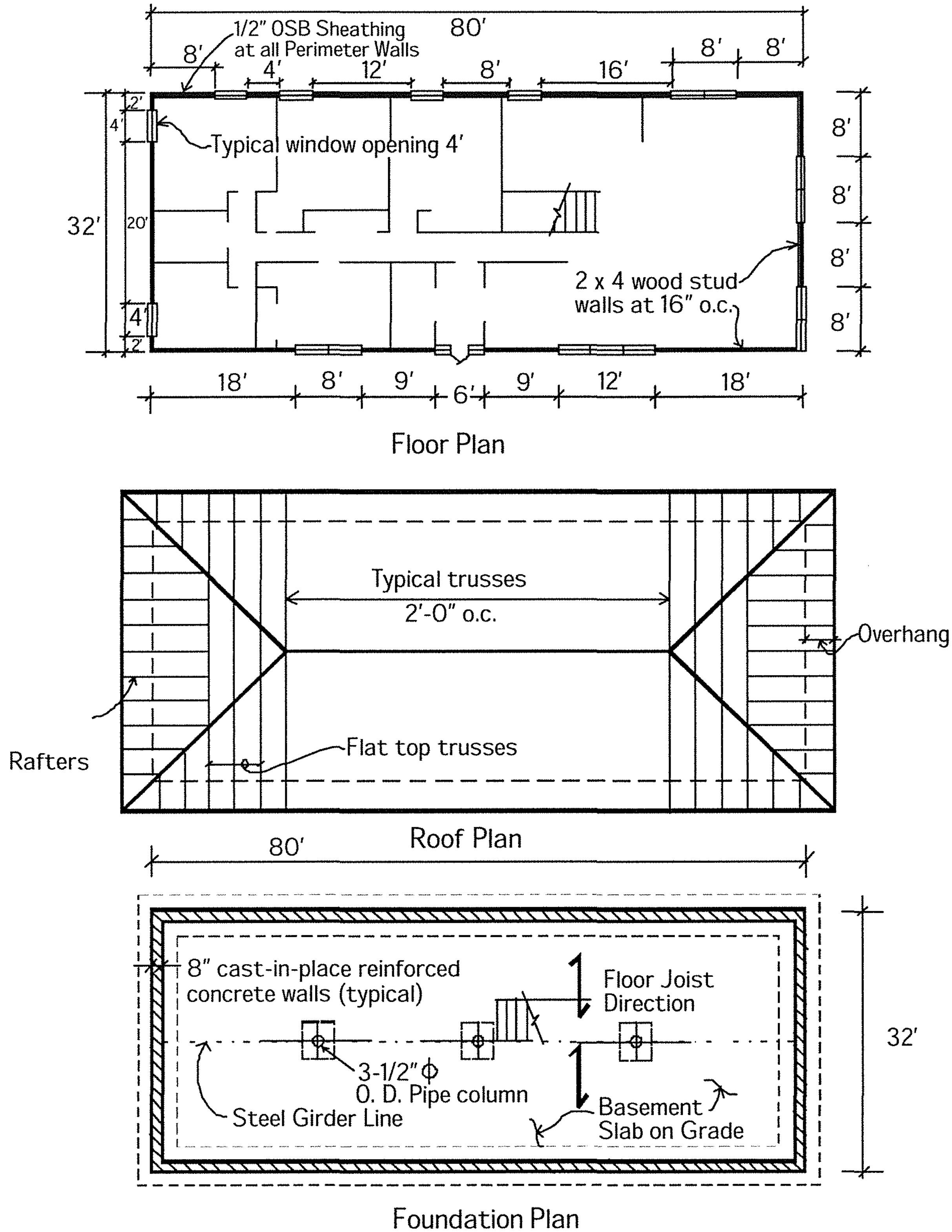


Figure 3.19 - Plans for Example #1

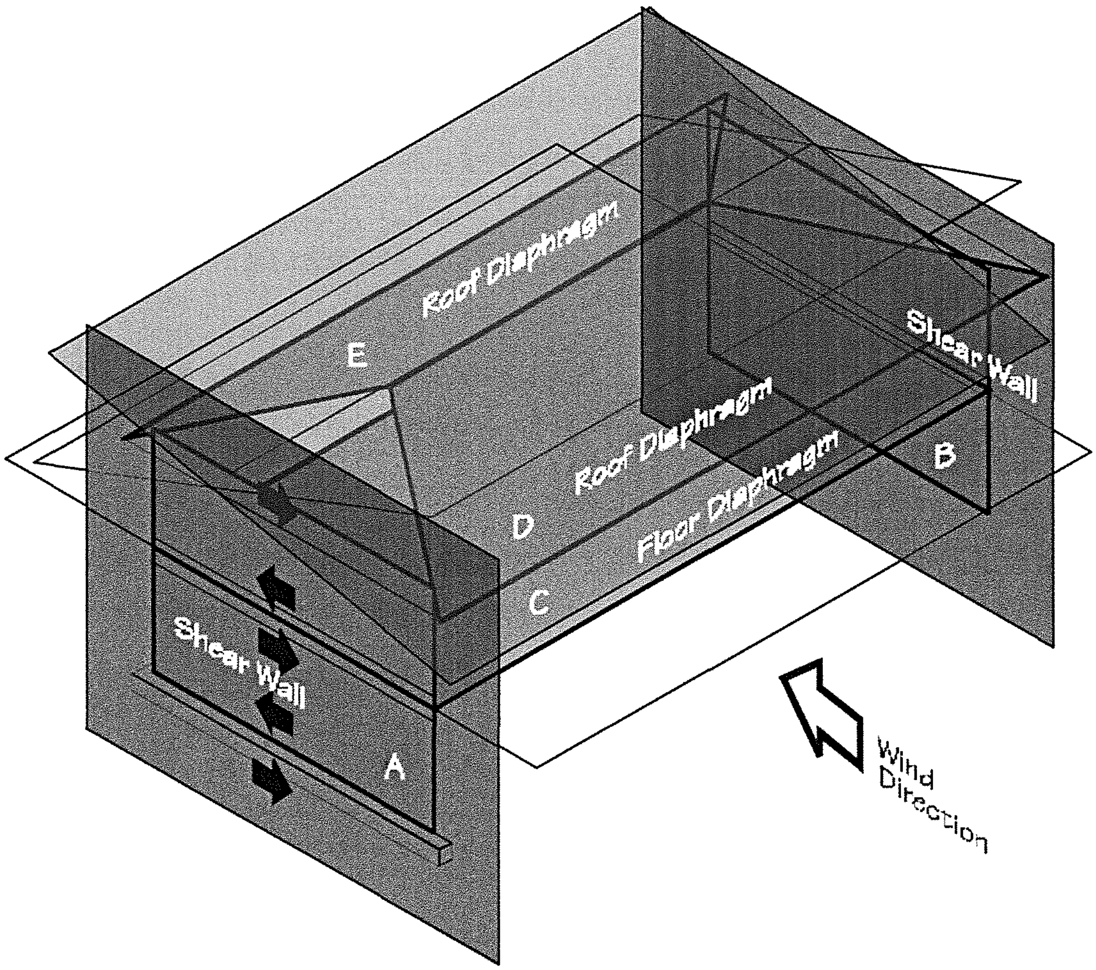


Figure 3.20 Structural Planes of Resistance

3. The exterior walls, whole or in part, will be used as the shear walls 80 feet apart. The floor and roof diaphragms will act as deep horizontal or inclined beams respectively, and transfer wind load they receive to their supports (called wind reactions), which in this case are the short shear walls **A** and **B**. They are desirable shear walls, since they have foundation walls below. They can directly transfer shear force to the foundation walls to resist sliding and overturning. The foundation wall is also important because its concrete mass is excellent mass for resisting overturning.

4. Review the shear wall plane **A** shown in Figure 3.21a by the traditional shear wall method. Windows exist at each end of the elevation. Thus, a rather substantial solid wall segment (20 feet) is available in-between for a solid shear wall application. **Always use the maximum length of shear wall available** since it will reduce the

overturning force and also allow the spacing of the anchor bolts to be further apart for the sliding force. Note again that a foundation wall exists below shear wall **A**. The shear wall being located in the center of the elevation does not receive the wind reaction directly. A drag strut is required to transfer the wind reaction to the shear wall. It is common to use the headers over window and door openings as the drag strut beam. Figure 3.21a illustrates the shear transfer between roof diaphragm and the shear wall when the wind reaction comes from the right side. The sliding and overturning anchorage forces are as shown, and the tension tie-down is at the right side of the shear wall. If the wind reaction comes from the left side all the arrows shown will reverse direction. Overturning anchorage is now required on the left side of the shear wall. This is explained to make the point that **wind can come from either direction and the overturning anchorage tie-down must be provided at both ends of the shear wall.**

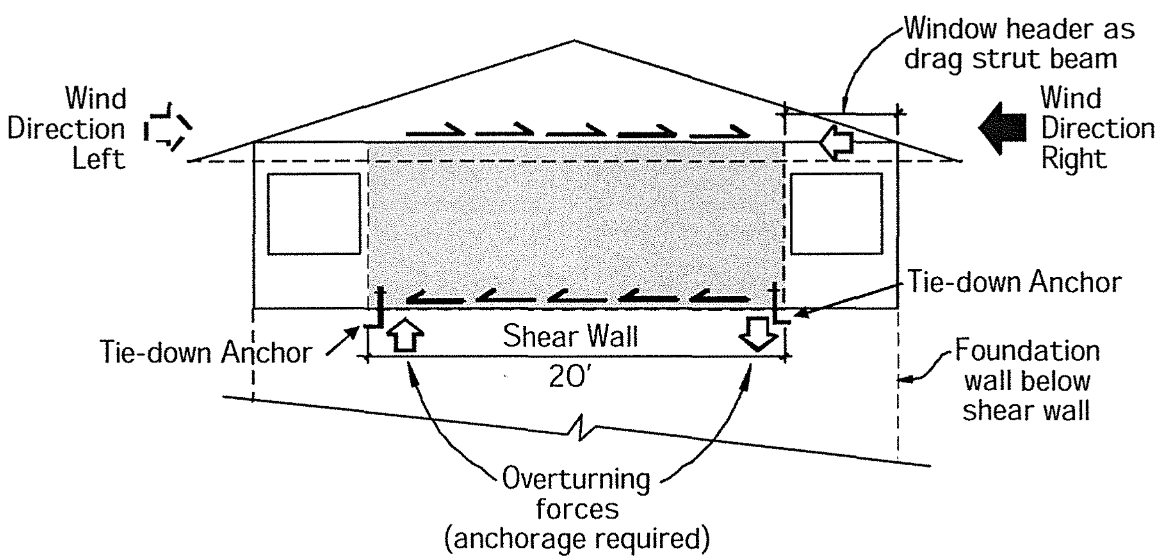


Figure 3.21a - Transverse Shear Plane A

4A. Also, review the shear wall plane **A** shown in Figure 3.21b by the perforated shear wall empirical method. Assume structural sheathing exists above and below the windows at each end of the elevation. Now, the entire perforated wall (32 feet) is available as a shear wall. This method exactly conforms to the general rule to **always use the maximum length of shear wall available.** The perforated shear wall method produces the smallest overturning force and also allows the spacing of the anchor bolts to be further apart for the sliding force. Note again that a foundation wall exists below shear wall **A**. The shear wall being located across the entire elevation does receive the wind reaction directly. No drag strut is required to transfer the wind reaction to the shear wall.

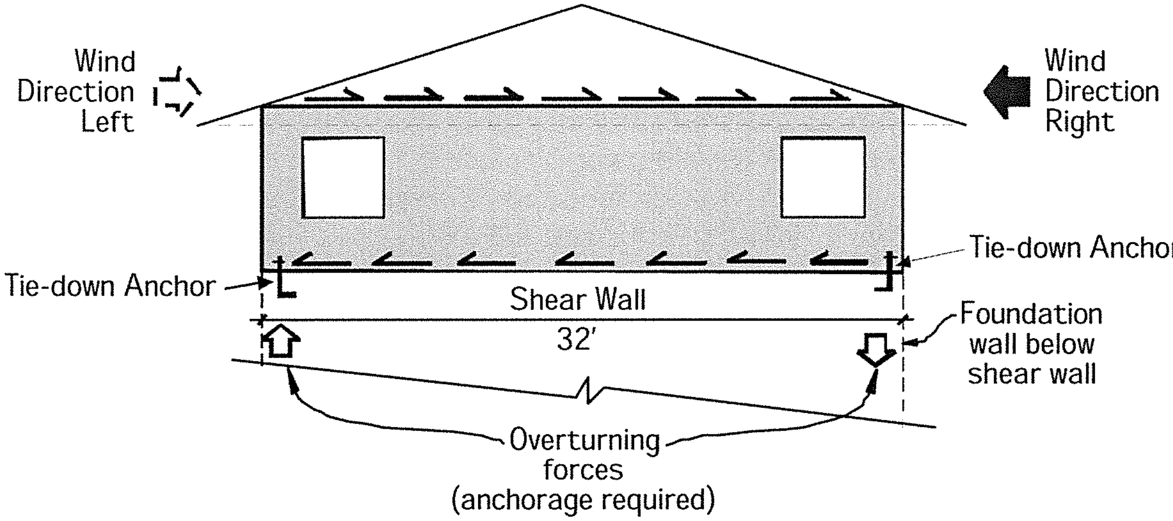


Figure 3.21b - Transverse Shear Plane A
Perforated Shear Wall Empirical Method

5. Review shear wall plane **B** shown in Figure 3.22a by the traditional shear wall method. Two separate windows exist, leaving two 8-foot solid wall segments available as shear walls. Whether this is structurally adequate will depend on the wind load magnitudes, but most likely for a one-story residence the two-combined shear wall segments should be structurally sufficient for handling the overturning and sliding. Two drag struts will be required to transfer the wind reaction from the left to the two shear wall segments. It is assumed that both shear wall segments will be required to handle the wind reaction. When the wind reaction comes from the right the shear and anchorage arrows will reverse direction as described in step #4. This time only the drag struts left and right of the left shear wall segment are required to transfer the wind reaction between the two shear walls, since the right shear wall starts at the end of the wall. Note again the positive influence of a foundation wall existing below shear plane **B**.

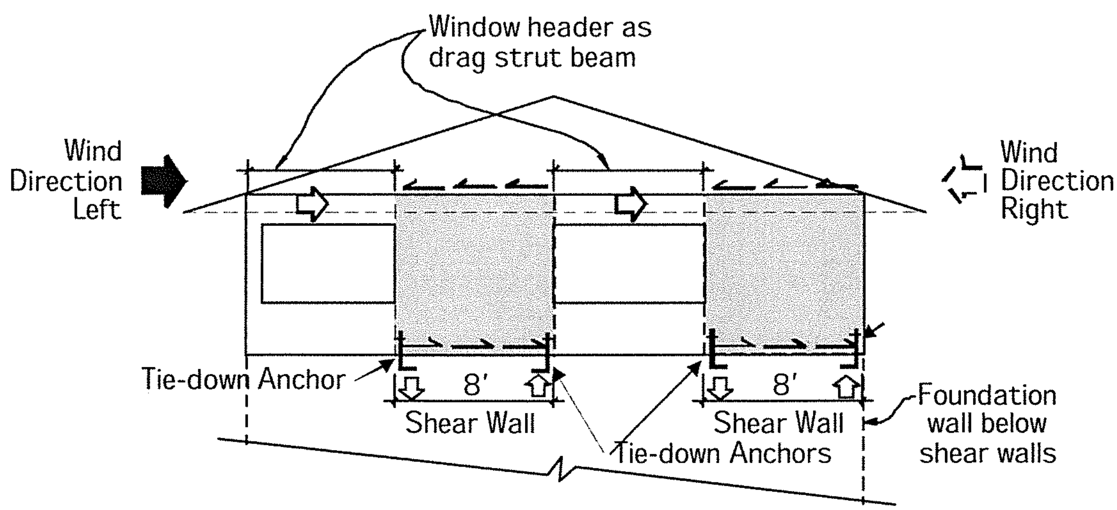


Figure 3.22a - Transverse Shear Plane B

Option: It could be possible to move either or both of the windows to create one large shear wall. The maximum shear wall length might become 16 feet. This is a design consideration that may be unworkable for the outside views from within the rooms, or it may be unsatisfactory for the aesthetics of the elevation.

5A. The shear plane **B** should also be looked at for economy using the perforated shear wall empirical method. Given that the left window is a corner window, and it is desired by the architect to omit all but the jamb, even with sheathing above and below the corner window there are no studs at the corner. Thus, the shear wall will run 24 feet, including the right window that must be structurally sheathed above and below. This approach eliminates two tie-down anchors. Figure 3.22b illustrates this solution to elevation plane **B**'s shear wall design. A drag strut will be required over the left window for wind coming from the left.

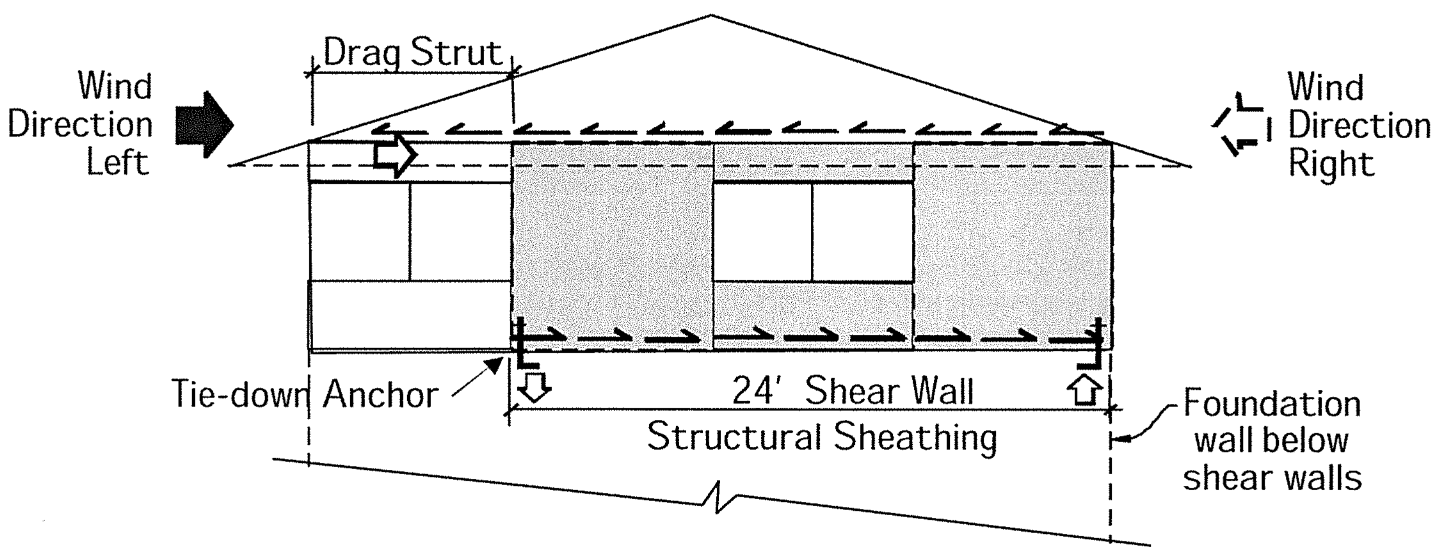


Figure 3.22b - Transverse Shear Plane B
Perforated Shear Wall Empirical Method

6. The above 5 steps have been done without numbers to avoid complicating the structural planning process. Chapter 5 will review an example with numbers and use the selection Tables of Appendix B.

B. Wind Applied Parallel to the Long Dimension of the Building.

1. Begin by assessing the proportions of the overall rectangle. Do they exceed L/W of 4? In this case 32/80 = 0.4 and does not exceed 4. Thus the proportions of the diaphragm are well within acceptable code prescribed limits. Knowing this, it is possible to begin the process with the perimeter of the residence to achieve total lateral stability. Drag struts may still be required to transfer the wind reaction to the shear wall segments, depending on the location of door and window openings.

2. Use imaginary planes as shown in Figure 3.23 to help visualize the important structural planes for wind resistance *parallel to the long dimension of the building.* Pass the planes through the long exterior stud and foundation walls, the first floor and the sloping roof planes. This helps to define the roof diaphragms **D** and **E**, the floor diaphragm **C** and the shear wall planes **A** and **B**. The wind reactions are the dark arrows located at the intersection of the roof, floor and exterior wall planes.

3. The exterior walls, whole or in part, will be used as the shear wall planes 32 feet

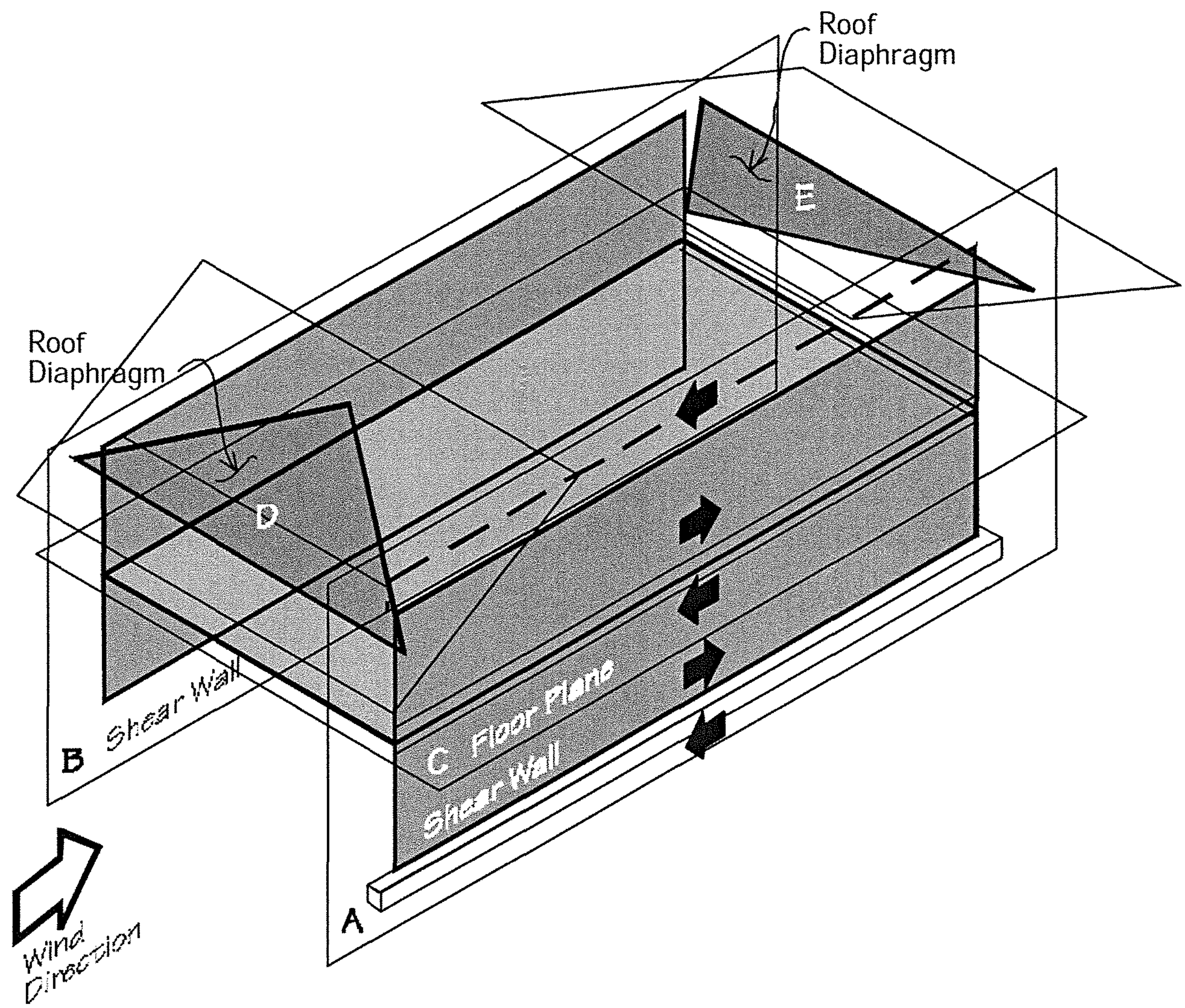


Figure 3.23. Structural Planes of Resistance

apart. The floor and roof diaphragms will act as horizontal beams and transfer wind load they receive to their supports, which in this case are the long shear walls. They are desirable shear walls, since they have foundation walls below them, and can directly transfer shear force to the foundation walls to resist sliding. The foundation wall is also important because its concrete mass can be used for resisting overturning. Note that smaller wind reactions will occur in this direction since the surface area of the end walls is much smaller than the long wall surface areas.

4. Review shear wall plane **A** shown in Figure 3.24a using the traditional shear wall method. Several windows plus an entry door exist along the elevation. A rather substantial length (18 feet) is available for two major shear walls, if needed. These shear walls are located at the ends of the elevation, which is good to avoid twist of the building under a diagonal wind load. No drag struts will be required to transfer wind reactions to the shear walls since both shear walls start at the ends of the elevation. When the wind reaction comes from the left, the left shear wall is structurally activated. It is highly unlikely that both shear walls are required to handle the wind reaction from the left. When the wind reaction comes from the right the dotted arrows are activated at the right shear wall, without involving the left shear wall. Thus only one shear wall functions for a wind reaction in one particular direction. Should both shear walls be required, a long drag strut would be required to link the two shear walls. Note again that a foundation wall exists below the potential shear walls. There is no need to use the shorter wall segments as shear walls. The longer shear walls are more structurally efficient.

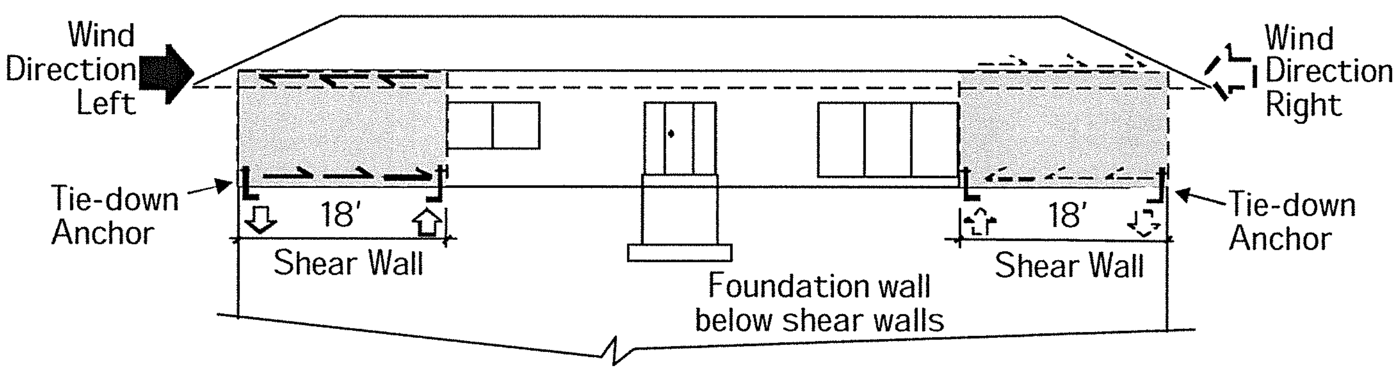


Figure 3.24a - Longitudinal Shear Plane A

4A. Review shear wall plane **A** shown in Figure 3.24b using the perforated shear wall method. Several windows plus an entry door exist along the elevation. Given the length of the wall, and the fact that it is only a one-story residence, it is unlikely that the whole wall will be required as a shear wall. However, if the entire wall is structurally sheathed, including above and below openings, then the entire wall can be a shear wall. Most likely there will be no uplift due to overturning and tie-downs won't even be required.

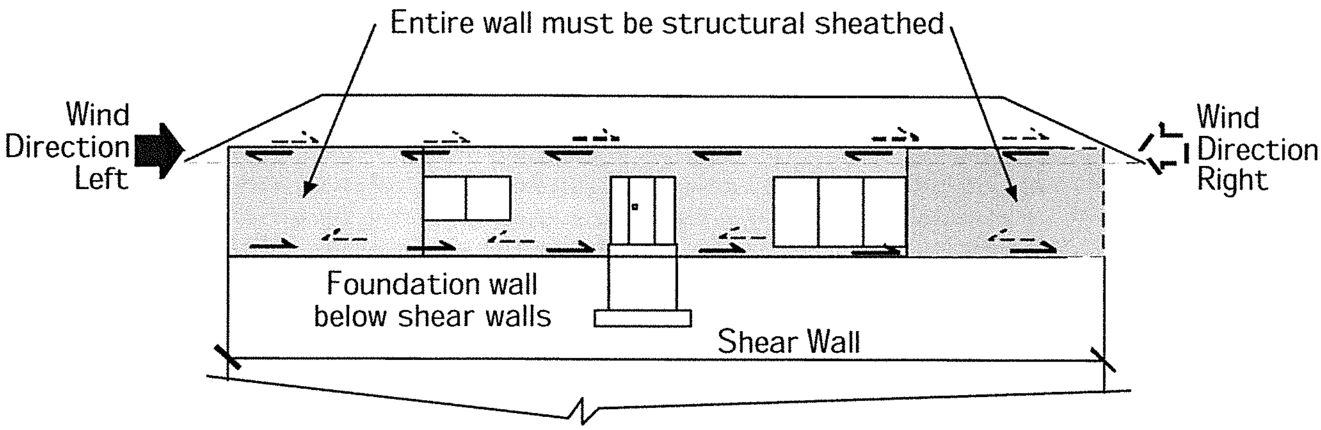


Figure 3. 24b - Longitudinal Shear Plane A
Perforated Shear Wall Empirical Method

5. Review shear wall plane **B** shown in Figure 3.25a. Five window locations exist, leaving two 8-foot solid wall segments available for shear walls at the ends of the elevation, or the 16-foot long solid wall segment near the left side of the elevation available as a single shear wall. Whether the two shorter end wall segments should be used, or the longer more central wall segment should be used as the potential shear wall(s) will depend on the wind load magnitudes. Most likely for a one-story residence the two end segments will be structurally sufficient and most economical to handle the overturning and sliding.

Since these two shear walls exist at the corners of the elevation, the shear wall on the right side will directly receive the wind reaction (solid arrows in Figure 3.25a) that is coming from the right. This right shear wall will most likely be structurally adequate to handle all the wind from the right direction. Therefore, the opposite end shear wall is not needed for sliding or overturning resistance. If this is the case, no drag strut will be necessary to transfer half the wind reaction to the opposite shear wall. However, when the wind reaction comes from the left (the dotted arrows in Figure 3.25a) the left shear wall directly receives the wind and the right end shear wall is not required. Again, no drag strut is required to force sharing of the total wind reaction.

Should one decide to use the 16-foot long wall segment as the shear wall (dotted section in Figure 3.25a) all by itself, drag struts would be required to transfer the wind reaction from the right or from the left. Tie-down anchors will be required at the ends of each shear wall segment, regardless of option selected. Note again that a foundation wall exists below all of these shear walls.

It is unlikely, but if all three shear walls would be required to equally share the wind reaction from either direction, drag struts between the shear walls would be needed to force the transfer of the wind reaction between the three shear walls.

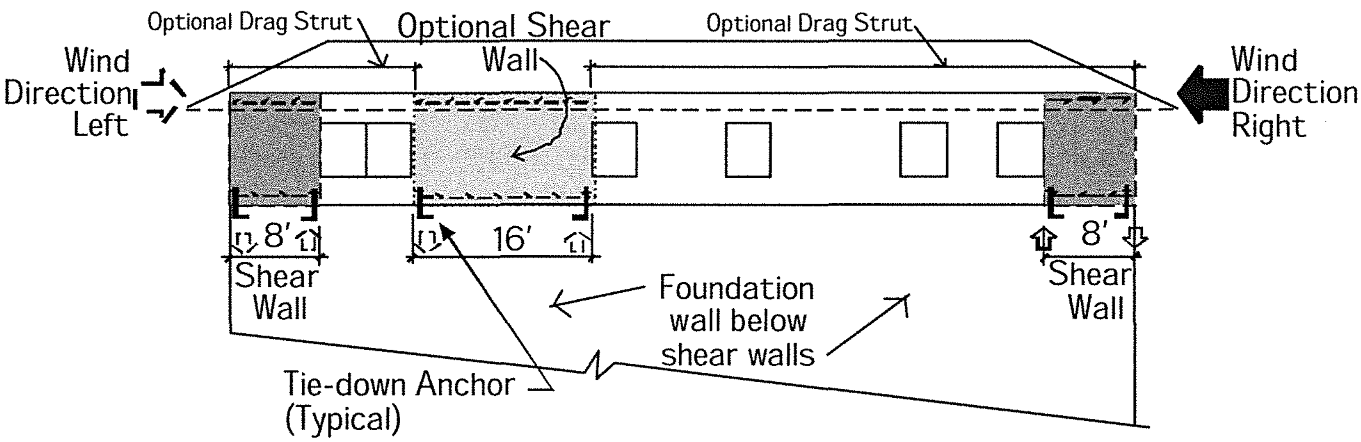


Figure 3.25a - Longitudinal Shear Plane B

5A. Review shear wall plane **B** shown in Figure 3.25b by the perforated shear wall empirical method. Structurally sheath above and below all five openings. The entire length of the elevation is now available as a shear wall. The main advantages include 1) no need for drag struts; 2) a reduction of tie-down anchors from 4 to 2 or most likely none. Since the entire wall is now structurally sheathed the tension uplift chain is also accounted for, and thus serves two purposes.

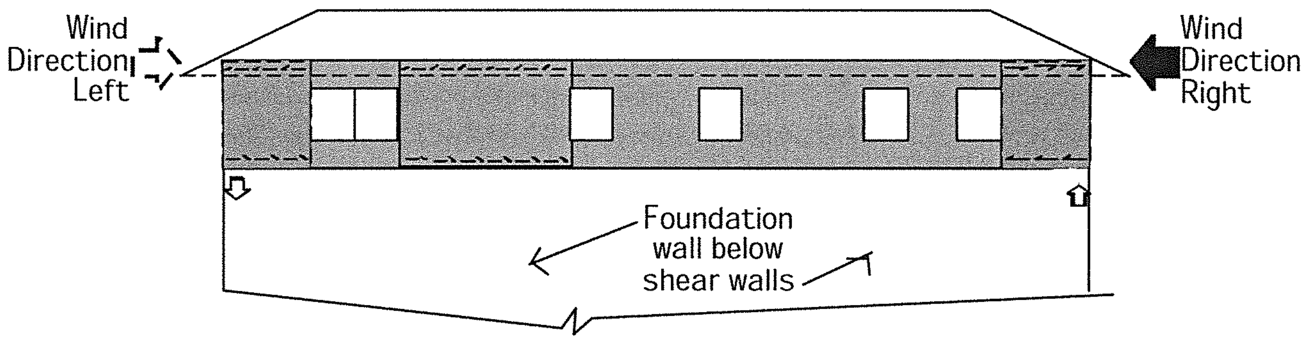


Figure 3.25b - Longitudinal Shear Plane B
Perforated Shear Wall Empirical Method

6. The above 5 steps have been done without numbers to avoid complicating the structural planning process. Chapter 5 will review this example with numbers and use the selection tables of Appendix A.

Example 2: This example is very complicated compared to the first example. It will test your stability understanding to the fullest. Figures 3.26 through 3.28 show a floor plan, foundation plan and roof plan for a one-story house with a set of intersecting gable roofs. The following information is given:

Roof: 6 in 12 slope hip roof form, composed of trusses spaced 24 inches apart. A cathedral ceiling exists with rafters at 16 inches on center, where shown on Figure 3.28. The roof intersections require jack rafters sitting on the trusses below. The structural roof-framing plan is shown in Figure 3.29, and the four elevations are shown in Figure 3.30. North is up on the plans.

Garage: Attached with a slab-on-grade.

Foundation Type: Cast-in-place Basement, 8 inch thick walls and a partial basement where shown on Figure 3.27. Two steel girder lines span the length of the basement and are supported by three steel pipe columns as shown in Figure 3.27. A fourth column is used to support the roof ridge beam that provides support for the vaulted ceiling framing.

Wood Stud Walls: Dimension lumber 2 x 4's at 16 inches on center.

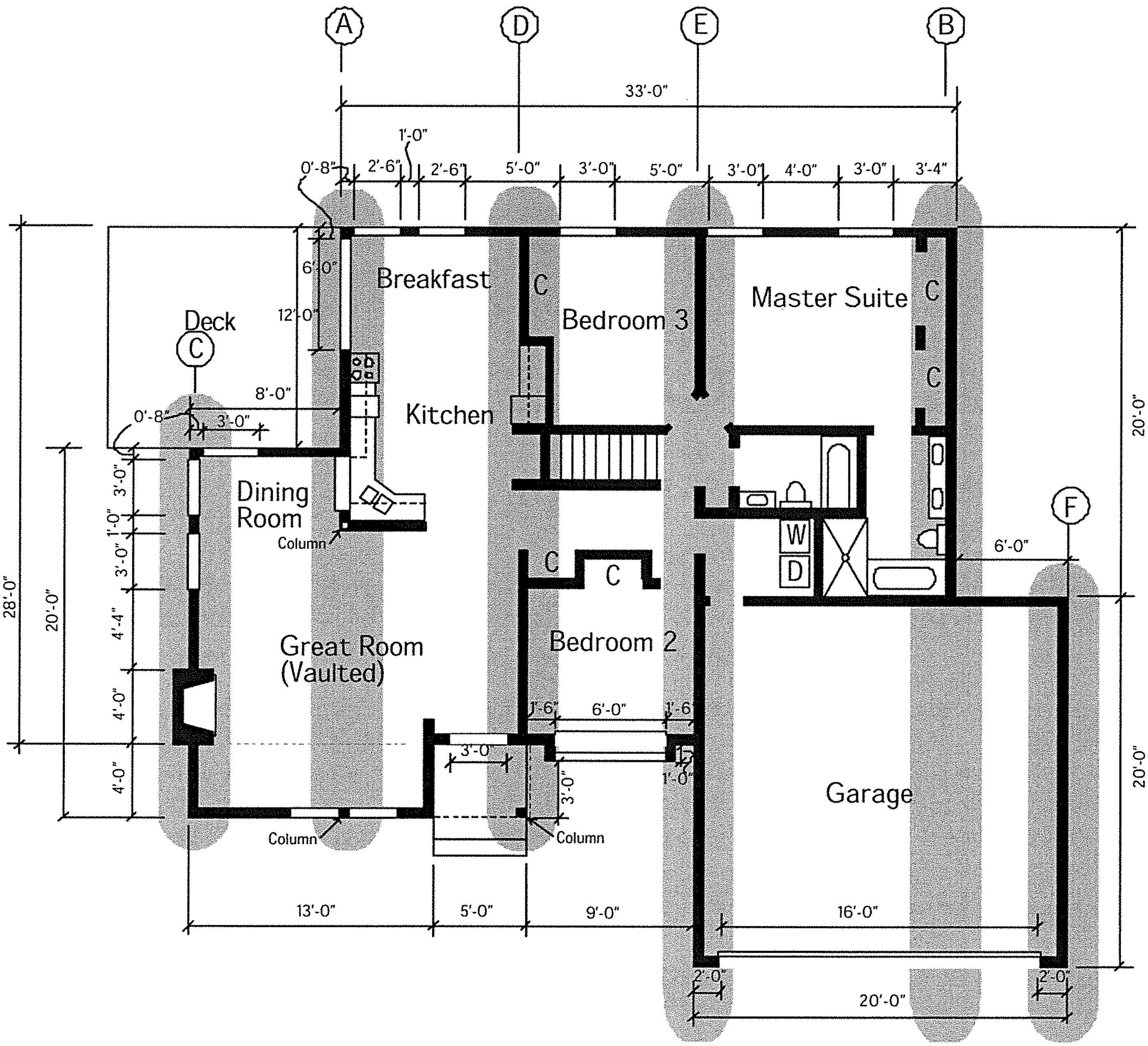


Figure 3.26 - First Floor Plan

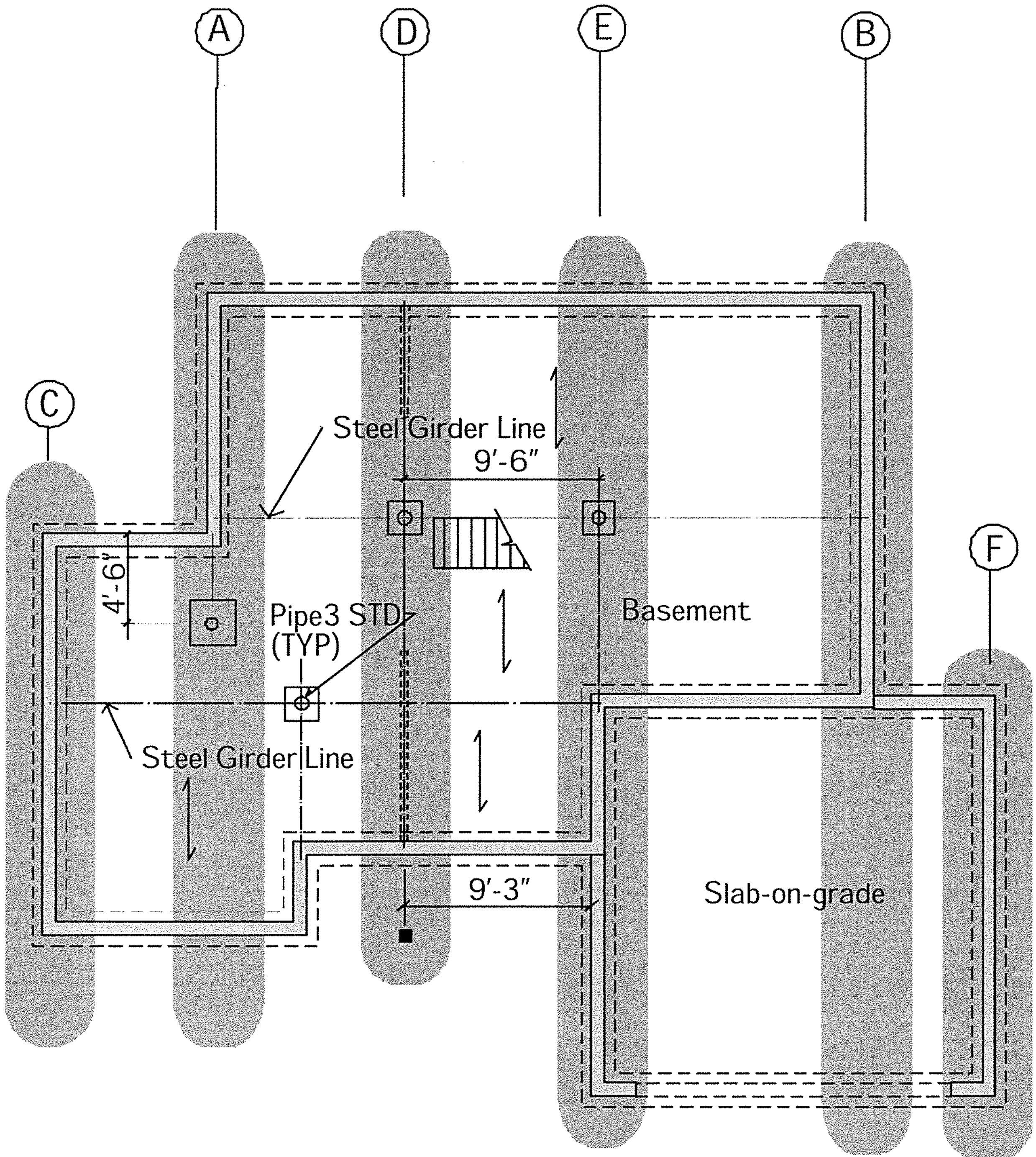


Figure 3.27 - Foundation Plan

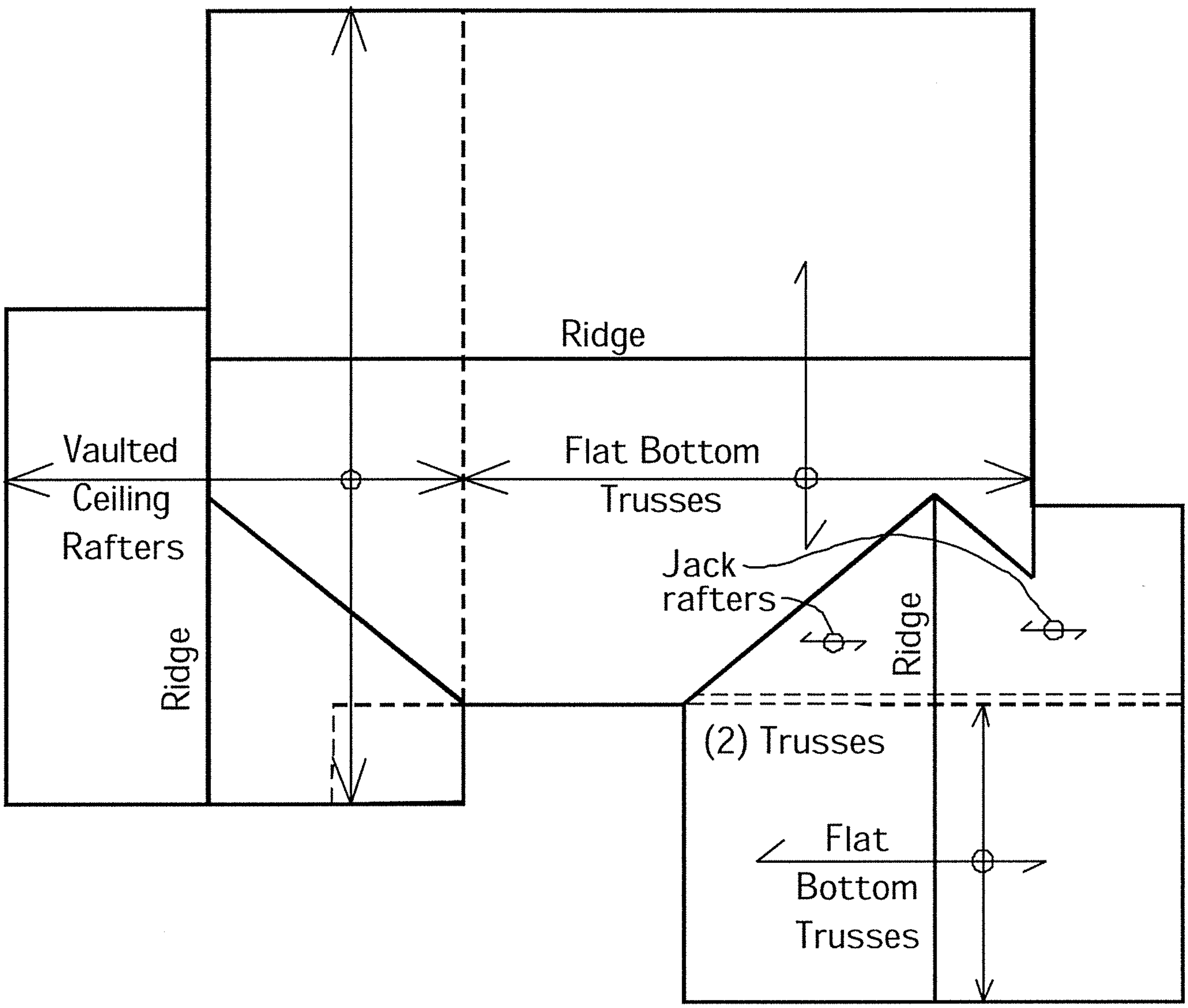


Figure 3.28 - Roof Plan

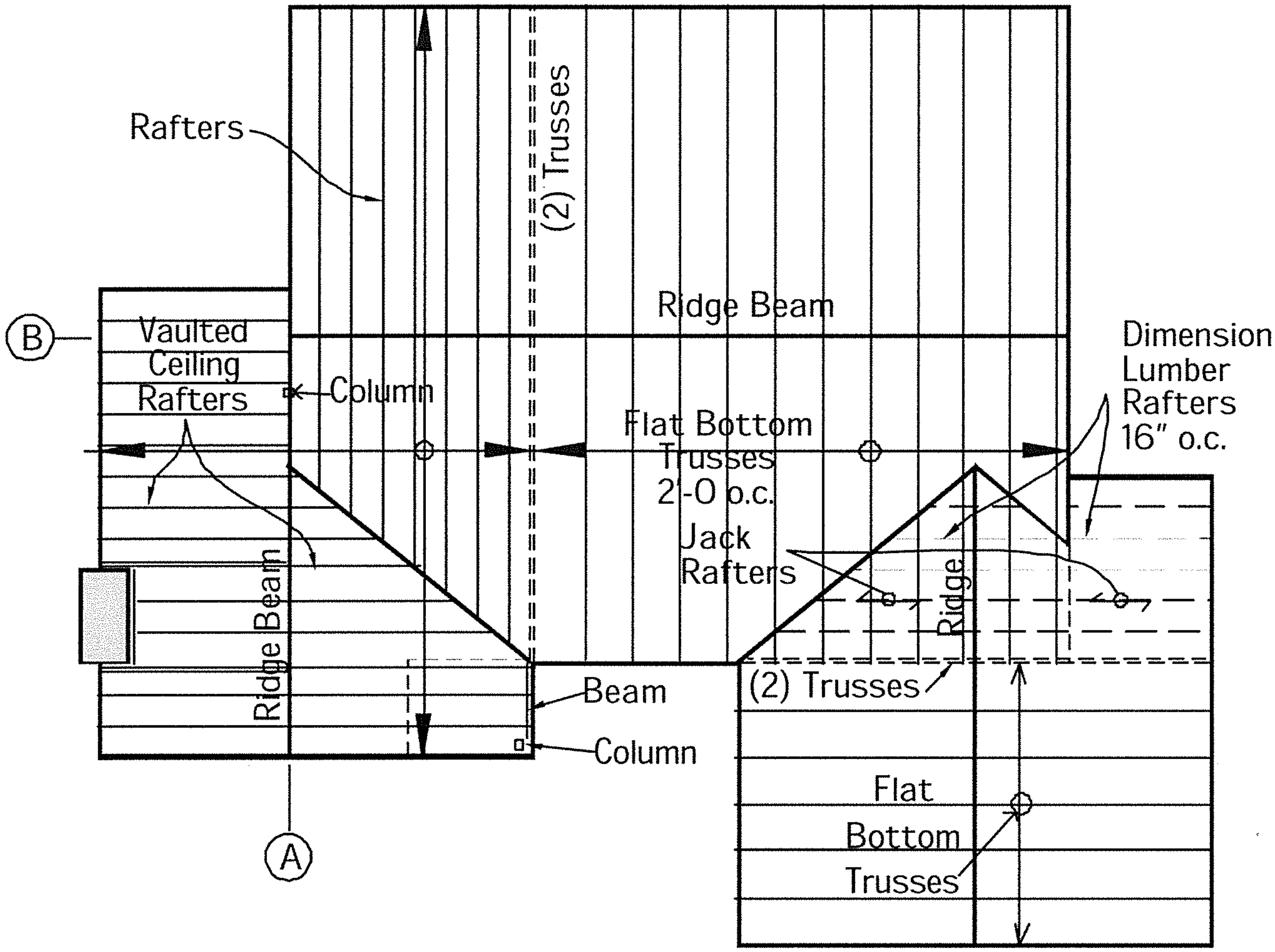


Figure 3.29 - Roof Framing Plan

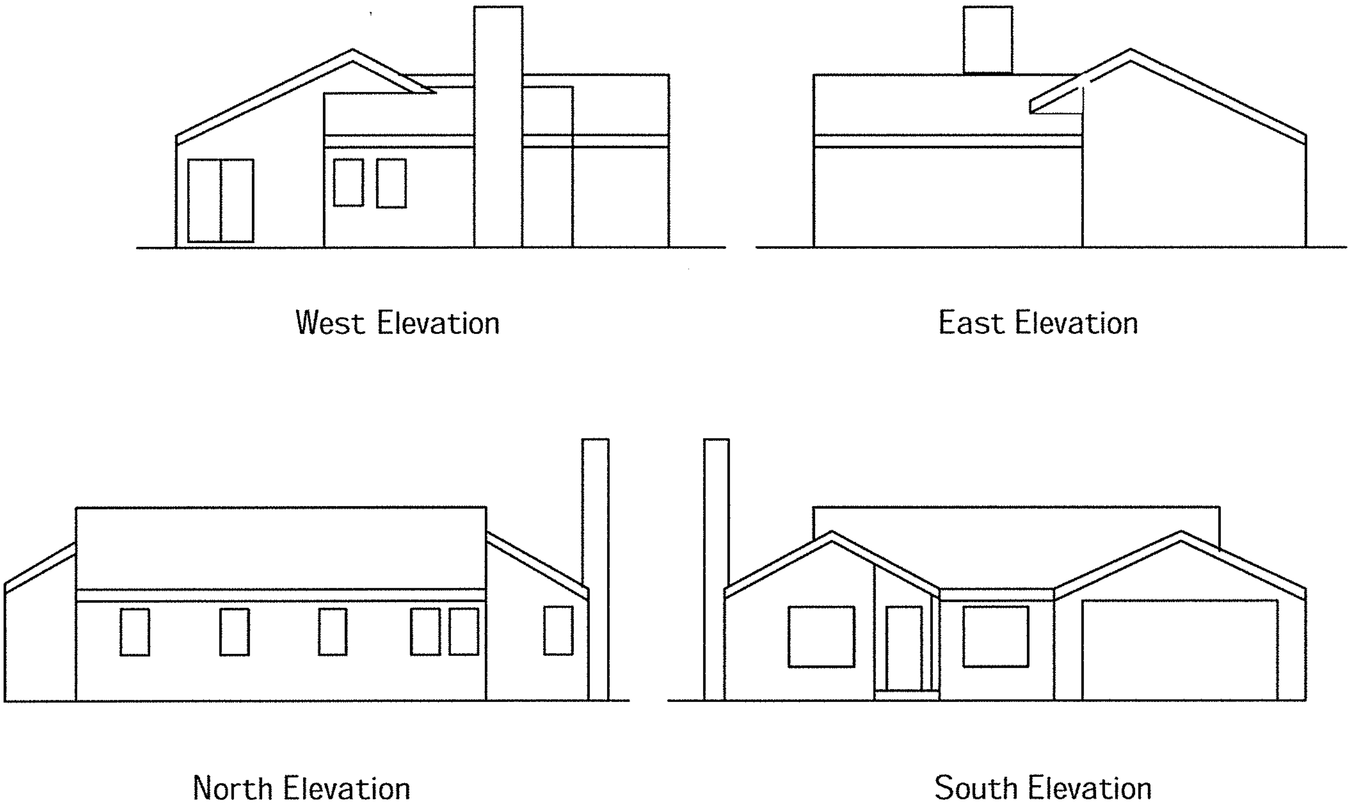


Figure 3.30 - House Elevations

Structural Planning Solution:

A. Wind Direction Perpendicular to the Long dimension of the Building

1. Assessing the proportions of the overall rectangle is difficult since there is more than one rectangle and they are offset from each other. When more than one gable roof exists and they are at different elevations, each roof form will become a separate diaphragm. This example has three gable roof forms and thus three diaphragms will become activated. Do any of the three roof spans perpendicular to the direction of the wind exceed L/W of 4? In this case 33/27 = 1.22 and does not exceed 4. The two other smaller diaphragms will certainly be less than that so the proportions of the three diaphragms are within the acceptable code prescribed limits. Knowing this, it is possible to proceed with the perimeter walls that border each diaphragm and unfortunately any interior walls as well to achieve total lateral wind stability. Drag struts will most likely be required given the plan, roof and foundation layout.

2. Use imaginary planes as shown in Figure 3.31 and 3.32 below to help visualize the important structural planes for wind resistance *perpendicular to the long dimension of the building* for the **main roof diaphragm #1**. Pass the planes through the exterior stud wall and foundation wall, the first floor and the sloping roof planes. The

vertical planes help to define the gable ends of the roof diaphragm #1, the floor diaphragm and the elements within the exterior/interior walls that lie in the planes **A** and **B**. Refer back to the first floor plan and basement plan to see the location of the **A** and **B** planes in plan view.

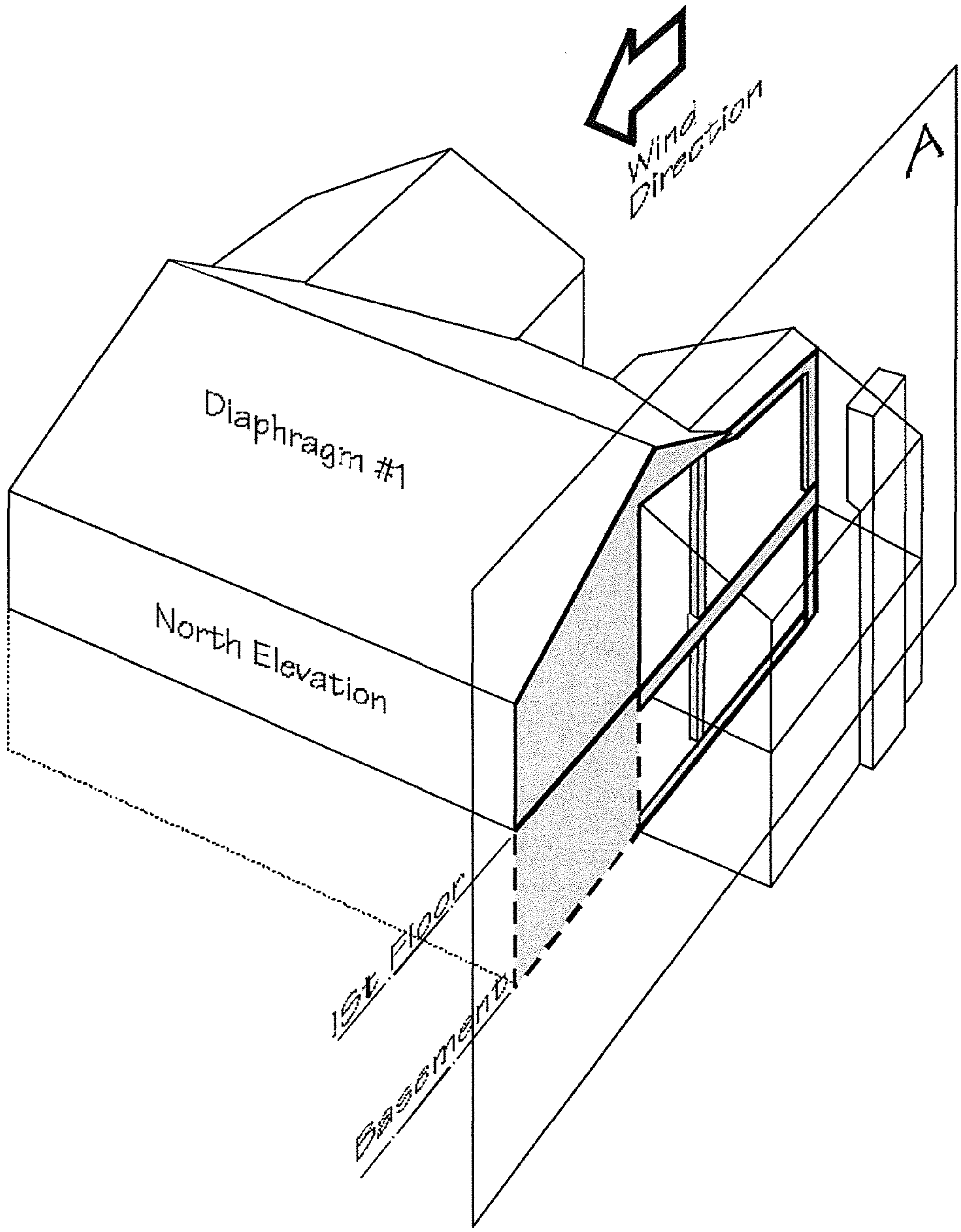


Figure 3.31 - Shear Plane A

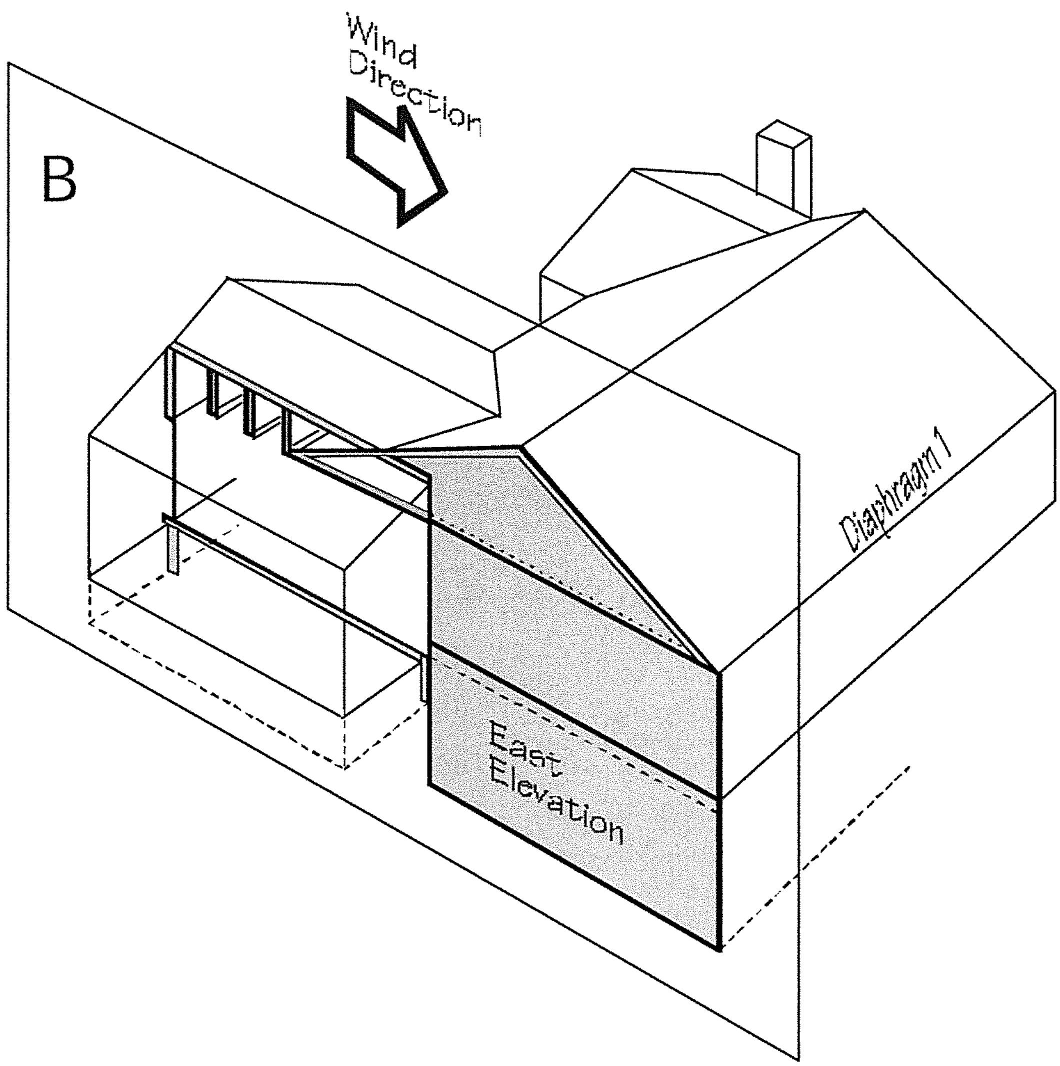


Figure 3.32 - Shear Plane B

3. The vertical plane in Figures 3.31 and 3.32 cut through the wall sections shown in Figures 3.33 and 3.34 respectively. They show in detail what can be used for the stability system related to roof diaphragm #1. The wall section at plane **A** in Figure 3.33 reveals the need for extensive and irregular shaped drag strut members. It also shows that only a 6-foot long wood shear wall is available to handle the overturning and sliding from the right or left wind reaction. The anchorage tie-down force will be large, but there is a substantial foundation wall directly below to provide sufficient dead load resistance. The drag strut beams would probably be formed from doubling the rafters and use of the ridge beam, plus adequate connections to the wood shear wall. The traditional shear wall design method will be employed here.

There is no benefit from the perforated shear wall empirical method, since there is no sheathed segment to the left of the door opening. The stiffness of the shear wall would also not improve.

To improve the structural stability conditions one might consider moving the sliding doors to the north elevation and extending the patio deck. This would create a 12-foot shear wall, reduce the anchorage tie-down force and create a stiffer shear wall to reduce movement during a wind loading. There would be the elimination of the drag strut header over the door opening, but the most complicated drag struts would still remain. The architect for this example would not move the opening and it was built as shown.

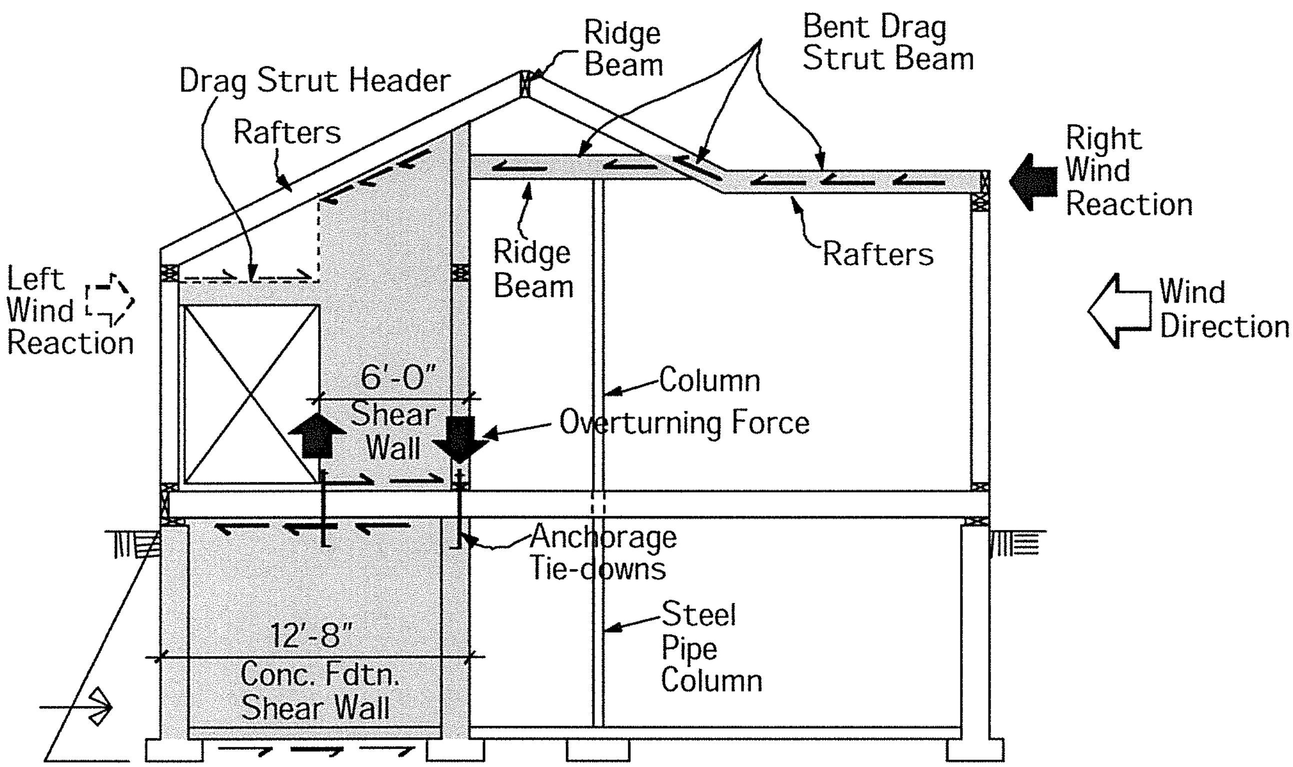


Figure 3.33 - Building Section at Plane A

Figure 3.34 is the building section cut by the vertical plane **B**. This section cuts through the garage necessitating a 20-foot long drag strut. This drag strut would be a combination of the bottom chord of the roof truss and the creation of a beam perpendicular to the garage roof trusses. **Diagonal braces spaced at 4 feet apart are an important framing addition to transfer wind along the garage front elevation to the roof diaphragm over the garage.** Consider how flexible the front elevation will be given a 7' x 16' opening cut into the wall. The braces will help to keep the front elevation from imploding if subjected to a left wind direction. Even with a high wind resistant garage door, the wall around it needs to be strong and stiff, and anchored to the foundation.

The exterior wall behind the garage is quite a substantial wood shear wall and has a foundation wall below it for sliding and overturning force transfer. The left wind reaction transfers along the drag strut to the wood shear wall, which in turn is anchored to the foundation wall below. The final resistance to the wind reaction is the friction along the bottom of the concrete footing and the passive soil pressure block on the leeward side of the foundation wall. Again, the traditional shear wall method is the logical choice, since the only part of the elevation that is useful as a shear wall is the exterior wall as shown.

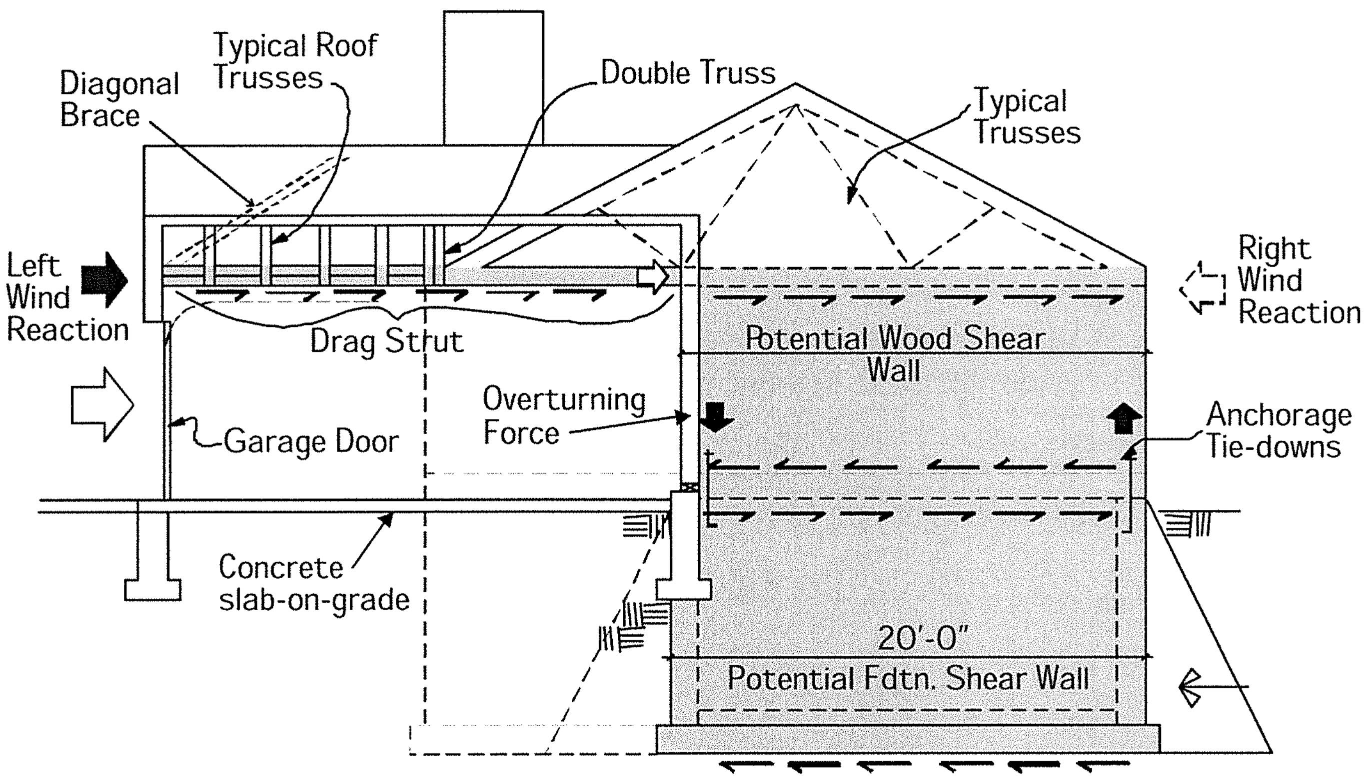


Figure 3.34 - Building Section at Plane B

4. Figure 3.35 illustrates the second set of vertical planes **C** and **D** cut at the extremities of the **roof diaphragm #2**. Note that this diaphragm is perpendicular to the main roof diaphragm #1. The wind direction is the same as for Figure 3.32.

5. The vertical planes **C** and **D** in Figure 3.35 cut through the wall sections shown in Figures 3.36a and 3.37 respectively. They show in detail what can be used for the stability system related to roof diaphragm #2.

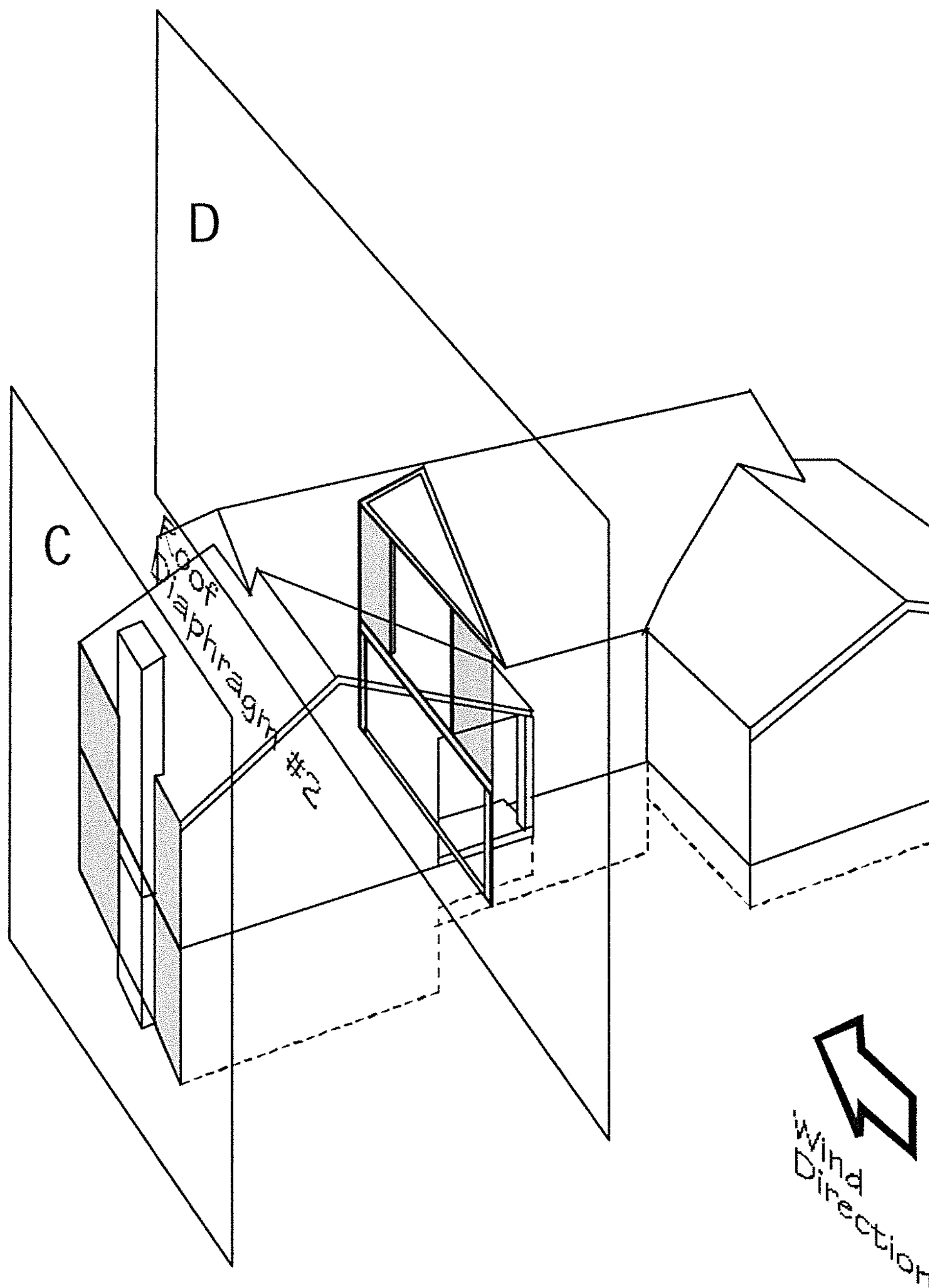


Figure 3.35 - Shear Planes C & D

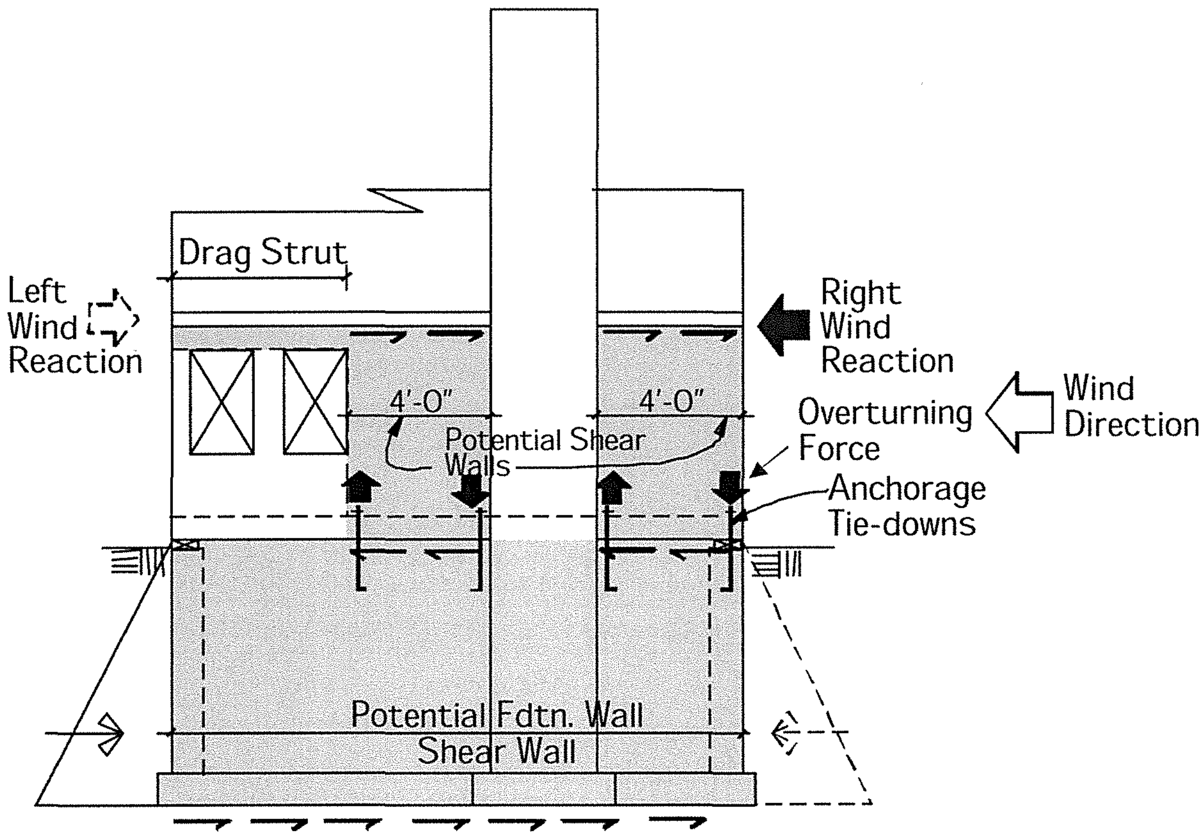


Figure 3.36a - Building Exterior Wall at Plane C

Review the exterior wall cut by plane **C** shown in Figure 3.36a for potential shear walls using the *traditional shear wall design method*.

The windows and the fireplace break the wall into distinct segments. Only a 4-foot section either side of the fireplace is available to act as wood shear walls. The fireplace unit is a "zero clearance" type, and thus can be boxed in wood framing. It was stated earlier in this chapter that 4 feet should be considered the shortest shear wall to resist the stability issues of overturning and sliding. It is fortunate that two such walls exist to share the right or left wind reaction. A drag strut header over the windows will be required to transfer the left wind reaction to the left shear wall segment. It is assumed that both shear wall segments will be required to handle the wind reaction and thus the fireplace wood framing will have to be detailed to act as a drag strut. When the wind reaction comes from the right, the shear and anchorage arrows will be as shown in Figure 3.36a. This case sends the right wind reaction directly into the right shear wall. The fireplace construction, acting as the drag strut again will activate the left shear wall segment. Note again that a foundation wall exists below both shear walls. The final resistance to the wind reaction is the friction along the bottom of the concrete footing and the passive soil pressure block on the leeward side of the foundation wall.

Review the exterior wall cut by plane **C** shown in Figure 3.36b for potential shear walls using the *perforated shear wall empirical design method*.

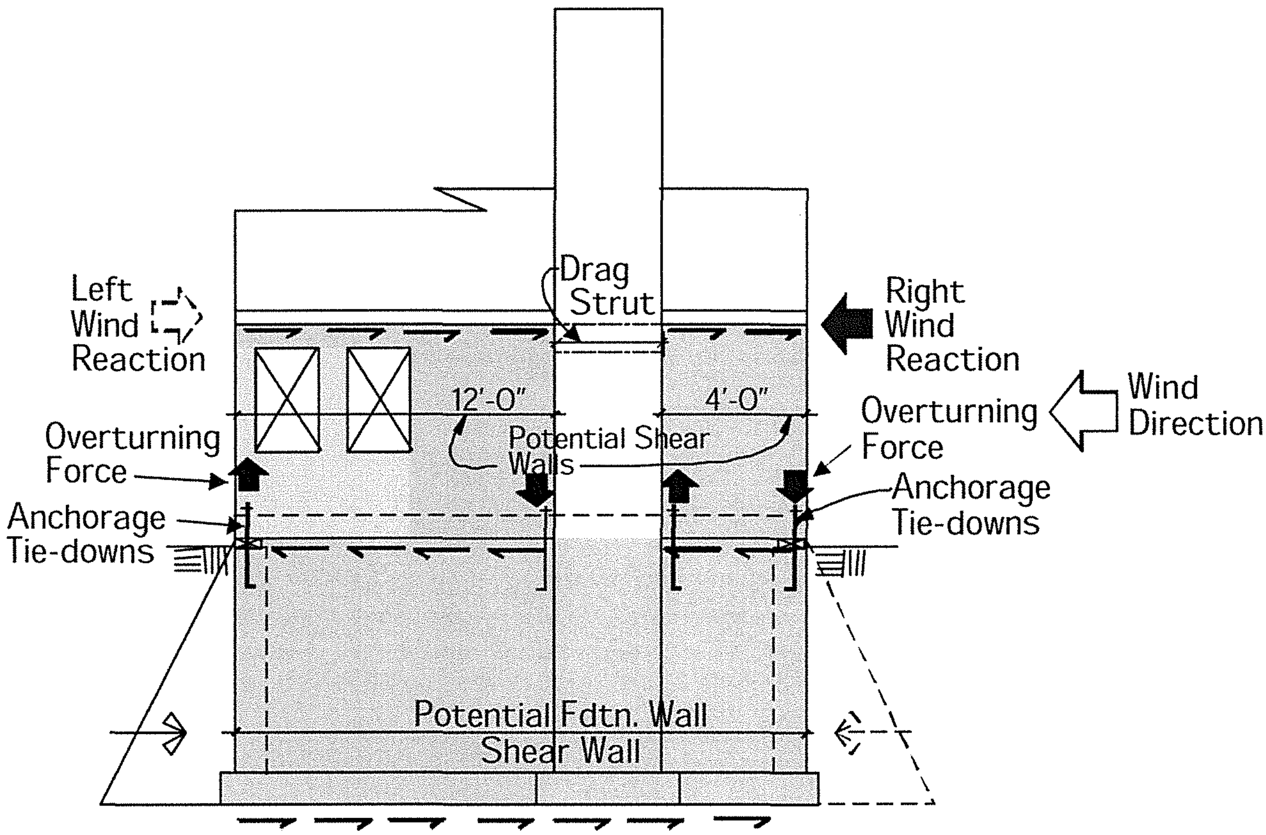


Figure 3.36b - Building Exterior Wall at Plane C

There is an opportunity to take advantage of the perforated wall method. Two windows exist near the left corner, but are separated by a small wall dimension. Structural sheathing above and below the windows will permit the length of wall between the fireplace and the left corner as one shear wall as shown in Figure 3.36b. Two tie-down anchors will likely be required at the ends of the wall. The right shear wall segment will still be treated by the traditional method as described above.

Option: It could be possible to move the right window to create a longer shear wall segment on the left side if stiffness is an issue. The window could be placed adjacent to the other dining room window on the north elevation. This is a design consideration that may be unworkable for the outside views from within the rooms, or it may be unsatisfactory for the aesthetics of the elevation.

Review the vertical section wall cut by plane **D** shown in Figure 3.37 for shear wall potential by the *traditional shear wall design method*.

There are two interior wood stud walls that could be structurally sheathed on one side to create shear walls within the vertical plane **D**. Both walls may not be needed as shear walls depending on the magnitude of the right or left wind reaction. It is permitted by most building codes to utilize drywall as shear resistant for shears less than 150 plf, but this author prefers to let drywall sheathing be a reserve capacity added to the safety factor, yet not relied on exclusively for the shear resistance of the house. It is common to add a layer of drywall over the structural wood sheathing for painting and finishing purposes. The major problem in this vertical wall section is that no foundation wall exists below either potential wood shear wall. Three possible structural design approaches can be considered for such a condition.

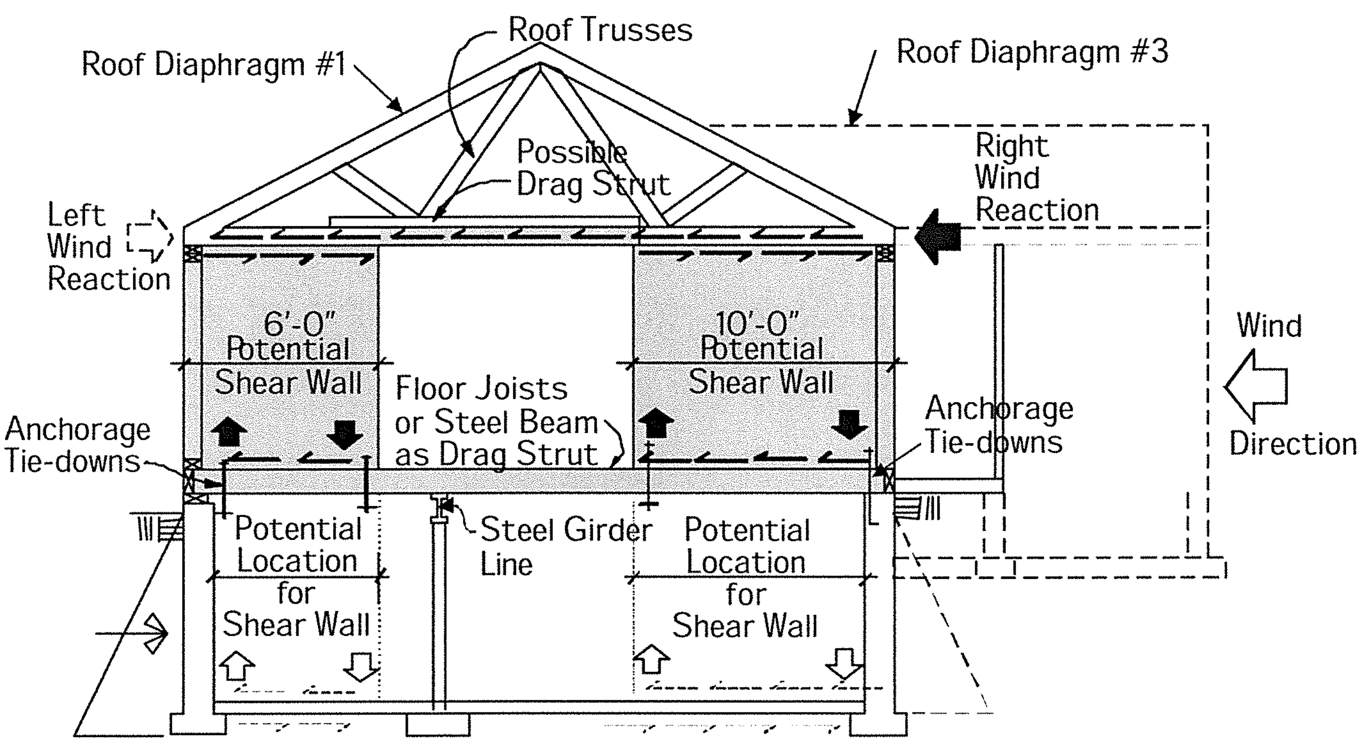


Figure 3.37 - Building Section at Plane D

a. The first approach is somewhat intuitive, but not necessarily conservative. It assumes that roof diaphragm #1 contains the vertical plane **D** and will interactively pick up much of the wind reaction from plane **D** and send it into the diaphragm to its ends. The shear walls in vertical planes **A** and **B** will therefore receive a larger wind reaction. This means there is no need for either of the potential shear walls in plane **D**.

b. The second approach is somewhat conservative, but will add stiffness to the house at the vertical plane **D**. It ignores the roof diaphragm #1 and transfers the wind reaction to one of the potential wood shear walls. It then doubles the first floor joists under the shear wall, using it as a drag strut to transfer the overturning and sliding to a wood stud wall sheathed with structural panels in the basement. It is an architectural design decision whether to place this new wall under the left or right shear wall. Figure 3.37 shows both options. Either new shear wall should be anchored along the vertical edge that comes in contact with the foundation wall so as to activate its dead weight. The opposite end of the shear wall may require a footing below the slab to provide enough overturning uplift resistance.

c. The third approach inserts a steel beam under the shear wall(s) to act as a drag strut, but also as a bending member that receives the overturning forces by means of tie-down anchors to the steel. This eliminates the need for shear walls in the basement, but means that the ends of the steel beam must be anchored to the concrete walls to resist uplift.

Review the vertical section wall cut by plane **D** shown in Figure 3.37 for shear wall potential by the *perforated shear wall empirical method*.

There is no possibility to use the perforated shear wall empirical method. The segments have openings that traverse the full height of the wall plane.

6. Figure 3.38 illustrates the vertical planes **E** and **F** that define the extremities of **roof diaphragm #3**. Note that vertical plane **E** cuts through the roof diaphragm #1 much like vertical plane **D** above. It must be also noted that this time the garage and its two-door arrangement will influence our decisions.

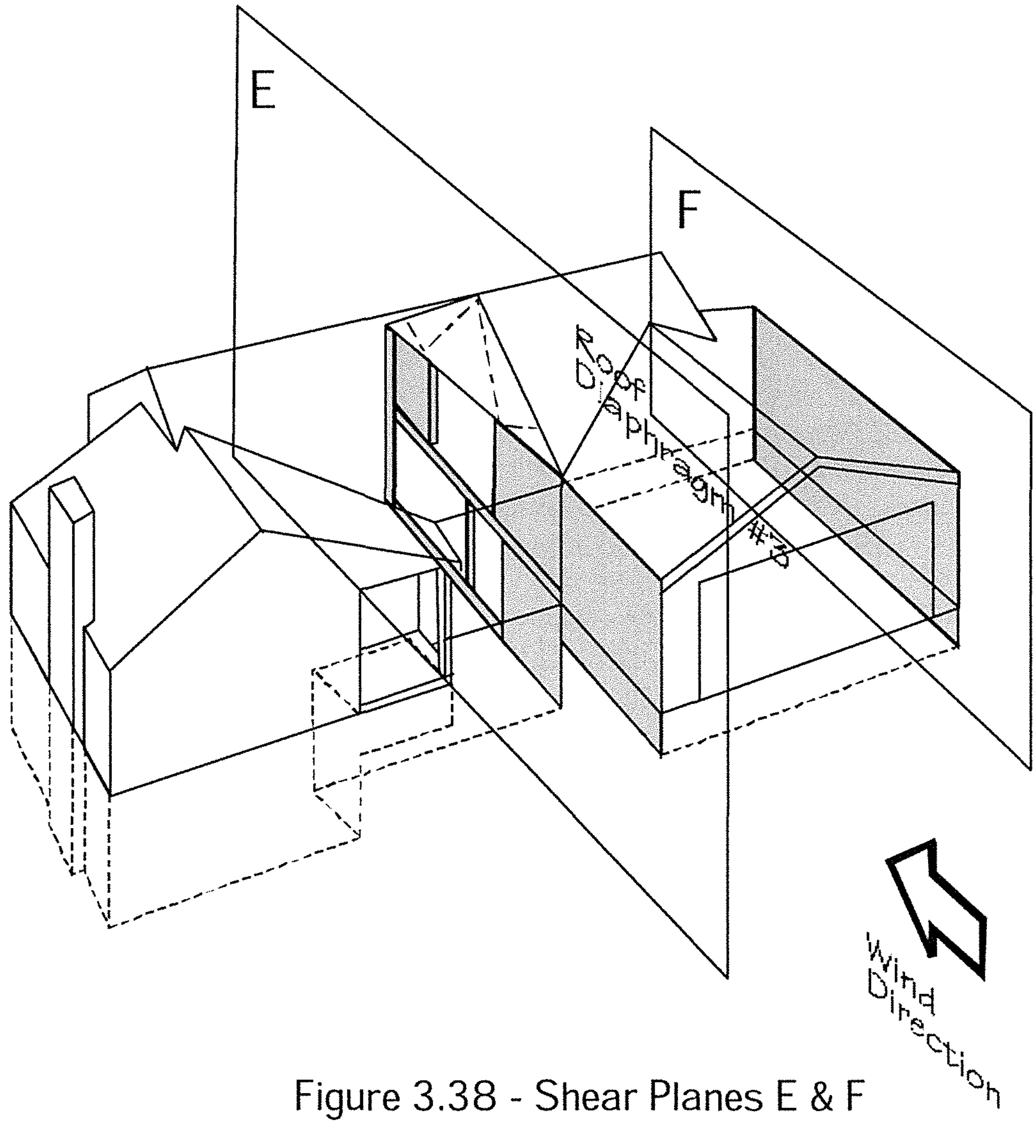


Figure 3.38 - Shear Planes E & F

7. The vertical planes **E** and **F** in Figure 3.38 cut through the wall section and exterior wall shown in Figures 3.39a, 3.39b and 3.40 respectively. They show in detail what can be used for the stability system related to roof diaphragm #3.

   a. Review the building section cut by vertical plane **E** illustrated in Figure 3.39a by the *traditional shear wall design method*.

      Three potential interior stud walls can be used as shear walls. A similar discussion to the one for the building section cut by vertical plane **D** occurs similarly here. Thus, the discussion will be abbreviated.

      The roof diaphragm #1 could be considered to interact with the interior potential shear walls under the main roof trusses. Thus, there is no need for any shear walls in this location. However, the exterior garage wall and the piece of the interior wall that completes the garage wall in Figure 3.39a require special consideration. Chapter 1, Figures 7 and 8 of the *WMM*, illustrate when wind pressure is directed into the door opening. For that case, all the sides explode outward. The illustration also shows when the wind blows from the side opposite the garage door, all four sides implode. These two conditions produce wind pressures or suctions that are at least 50 to 100 % greater than when the door is closed, as stated near the beginning of this chapter. **It is recommended that all the garage walls be anchored with tie-downs, have shear walls on all four walls and provide for the increased uplift.**

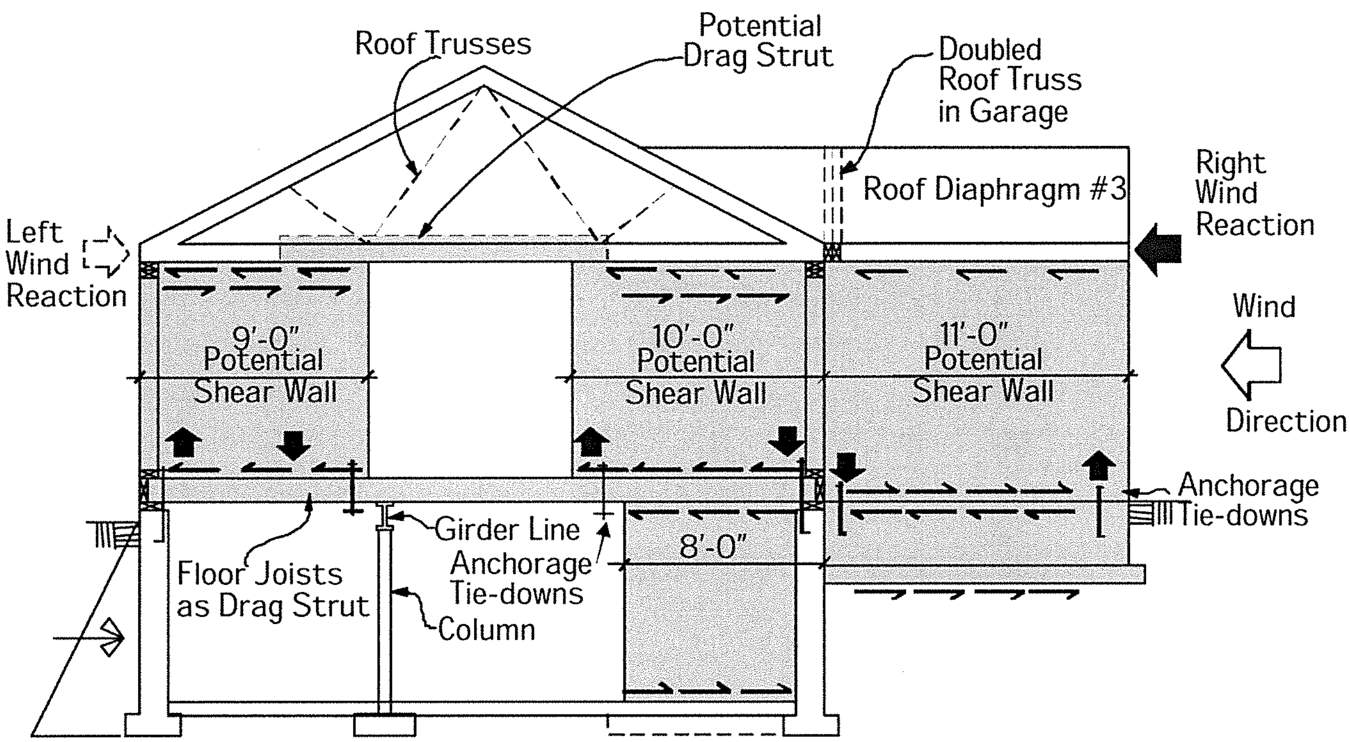


Figure 3.39a - Building Section at Plane E

Figure 3.39b shows the best choice of shear walls for the vertical section cut through plane **E**. It only utilizes the 20 foot long garage wall, which has a combination basement and slab-on-grade foundation condition underneath the garage wall length.

b. Review the vertical section wall cut by plane **E** shown in Figures 3.38 and 3.39b for shear wall potential by the *perforated shear wall empirical method.*

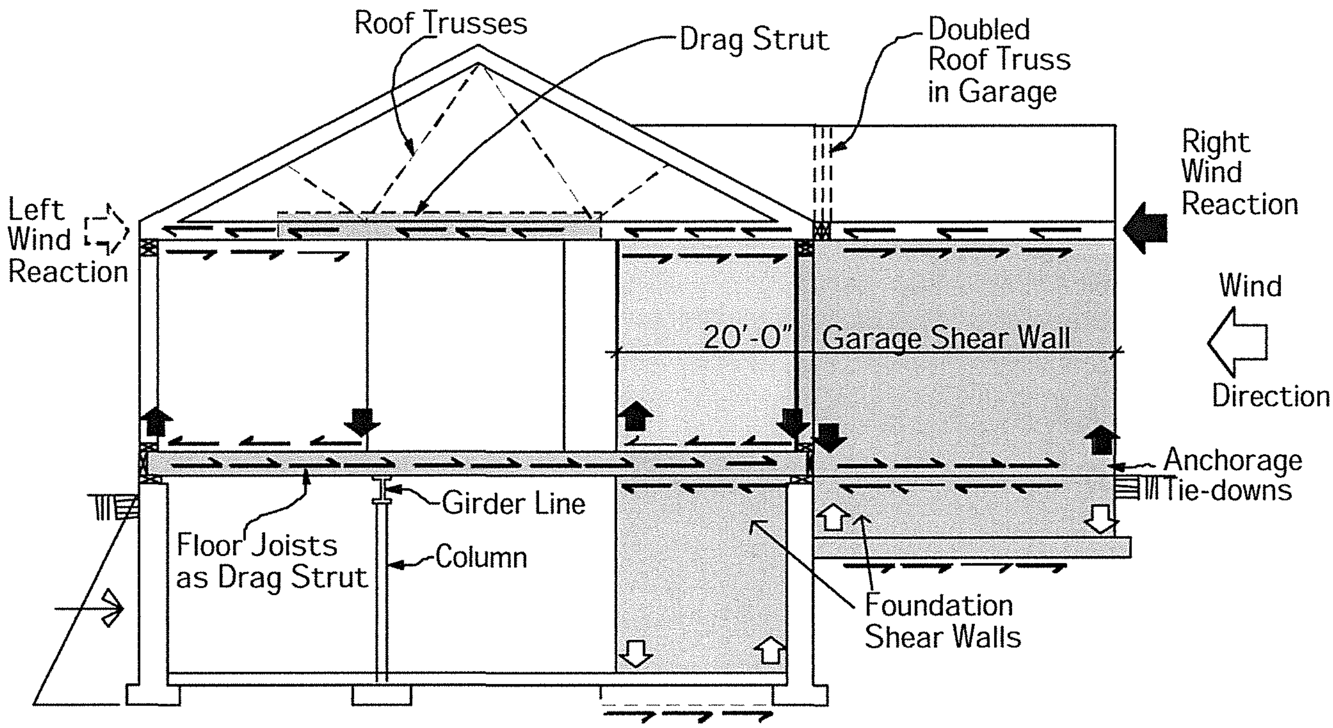


Figure 3.39b - Building Section at Plane E

There is no possibility to use the perforated shear wall empirical method. The segments have openings that traverse the full height of the wall plane.

Review the exterior garage wall that lies in the vertical plane **F** shown in Figure 3.40 by the *traditional shear wall design method*.

This wall is the garage wall only. It has a foundation below that must provide the ballast against direct uplift and overturning, while friction between the bottom of the footing and the soil resists the sliding with an assist from the passive soil block on the leeward side of the garage. The wood stud wall is sheathed with structural sheathing to create the shear wall. Anchorage tie-downs at the ends of the wall handle the overturning. The anchor bolts spaced uniformly between bottom plate of the wall and the foundation resist the sliding.

Review the vertical section wall cut by plane **F** shown in Figure 3.40 for shear wall potential by the *perforated shear wall empirical method.*

There is nothing to be gained by the use of the perforated shear wall empirical method. The same design results.

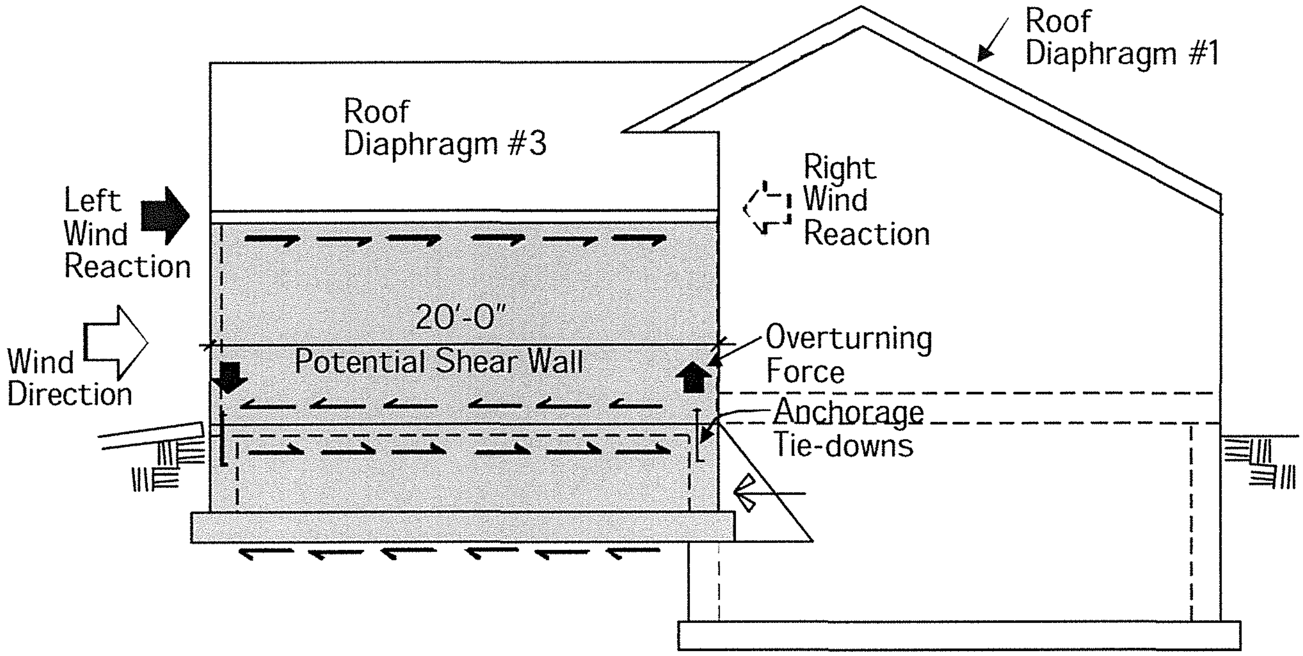


Figure 3.40 - Exterior Wall at Plane F

B. Wind Direction Parallel to the Main Roof's Ridge

1. The approach to wind resistance for the stability issues of overturning and sliding will not be done in its entirety as was done for the wind direction perpendicular to the main roof ridge. The same explanations would be presented and much repetition would result. Only the issues related to the garage will be presented.

2. Figure 3.41 illustrates the vertical planes **G** and **H** that define the extremities of **roof diaphragm #3** for the wind direction shown. Note that vertical plane **G** cuts through the roof diaphragm #1 much like vertical plane **D** above. It must be also noted that plane **H** includes the garage front elevation and its two-car door opening. Figures 3.42 and 3.43 illustrate the location of the two vertical planes in plan view for the first floor and the basement respectively.

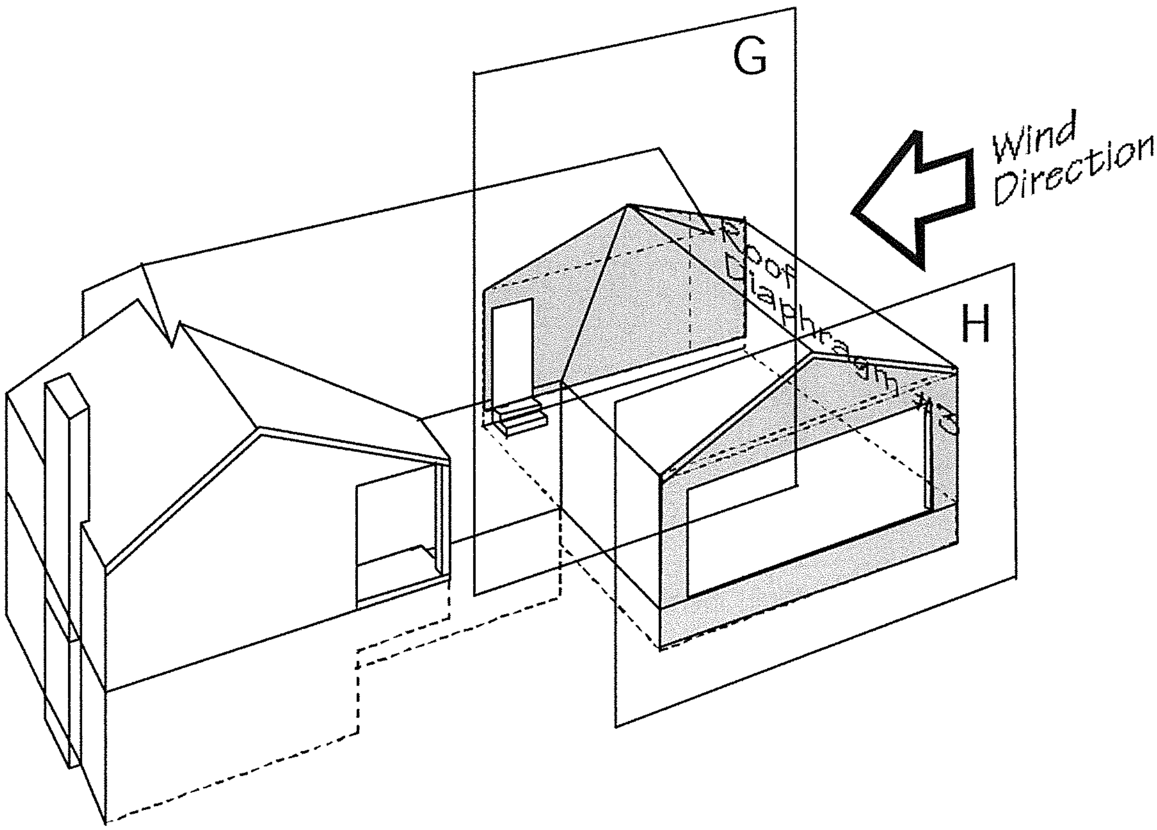


Figure 3.41 - Shear Planes G & H

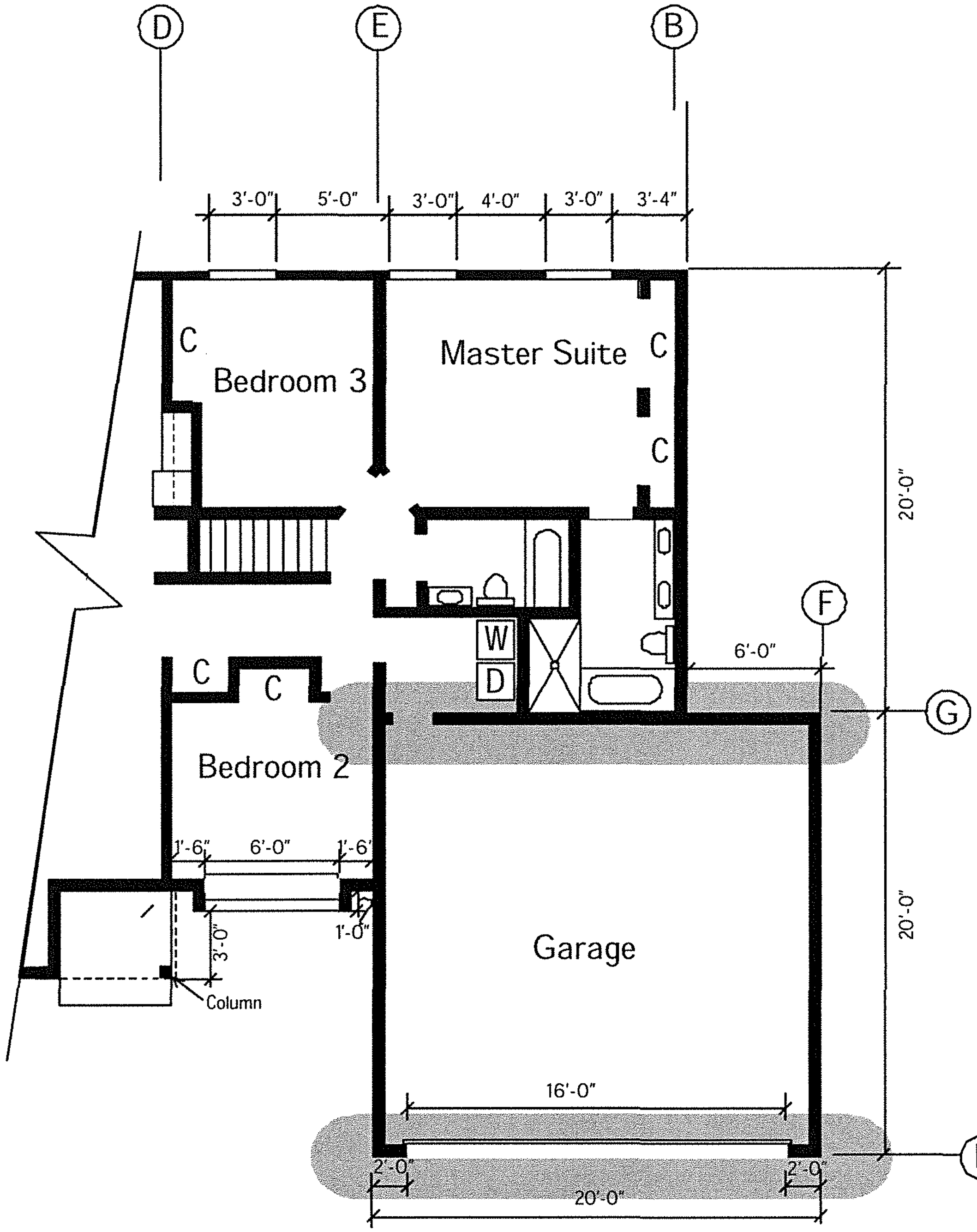


Figure 3.42 - Partial First Floor Plan

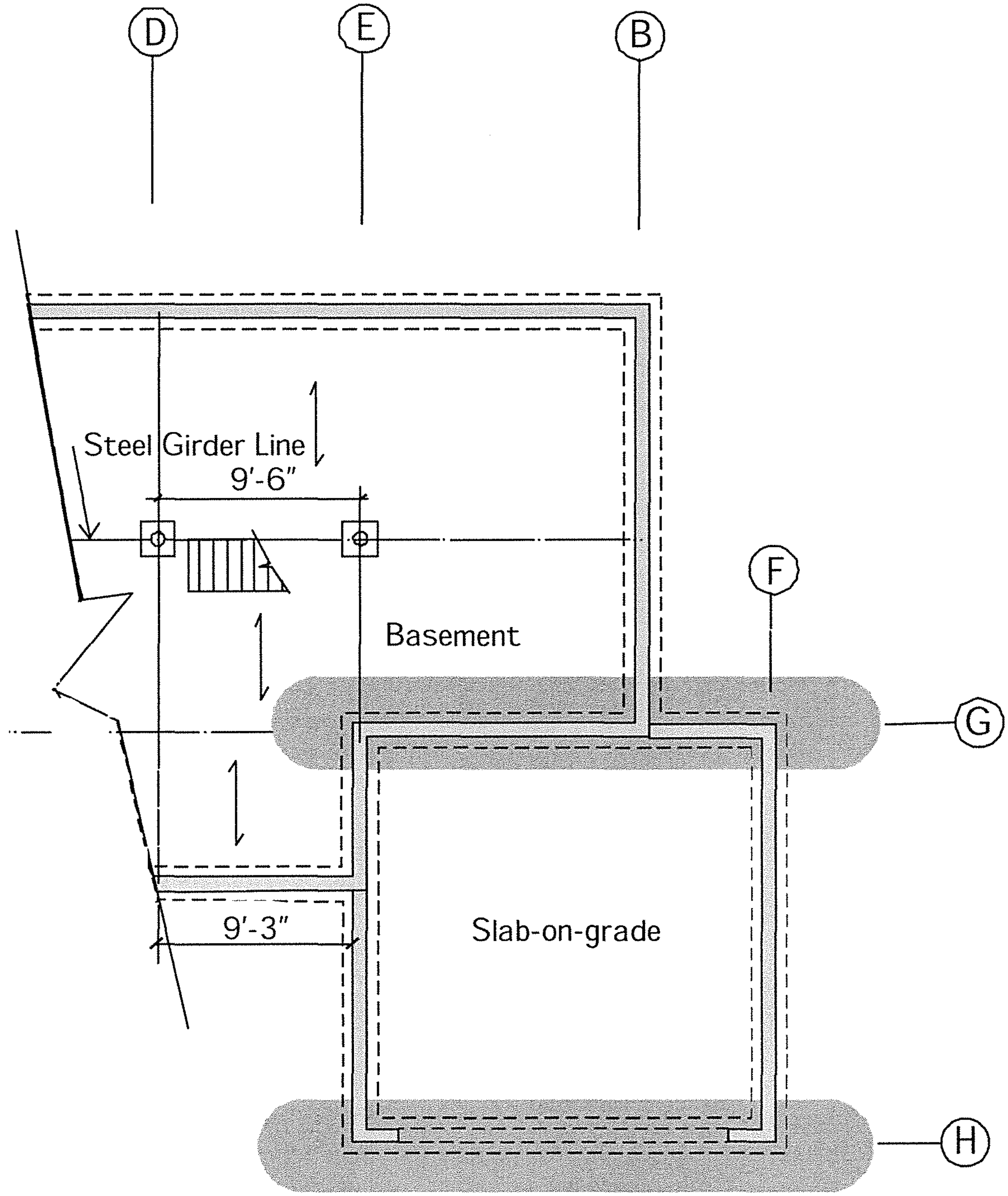


Figure 3.43 - Partial Foundation Plan

3a. Review the wall conditions that lie in the vertical plane **G** in Figure 3.44 by the *traditional shear wall design method.*

There is a substantial length of stud wall to act as a shear wall if the wind reaction can be transferred to it from the perpendicular roof trusses. The stud wall must be sheathed with structural panels. Dimension lumber blocking will be required between enough trusses to provide the requisite shear transfer to the top of the shear wall. This will be called a drag strut, and it must transfer the wind shear via metal connections to the shear wall. The shear wall must resist overturning and sliding to provide stability for this end of the garage. Tie-down anchorage is required at the ends of the wood shear wall. This anchorage is resisted by the concrete foundation directly below the stud wall. Sliding is resisted partly by the garage slab's friction against the soil and partly by the bottom of the foundation walls in friction against the soil.

The garage walls that are perpendicular to the direction of the wind are the windward and leeward walls that receive suction or pressure and must bend to resist the magnitude of that wind. When wind blows into the garage front elevation, even with the garage door closed and secure, these same walls are considered sidewalls that will again bend in response to the suction they receive. **All garage walls must be able to resist the pressures that result from wind in either wind direction, and for double that wind magnitude when the garage door is open.**

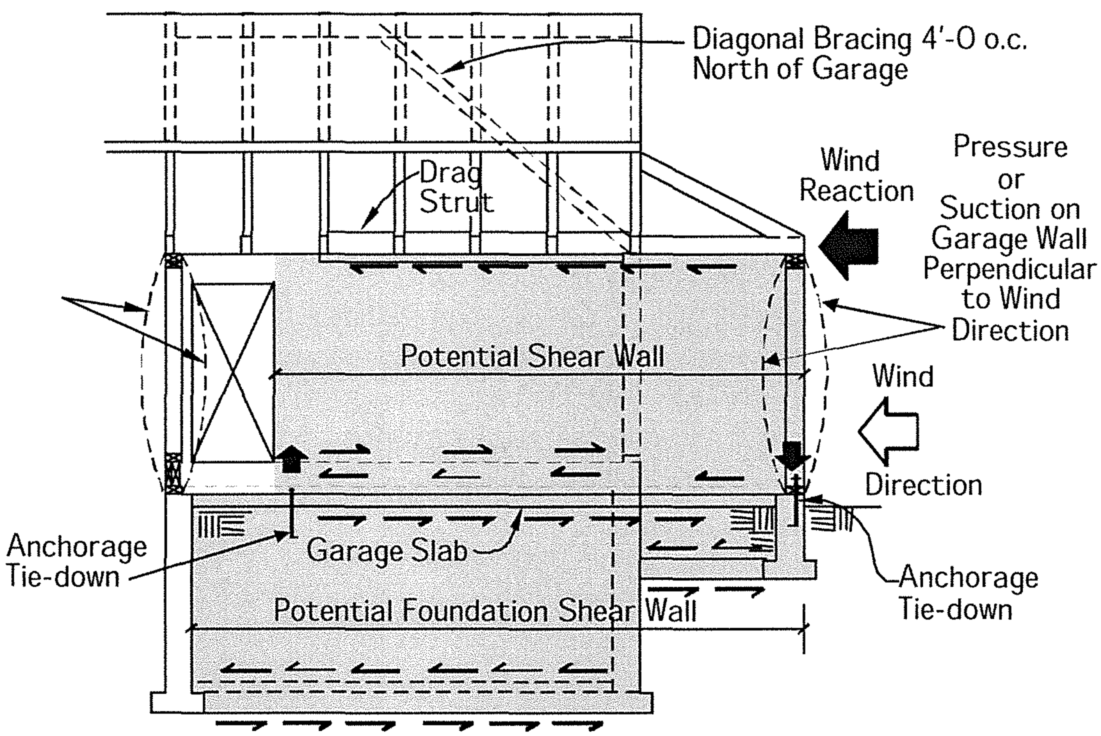


Figure 3.44 - Building Section at Plane G

3b. Review the vertical section wall cut by plane **F** shown in Figure 3.45 for shear wall potential by the *perforated shear wall empirical method*.

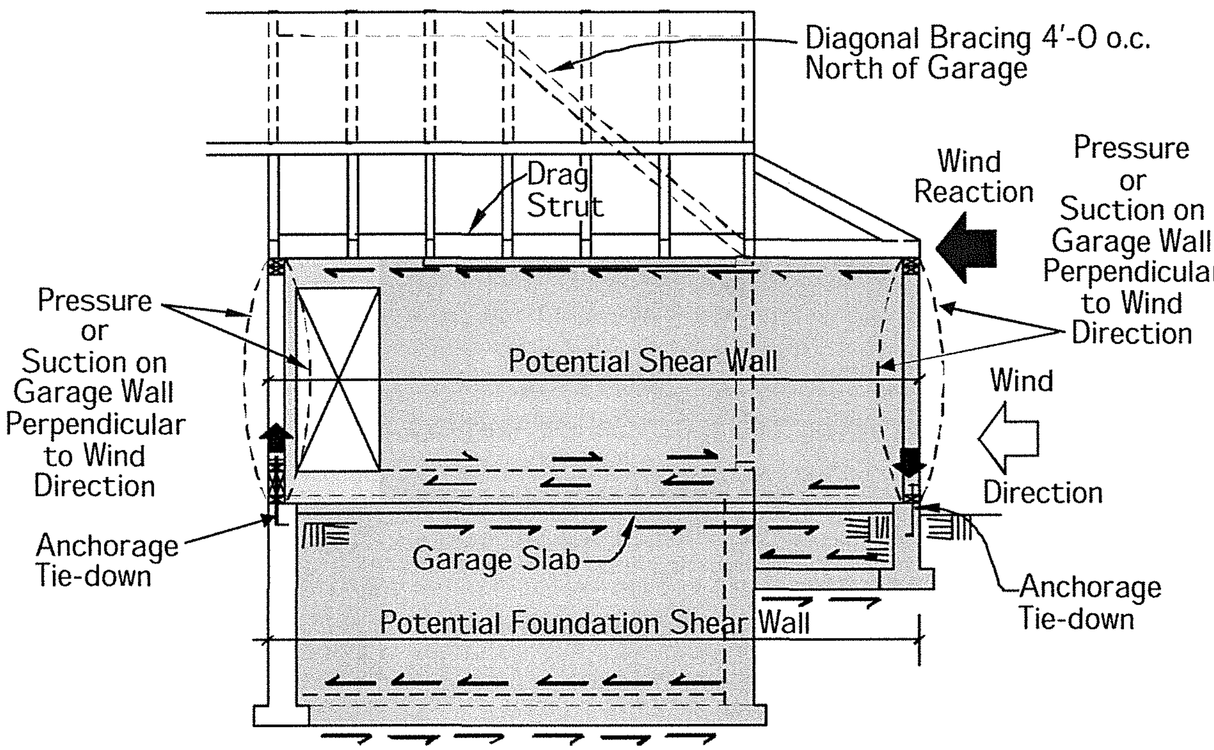


Figure 3.45 - Building Section at Plane G

4. Review the wall conditions that lie in the vertical plane **H**. The wood stud wall, within the vertical plane, contains the double garage door. This is a very flexible wall given that only two feet either side of the door is available as a potential shear wall. This is not an acceptable condition for stiffness, strength or overturning anchorage as illustrated in Figure 3.9 in this chapter. It is not likely that the garage can increase in size to apply the idea shown in Figure 3.10, nor will be possible to modify the design of the house to provide additional length as shown in Figure 3.10, items 4A and 4B. The only acceptable solution is the most expensive solution: a steel frame hidden around the door opening, as depicted in Figure 3.11. A structural engineer will be required to design and detail that steel frame.

5. The above steps have been done without numbers to avoid complicating the structural planning process. Chapter 5 will review another example with numbers and use the selection Tables of Appendix **B**.

Conclusions

This chapter has elaborated on the structural planning aspects of accommodating the essential aspects of stability for the residence; overturning and sliding. It has offered ideas for the architect or developer to consider in designing homes to make them more cost effective for wind resistance. Two examples, one simple and one complex, applied the principles described herein to illustrate and clarify their application.

Emphasis was placed on the garage's ability to provide adequate wind resistance, as this is a very vulnerable part of a residence in high wind conditions. The ability of the garage to withstand the pressures and suctions applied to it is instrumental in preventing the wind from attacking other adjacent parts of the house.

The two shear wall design methodologies were presented and discussed for both examples in this Chapter.

Post Script

Since this Manual was completed, new ideas have emerged with regard to stability for residential construction. The primary items include:

1. Prefabricated shear walls.

These engineered components will potentially reduce cost, assist compliance with the wind provisions of the model building codes, and reduce field labor to speed up construction. Five different companies have created different approaches to wind stability. The system names include CeeWal, Hardy Frame, Shear Max Panel, strong-Wall, and Z-Wall. Space does not permit including illustrations of these systems; however, their web sites are www.ceewal.com, www.hardyframe.com, www.shearmax.com, www.strongtie.com, and www.hawaii50.com/zwall respectively.

2. Drag-struts and tie-down anchors.

These engineered products are made from steel, are easy to install and avoid eccentricity between the tie-down and the double stud, where bending can occur. The manufacturer is Zone Four, website: www.zonefour.com.

3. Post-tensioned concrete block foundation walls

A technique intended to vertically anchor by post-tensioning, the concrete block foundation units without the need of grouting. Vertical steel tie rods run from the footing to the top of the block wall. Manufacturer is Dur-O-Wal at website: www.dur-o-wal.com.

Selected References

3.1 "Minimum Design Loads for Buildings and Other Structures", *American Society of Civil Engineers*, ASCE 7-93, p.8.

3.2 ASCE 7-93, p.17.

3.3 " DASMA Garage Door Wind Load Guide- Based on 1996 BOCA National Building Code", *Door and Access Systems Manufacturing Association,* January 1998, pp.1, 2.

3.4 "National Design Specification for Wood Construction", ANSI/AFandPA NDS-1997, 1997 Edition, *American Forest and Paper Association*, *American Wood Council,* p.78.

3.5 Douglas, and Sugiyama, H., "Perforated Shearwall Design Approach ", ASAE, Winter 1994.

3.6 "1996 Revisions to1994 Standard Building Code", *Southern Building Code Congress International*, Art. 2313.2, pp. 604-605.

3.7 "Commentary - WFCM Wood Frame Construction Manual for One- and Two-Family Dwellings", *American Forest and Paper Association and American Wood Council*, 1995 SBC, High Wind Edition, p.71.

Chapter 4 - Numeric Examples - Uplift Chain Resistance

Introduction

Chapter 2 described the structural planning issues related to the development of an uplift chain of resistance. It was a qualitative discussion of the design process, explored the various construction methods one might encounter, and the techniques to achieve the tension chain of resistance regardless of construction method. This Chapter will work a numeric example in sufficient detail to address a variety of construction methods. The example will illustrate step-by-step how to achieve an uplift resistance design system for one and two story residences. Tables found in Appendix A address typical construction conditions. The Tables are intended to minimize actual calculations.

One Story Residences

The Uplift resistance chain for all one-story residences will utilize the selection items found in Figure 4.1. The appropriate Design Tables in Appendix A are referenced in Figure 4.1 to provide a simplified approach to design of the Wind Uplift Resistance Chain.

General Design Procedure

1. Determine the geographic location for the residence. Use the Wind Speed map found in Figure 1.1. Regions where a 70-MPH wind speed is shown should be increased to 80 MPH. The minimum wind speed recommended by this author is **80 MPH**, and that is the minimum wind speed assumed by the Tables found in the Appendices.

2. Establish the following geometric variables for the residence:

   Building Width: This is the width (in feet) perpendicular to the ridge of the gable roof. If two intersecting gables occur, then two different building portions exist. There will be two building widths for such situations (see example 1). The Tables cover 20', 24', 28' and 32' only. It is permissible to interpolate between Tables for building widths that are between two Table width values.

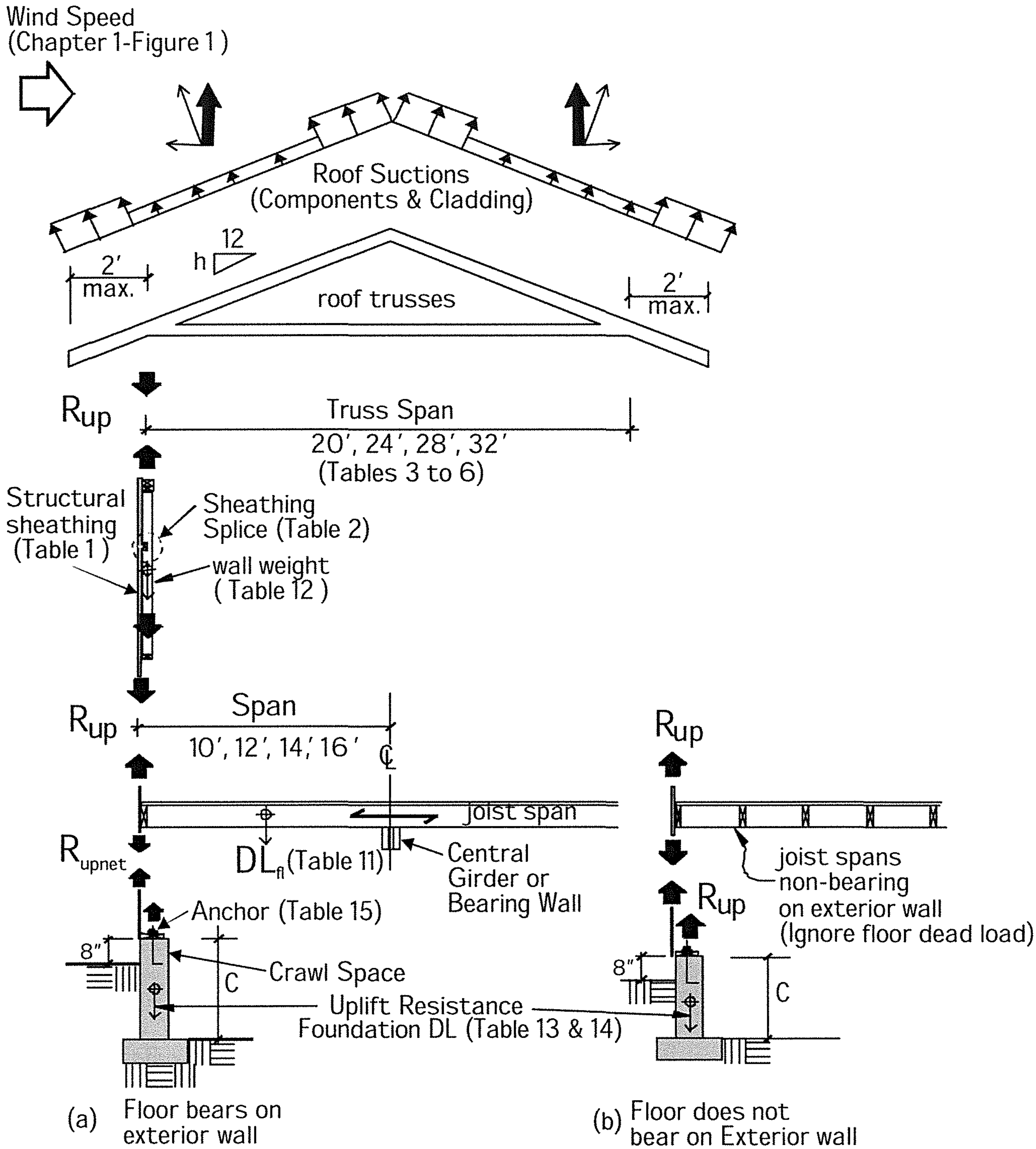


Figure 4.1 - One Story - Crawl Space

Roof Slope: This is the rise "**h**" in inches for each foot of roof projected length.

3. Select the appropriate Table (A.3 through A.6) in Appendix A. Each of these respective Tables is highlighted in large boldface type in the upper right corner (R_{up}) with building width below. Enter the selected Table with **wind speed** and roof slope "**h**" and read the uplift force in pounds per lineal foot of building length. This value is the direct wind uplift (R_{up}) for design from the roof down to the first floor. *The uplift force at each truss depends on the spacing of the trusses. Based on the spacing of the trusses, select the appropriate multiplier at the bottom of Table A.3 through A.6 to determine the uplift force at each truss.*

4. Determine the dimensional relationship between roof truss spacing and wall studs spacing. Verify whether the truss and stud spacing are the same, and if they are in alignment or not. See Figure 2.4 on page 9 and follow the discussion and illustrations on pages 9-12. This will help to determine which metal connector to select from the many types available to attach the roof trusses to the wall. Select the connector from manufacturer catalogs [4.1] & [4.2] based on an uplift capacity greater than R_{up} x **multiplier** value taken from the bottom of Tables A.3 – A.6 in Appendix A. These multiplier values are based on anticipated connector spacing. See the last two sentences (in Italics) of item 3.

5. a. Structural panels used to develop tension chain resistance

Select Table A.1in Appendix A to determine the appropriate structural panel to sheath the exterior stud wall. You must decide if the sheathing will have its face grain parallel or perpendicular to the vertical tension force direction. Move down the appropriate (T_{all}) column until you find a value larger than the (R_{up}) value selected in step 3. This will provide the thickness (**t**) and span rating for the structural panel that has ($T_{all} > R_{up}$). This is the panel you will select. Even if thinner panels work, it is this author's recommendation to select ½ inch as a minimum panel thickness.

b. Use of metal connectors to develop the tension chain resistance

Based on the roof truss and wall stud spacing and alignment results of step 4, appropriate metal connectors can be selected. Reference is again directed to Figure 2.4 (pg.9) and the discussion and illustrations on pages 9-12. Choice of metal connectors to tie the wood studs to the top and bottom plate are based on this alignment or non-alignment between studs and roof truss. Pages 10 to 18 in Chapter 2 offer details for any framing arrangement. Consult manufacturer's catalogs for allowable metal connector uplift capacities. It is required that the connectors selected satisfy: Connector Capacity > R_{up} x **multiplier.**

6. a. Structural panels used to develop tension chain resistance.

Determine arrangement of structural panels to cover the entire wall height from the top of the double top plate to the bottom of the mudsill. Butt splice locations must transfer the uplift force R_{up}. It was presented in Figure 11 of Chapter 4 in the *WMM* (pp. 37-38), that it is impossible to sheath the walls from a typical 8-foot tall structural panel, thus a splice is required at some point. The splice location must have wood blocking behind the structural panel between each vertical stud as shown in Figure 2.9 in this Manual. The required nailing to complete the tension chain past any butt splice is based on the uplift force R_{up}.

Select Table A.2 in Appendix A to determine the appropriate nailing at the butt spliced panel locations with the 2x4-splice plate behind. Enter the Table with the selected structural panel thickness "**t**" from step 5. Move down that column until a common or box nail capacity seems worth testing. Generally try for the box nail capacities first, to avoid splitting. Also, avoid spacing nails closer than 2 inches o.c. The following equation will determine the spacing (**S** in inches) for the nail capacity selected: **S** = (nail capacity/ R_{up}) x 12in/ft.

b. Use of metal connectors to develop the tension chain resistance

If insulation board is used to sheath the exterior stud wall, it is essential to attach metal connectors at the double top plate and the bottom plate to the studs. Manufacturer's catalogs provide capacities of these connectors for direct uplift. The required condition to satisfy is the selected connector provide sufficient capacity to resist the wind uplift force: Connector Capacity > R_{up} x **multiplier.**

7. Determine the weight of the exterior wall ($\mathbf{Wall_{DL}}$).

Select Table A.12 in Appendix A. Choose from ½"gypsum board or 1" plaster on wood lath for an interior wall finish, and use structural panels for the exterior material. You must also know the stud spacing for the wall.

8. Determine whether the floor joists bear on the exterior foundation wall.

If the floor framing spans perpendicular to the exterior wall select Table A.11 in Appendix A. This Table is highlighted in large boldface type in the upper right corner with ($\mathbf{DL_{fl}}$) **.** Enter the Table with the span to the interior girder or wall and the spacing of the floor framing. Select the dead load value ($\mathbf{DL_{fl}}$) in pounds per lineal foot of building length. Subtract that dead load from the (R_{up}) value determined in step 3. Also, subtract the stud wall dead load ($\mathbf{Wall_{DL}}$). The resulting value is the design net uplift (R_{upnet}) at the foundation. For the following conditions the R_{upnet} equations are:

a. When the exterior foundation wall supports the floor framing and the floor framing laps over the central girder:

$$R_{upnet} = R_{up} - \mathbf{Wall_{DL}} - \mathbf{DL_{fl}}$$

b. When the floor framing does not bear on the foundation wall:

$$R_{upnet} = R_{up} - \mathbf{Wall_{DL}}$$

c. When the exterior foundation wall supports the floor framing and the floor framing is continuous over the central girder:

$$R_{upnet} = R_{up} - \mathbf{Wall_{DL}} - \mathbf{3/4 \times DL_{fl}}$$

Figure 4.2 illustrates the two construction approaches to the installation of floor joists. The simple span implies a lapping of joists over the girder or interior stud wall partition. The continuous span implies that one long joist spans over the interior girder or stud wall partition. The amount of floor dead load that goes to the exterior bearing wall foundation depends on which approach to floor framing is being employed. This explains the difference between the equation for R_{upnet} in (a.) and (c.) above. Less floor dead load goes to the exterior bearing foundation wall when the joists are continuous. Contractors prefer placing long continuous joists to minimize placement time. Thus engineered floor members such as solid web "I" joists or 4 x 2 floor trusses would be preferred over dimension lumber. Available lengths for dimension lumber seldom exceed 20 to 24 feet, while engineered joists or trusses can easily be obtained in 30 to 40 foot lengths.

9. Determine the wall weight at the gable ends.

Gable end wall weight will include a triangle of wall above the typical rectangular wall. This is illustrated in Figure 4.3 for two conditions: (1) a stud wall of varying height makes up the gable triangle; or (2) a special fabricated end wall truss without diagonals to complete the gable end wall.

The following approach is recommended by the author:

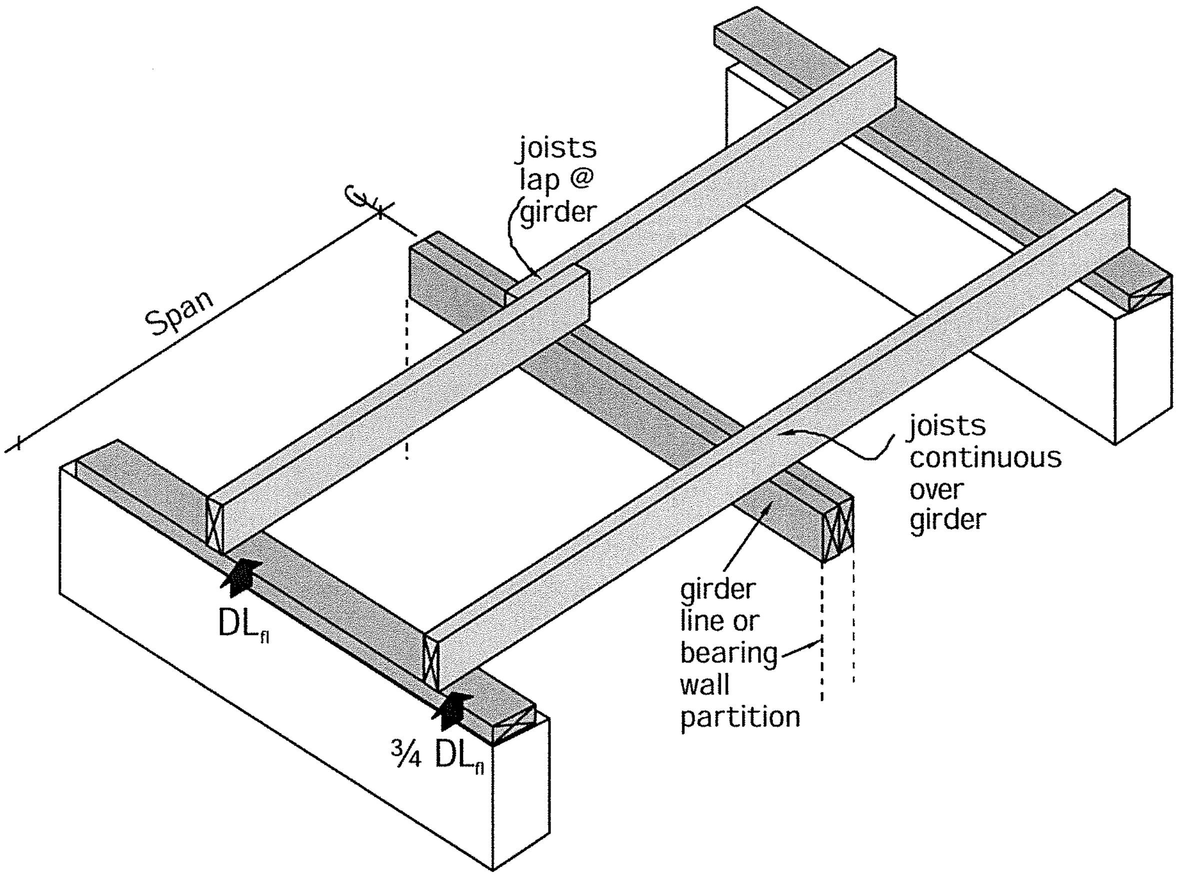


Figure 4.2 - Simple vs. Continuous Floor Joists

a. Select Table A.16 in Appendix A and enter the Table with roof slope vertical component (**h**) and the building width. Read $\mathbf{H_{avg}}$ in feet, which is the average gable wall height if uniformly transformed into a rectangle as shown in the illustration.

b. Select Table A.12 in Appendix A and enter the Table at the heading "gable end". Based on 2x4 or 2x6 members comprising the gable end triangle, read wall weight for 12", 16" or 24" spacing of vertical elements in the gable end triangle.

c. Multiply the R_{up} by one foot, assuming the maximum truss spacing of 24 inches, to arrive at a wind uplift force per foot along the entire gable. This is conservative, but justified on the basis of the typical flexibility of gable end walls.

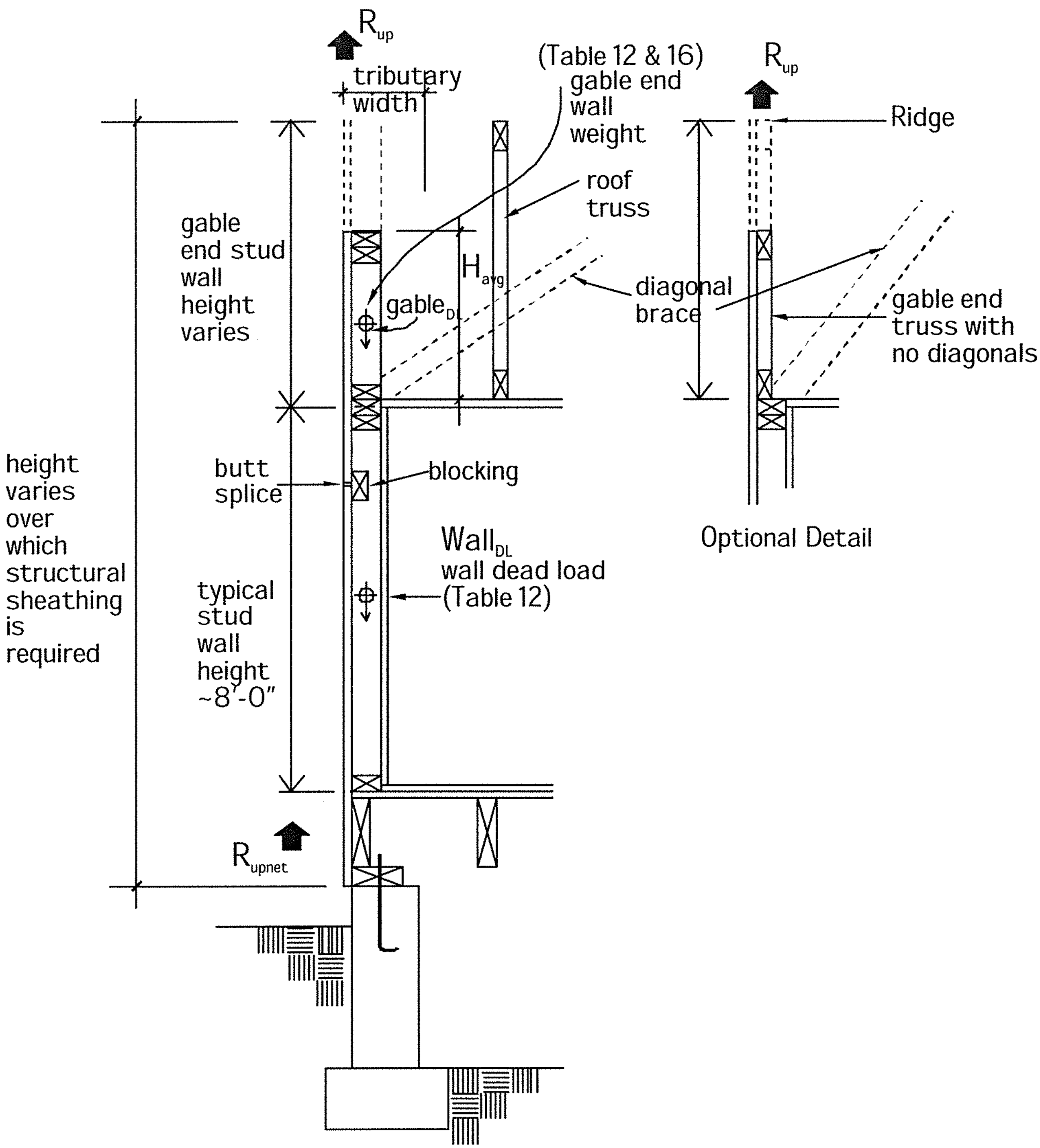


Figure 4.3 - Gable End Wall Construction

d. Structural panels should traverse the horizontal joint between the double top plate of the rectangular wall and the bottom plate of the triangular gable end. This author does not recommend the use of insulation board sheathing and metal connectors on gable end walls for tension chain uplift resistance. Use Table A.1 and A.2 of Appendix A to verify selection of panel thickness and type and butt splice nailing and blocking requirements respectively. This process has been described in step 5 and 6 above.

e. The final equation for the triangular gable end wall weight is: $\mathbf{G_{DL}} = \mathbf{H_{avg}} \times \mathbf{Wall_{DL}}$ in plf of wall length.

10. Net uplift (R_{upnet}) at the foundation of gable end walls depends on several construction conditions, and results in several equations as follows:

a. When the exterior foundation wall supports the floor framing and the floor framing laps over a central girder:

$$R_{upnet} = R_{up} - \mathbf{Wall_{DL}} - \mathbf{DL_{fl}} - \mathbf{G_{DL}}$$

b. When the floor framing does not bear on the foundation wall or when it is a garage wall with a floating slab:

$$R_{upnet} = R_{up} - \mathbf{Wall_{DL}} - \mathbf{G_{DL}}$$

11. Select the foundation that will provide sufficient weight to resist the R_{upnet} wind uplift force.

Select Table A.13 for crawl space homes or Table A.14 for basement homes from Appendix A as construction conditions dictate. Enter either Table to find a value in pounds per foot of building length that exceeds the R_{upnet} uplift force for either a grouted concrete block wall or a cast-in-place (CIP) concrete foundation wall. Choose the assembly that your contractor deems most economical for his operation. Perform this operation for the various types of foundation conditions applicable to your residence, i.e., crawl, basement, garage, bearing wall or non-bearing wall.

12. Select the anchor bolt spacing and washer at the connection between the superstructure and the foundation.

Table A.15 in Appendix A provides vertical anchor capacities for various anchor spacing and choice of washer. The following relationship must be satisfied:
Anchor Bolt Resistance > R_{upnet} x **selected spacing.**

13. Detail the special uplift condition at the ends of the garage door opening. Refer to Chapter 2 – "Uplift Recommendations for Sections of Walls with Openings".

    The uplift force R_{up} and the span $\mathbf{H_L}$ of the opening are used to determine the concentrated force $\mathbf{R_H}$ for design of the mechanical anchor and/or structural panel. The equation is : $\mathbf{R_H} = R_{up} \times \mathbf{H_L} \div 2$

Example 1 - Structural Wood Panel Option

Given:

A one story wood platform framed residence located in Urbana, IL. Figure 4.4 is a perspective of the home and attached garage. The roof is framed with wood metal plated trusses, and the roof plan is shown in Figure 4.5 following the perspective. Note there is a 1'-6" overhang at the low end of all gables. The crawl space foundation wall is composed of insulated forms with cast-in-place concrete and vertical and horizontal reinforcement. The first floor framing is composed of solid web engineered "I" joists, spanning between the exterior walls and bearing on the interior girder line. The joists are continuous over the central girder. The girder is bearing on concrete piers spaced 11' apart. The combination floor framing and foundation plan is show in Figure 4.6. A typical wall section is shown in Figure 4.7 and an alternate foundation is shown in Figure 4.8. All of these Figures follow in order on the following pages.

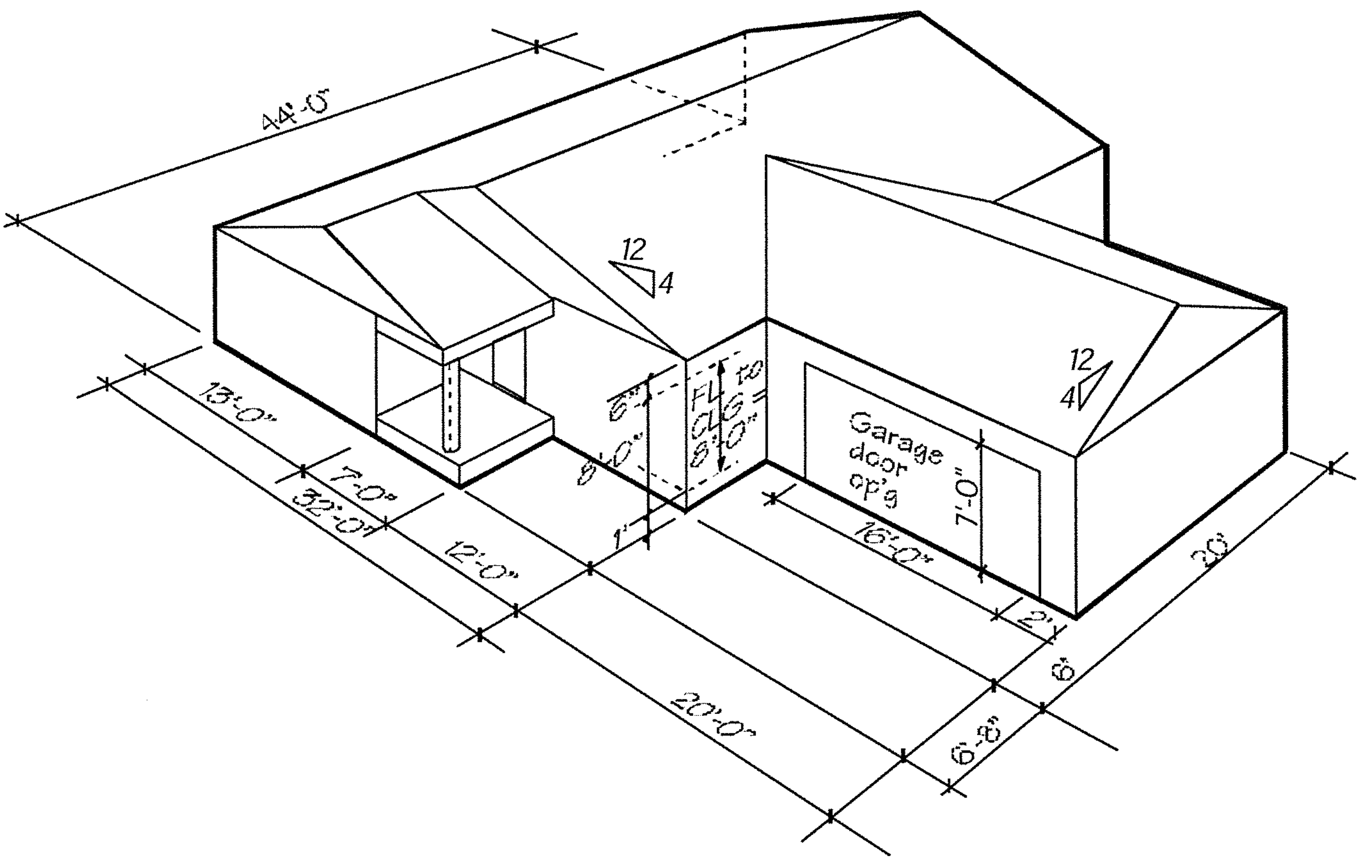


Figure 4.4 - Perspective of One Story Residence

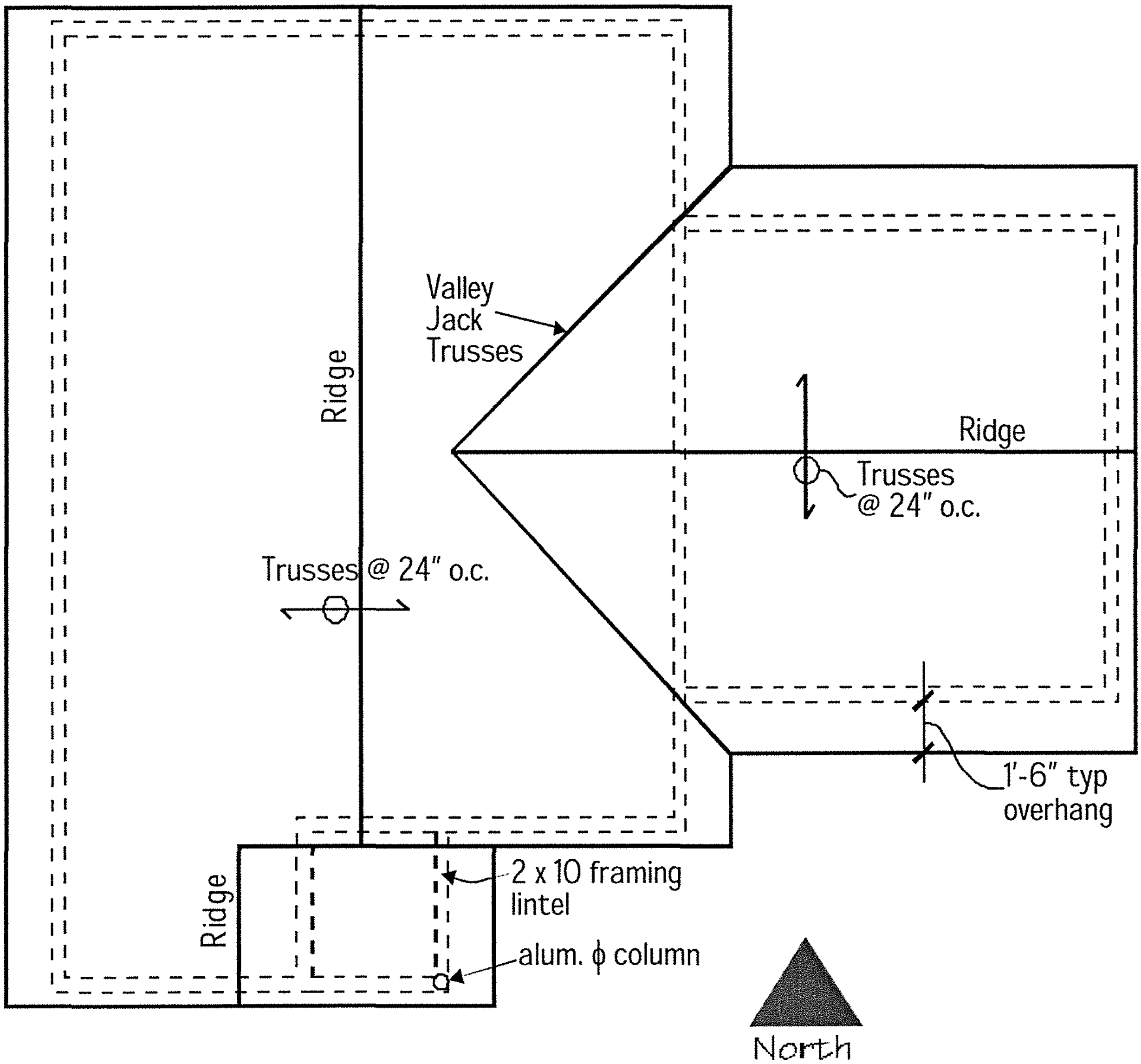
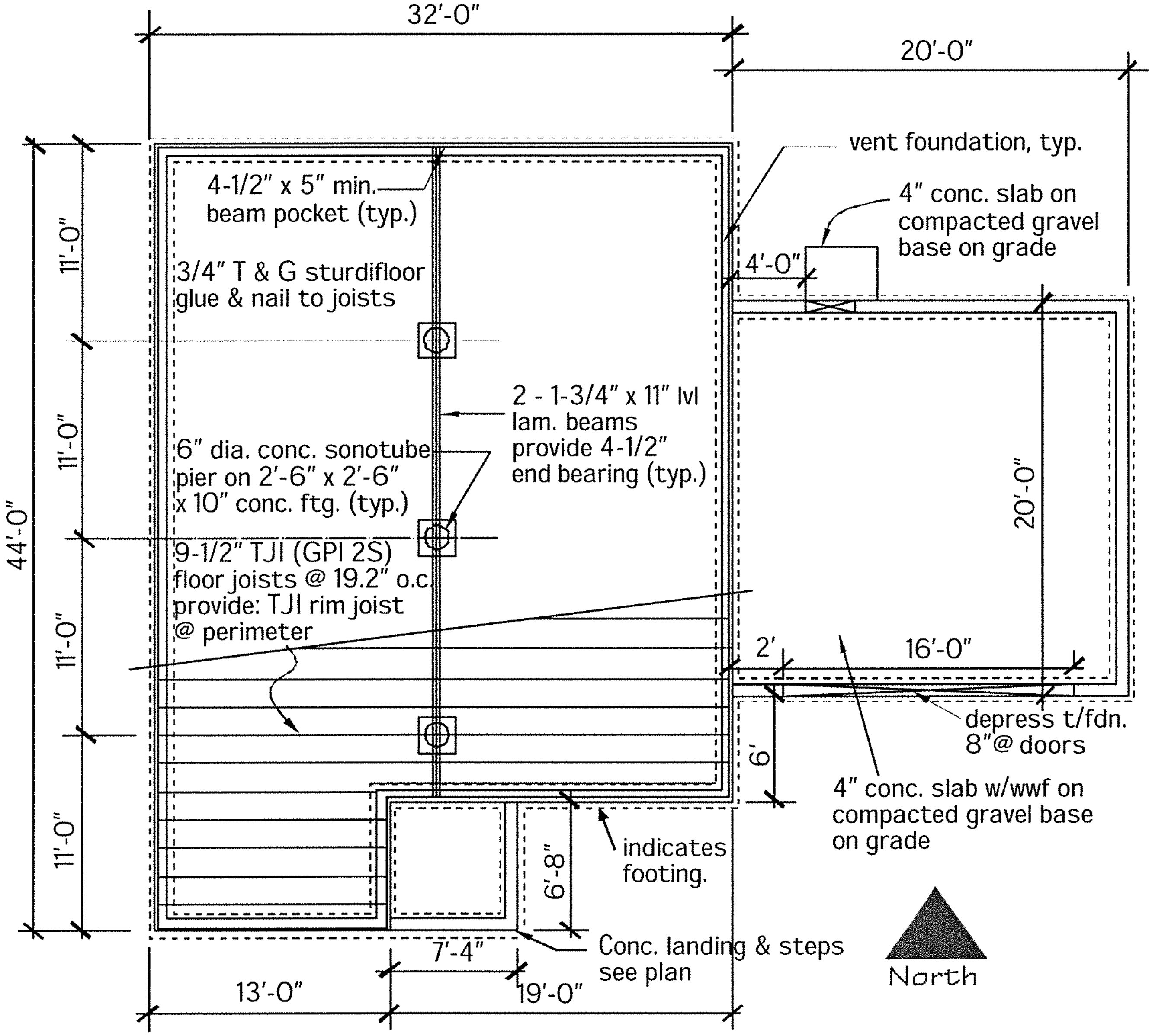


Figure 4.5 - Roof Plan



Note: Where joists run parallel to exterior wall, leave a clearance of 9" between the first floor joist and the inside face of stud.

Figure 4.6 - Foundation & Framing Plan

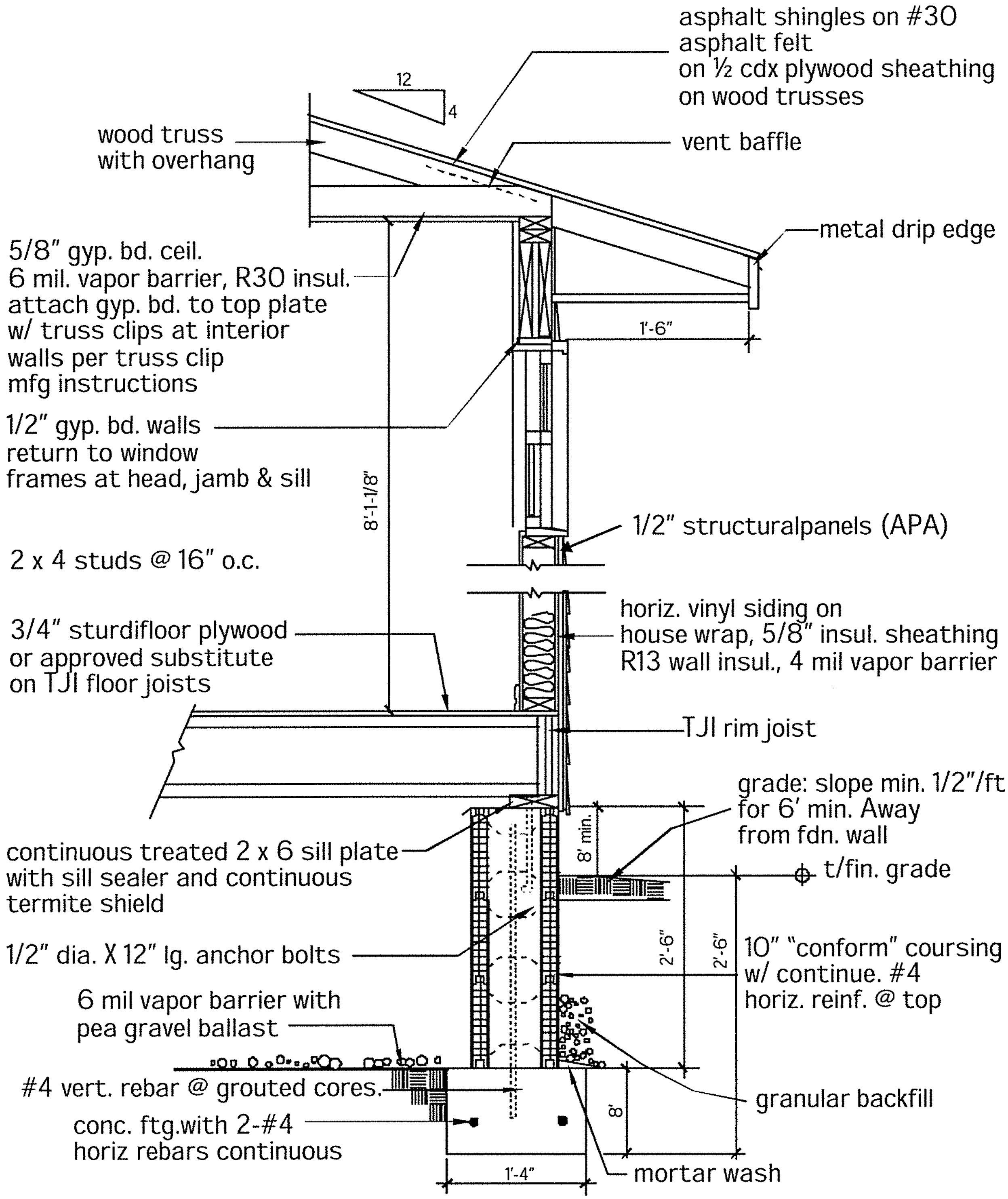


Figure 4.7 - Typical Wall Section

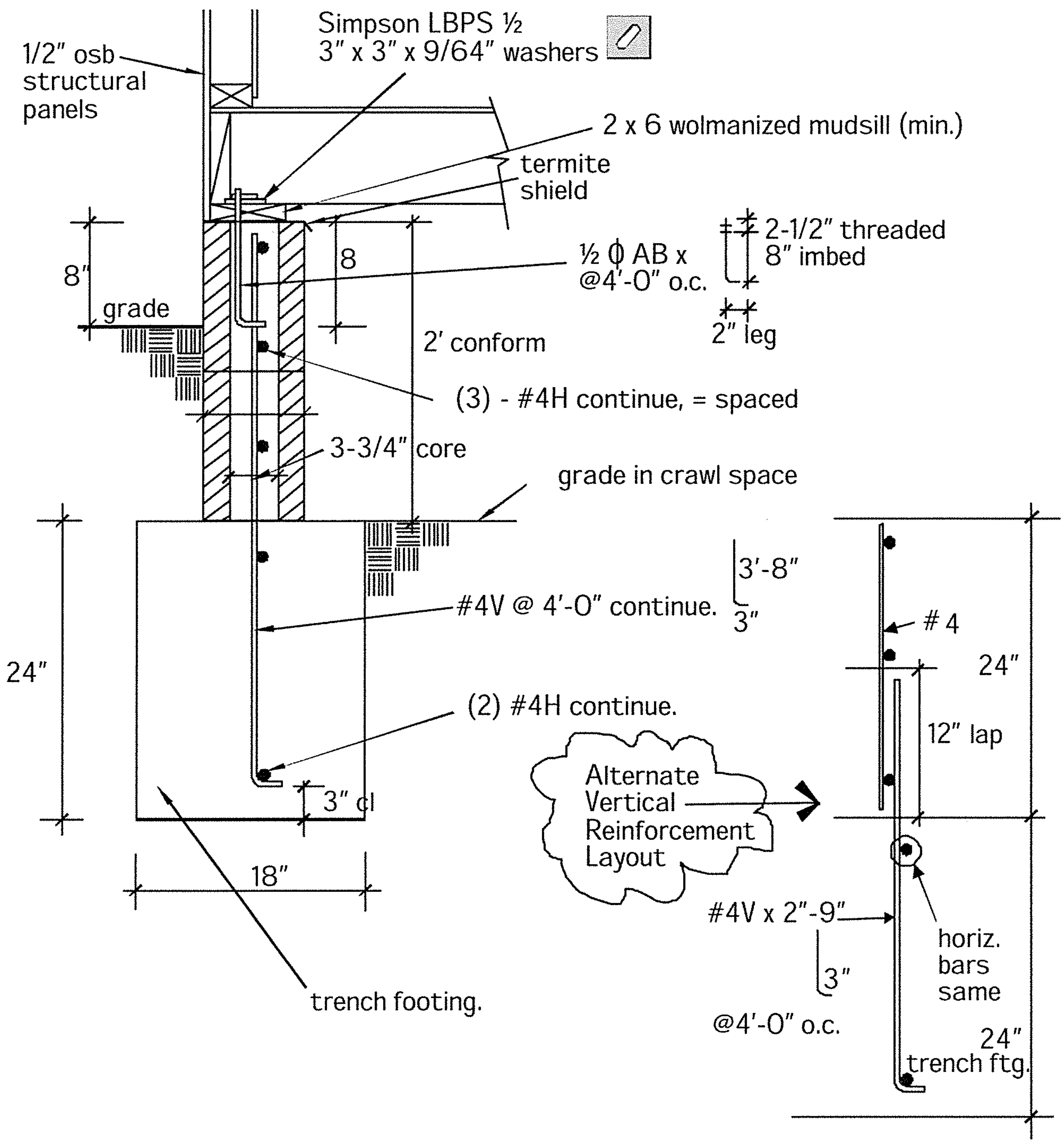


Figure 4.8 - Alternate Foundation Detail

Solution:

Follow the procedure outlined in the previous section:

1. The Residence is located in Urbana, IL. The ANSI 7-93 Wind Map indicates a 70-MPH wind speed. This is less than the minimum recommended by this author. A discussion with the building department in Urbana indicates that they have adopted the 1996 BOCA Code, which also advocates a 70-MPH wind speed; however they advocate and promote a 90-MPH wind speed. Incentives to contractors, builders and developers are described in Chapter 1, regarding the City of Urbana and their rebate program.

   Use 90-MPH wind speed

2. Establish geometric parameters:

   House: **Roof slope is 4 in 12**
   Roof Truss Span = 32 feet.

   Garage: **roof slope of 4 in 12.**
   Roof Truss Span = 20 feet.

3. Refer to Table A.6 in Appendix A. This Table has the upper right hand guide mark R_{up} **32** to ease the appropriate Table selection process. Enter the "Direct Uplift" portion of the Table with 90 MPH and the roof slope category 3:12 to 6:12. Where these two variables cross, read 386 pounds per foot of building length. This is the uplift force at the roof truss to wall connection along the east and west walls of the basic house. Since the roof trusses are spaced at 24" o.c., multiply R_{up} x 2' = 772 pounds/truss uplift.

   House: East and West walls: R_{up} **= 386 plf**
   Uplift at each roof truss end = 772 Lbs. @ 2'-0"o.c.

   Refer to Table A.3 in Appendix A. This Table has the upper right hand guide mark R_{up} **20** to ease the appropriate Table selection process. Enter the "Direct Uplift" portion of the Table with 90 MPH and the roof slope category 3:12 to 6:12. Where these two variables cross, read 280 pounds per foot of building length. This is the uplift force at the roof truss to wall connection along the north and south garage walls. Since the roof trusses are spaced at 24" o.c., multiply R_{up} x 2' = 560 pounds/truss uplift.

   Garage: North and South walls: R_{up} **= 280**

plf

   Uplift at each roof truss end = 560 Lbs. @ 2'-0"o.c.

4. There is no intentional relationship between the alignment of the roof trusses and the wall studs suggested by the contractor for the house or the garage. It will be assumed that the truss/stud relationship is random; therefore, the metal connector types **G**, **V** or **H** are appropriate choices as found in Figure 2.6a. The logical choice would be the connector that has a capacity close to the required uplift force (R_{up} x **multiplier**) and engages both plates of the double top plate. Using the Simpson Strong-Tie Manual, select doubling connector **H2.5** (**H** type) with a single connector capacity of 415 pounds [4.3] with a 1.33 or a 1.6 wind factor included. The provided capacity is 2 x 415 = 930 pounds/truss which is greater than the 772 pounds required per truss for the house in step 3. The appropriate detail for this condition is shown in Figure 2.8c, illustration (B) of Chapter 2.

House: East and West wall connection to Trusses: Simpson: (2) H2.5
Capacity = 930 Lbs./Truss

The garage has less uplift force per truss and may use a different metal connector. The **G** type has the ability to engage both plates of the double top plate. Using the Simpson Strong-Tie Manual, select connector **H1** (G type in Figure 2.6a) with a single connector capacity of 585 pounds [4.4] with a 1.6 wind factor included. The provided capacity of 585 pounds per connector is greater than the 560 pounds required per truss. Figure 2.6a has a good illustration of this detail.

Garage: North and South wall connection to Trusses: Simpson: H1
Capacity = 585 Lbs./Truss

5. The solution in progress assumes that structural sheathing will be used from the top of the double top plate to the bottom of the mudsill to avoid continued use of metal connectors. Enter Table A.1 of Appendix A knowing $\mathbf{R_{up}}$ = 386 pounds per foot of length. Choose face grain of structural panel either oriented vertically with the uplift force or perpendicular to the uplift force. If all panels will be oriented vertically, read 3072 pounds per foot of length. This is the smallest allowable ($\mathbf{T_{all}}$) capacity, resulting in the use of either 15/32", 1/2" or 3/8" panel thickness. Since the shear walls have not yet been discussed, it is probably a good choice to use 1/2" sheathing with a span rating of 24/0. Exposure 1 material will allow for an extended construction period and the ability of the panels to remain stable for intermittent periods of rain. This choice also works for the garage where the $\mathbf{R_{up}}$ = 280 pounds per foot of length. It should be noted that if the panels were placed perpendicular to the vertical tension direction the allowable tension capacity $\mathbf{T_{all}}$ = 1317 pounds per foot of length, which is also more than the 386 pounds per linear foot required. Thus the structural panels could be placed either parallel or perpendicular to the vertical tension direction. Some contractors place the structural panels in a combination of directions.

House and Garage: use APA 1/2" sheathing with a span rating of 24/0 and Exposure 1 material. Panels may be placed vertically or horizontally or in a combination of the two.

6. Select Table A.2 from Appendix A to engineer a continuous tension chain past the butt splice location in the structural panels. The main house portion has the required $\mathbf{R_{up}}$ = 386 pounds per foot of length. The first trial will be 6d box nails. Enter the Table with that nail choice and the (**t**) selected above. Where the row and column cross read 72.1 pounds per nail. The following equation will determine the spacing (**S** in inches) for the nail capacity selected: **S**= (nail capacity/ $\mathbf{R_{up}}$) x 12in/ft . Thus, **S** = (72.1 / 386) x 12 = 2.24 inches. This close spacing may result in splits in the panels. A second trial might be 8d Common nails. Read 106.1 pounds per nail from Table A.2. Thus, **S** = (106.1 / 386) x 12 = 3.3 inches o. c. Use 8d Common nails at 3 inches on center. This also permits the blocking to be placed vertically, allowing more space for the batt insulation within the wall.

House and Garage: Use 8d Common nails at 3 inches on center at butt splices in the structural panels. A vertical 2x4 blocking is required between studs behind the butt joint.

7. Enter Table A.12 in Appendix A with the stud spacing and the interior and exterior wall finishes. From the section of Figure 4.7 note that the interior finish is ½" gypsum boards (drywall) and the exterior is now determined to be ½" structural panels. The wall framing is composed of dimension lumber 2 x 4 studs spaced at 16" o.c.. Read the wall weight as 3.2 pounds per square foot of wall surface. Since the wall height is slightly over 8 feet, the total wall weight: $\mathbf{Wall_{DL}}$ = 25.6 pounds per foot of wall length. Note that this wall weight is 2/3rds of the actual wall weight.

House and Garage: Exterior stud wall weight: $\mathbf{Wall_{DL}}$ = 25.6 plf

8. Reference Figure 4.6 to locate the spacing of the first floor joists running in the east/west direction at 19.2 inches o.c. These joists are continuous over the girder line and bear on the east and west foundation walls. The central girder line is 16 feet from the exterior wall. Enter Table A.11 in Appendix A and read the floor dead load $\mathbf{DL_{fl}}$ = 106.6 pounds per foot of wall length. Figure 4.9 illustrates the locations of bearing walls, where floor dead load $\mathbf{DL_{fl}}$ reduces the $\mathbf{R_{up}}$ value. The net uplift at the foundation for the east and west walls of the house is found from the formula in step 8(c.) of the procedure: $\mathbf{R_{upnet}} = \mathbf{R_{up}}$ - $\mathbf{Wall_{DL}}$ - **3/4 x** $\mathbf{DL_{fl}}$. Thus, $\mathbf{R_{upnet}}$ = 386 – 25.6 – (3/4) x 106.6 = 280.5 lbs. per foot of length.

House: Net Uplift at East and West Foundation walls: R_{upnet} = 280.5 plf

The garage has a floating slab-on-grade; thus Table A.11 is not required. Only the wall weight for the north wall reduces the uplift force R_{up} = 280 pounds per foot of length. Use formula (b) from step 8 above: $R_{upnet} = R_{up} - Wall_{DL}$ = 280 – 25.6 = 254.4 plf. This is the net uplift for selection of structural panel and for the butt splice nailing from Tables A.1 and A.2 respectively along the north garage wall.

Garage: Net Uplift at North foundation wall: R_{upnet} = 254.4 plf

The south garage wall contains the 16-foot door. There is no wall weight, except for the garage door opening header weight, which we will ignore for now. Thus, the roof uplift force R_{upnet} = 280 pounds per foot of wall applies.

Garage: Net Uplift at South garage door header: R_{upnet} = 280.0 plf

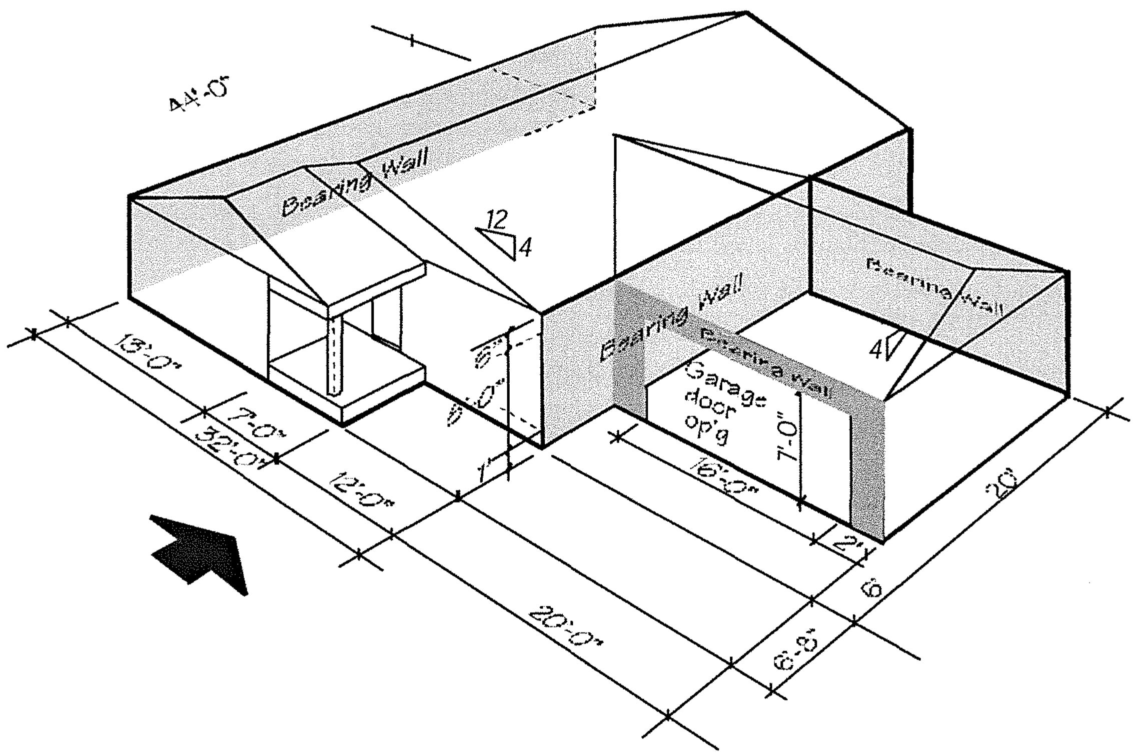


Figure 4.9 - Perspective of One Story Residence
Bearing Walls Highlighted

9. Gable end wall triangles exist at the top of the north and south walls of the house and the east wall of the garage. Figure 4.10 illustrates the gable end wall locations. Also, these three walls do not receive floor dead load. Figure 4.3 illustrates the tension chain conditions for a generic gable end wall. The uplift force is now multiplied by a one foot tributary width; therefore, the uplift force remains $\mathbf{R_{up}}$ = 386 pounds per foot along the elevation for the north and south house gable ends, and $\mathbf{R_{up}}$ = 280 pounds per foot along the east garage gable end wall. These are the vertical forces along the roof slope.

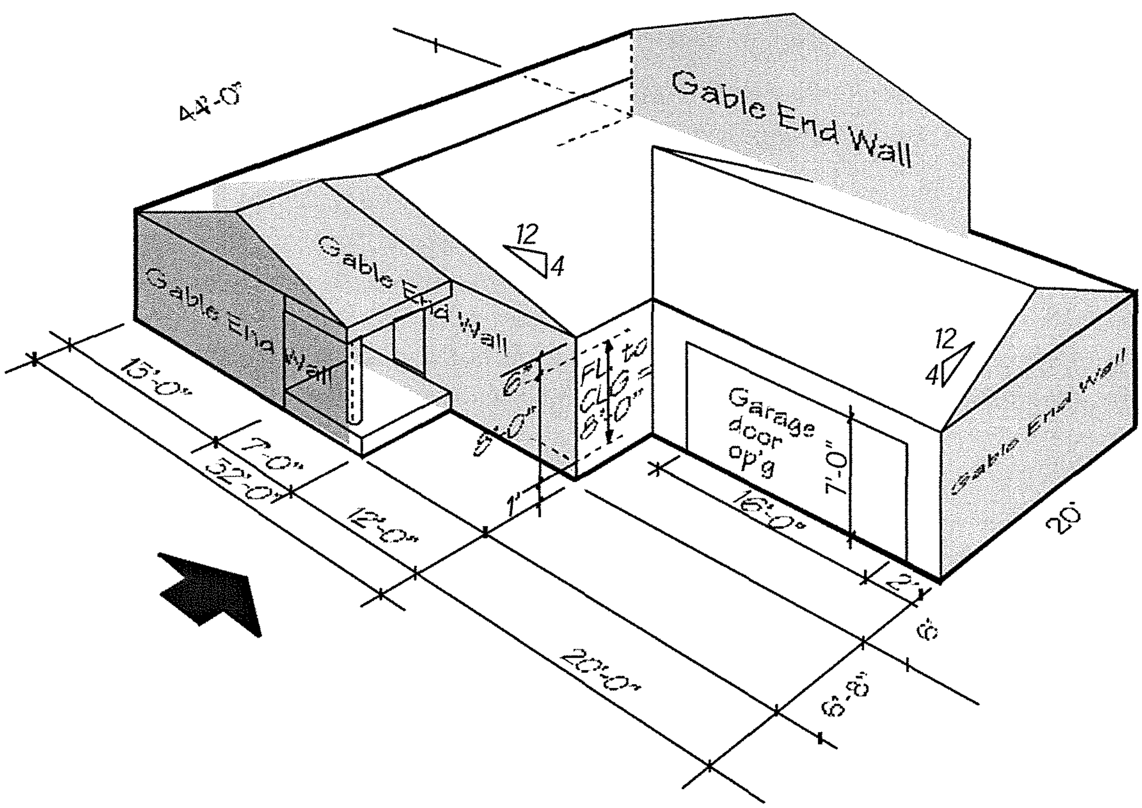


Figure 4.10 - Perspective of one story residence
Gable End Walls Highlighted

Consideration must be given to the weight of the triangular wall portion within the gable. Enter Table A.16 of Appendix A with the vertical component of the roof slope (**h**) in inches, and the roof truss span (**L**) in feet. Read ($\mathbf{H_{avg}}$) at the intersection of the respective column and row. Thus for the house portion **h** = 4" and **L** = 32'; therefore $\mathbf{H_{avg}}$ = 2.7 feet. The wall weight for the triangular gable wall is found in Table A.12 of Appendix A. Under the heading "gable end" for 2x4 studs at 16" o.c., read $\mathbf{Wall_{DL}}$ = 1.7 psf of vertical surface height. The gable end wall weight equation is: $\mathbf{G_{DL}} = \mathbf{H_{avg}} \times \mathbf{Wall_{DL}}$, thus for the house portion $\mathbf{G_{DL}}$ = 2.7 x 1.7 = 4.6 plf.

The garage gable end east wall has h = 4" and L = 20', thus from Table A.16 $\mathbf{H_{avg}}$ = 1.7 feet. The gable end wall dead load from Table A.12 is still **1.7** psf of vertical wall surface. The garage gable end wall uplift resistance is $\mathbf{G_{DL}}$ = 1.7 x 1.7 = 2.9 plf.

House: Gable End Triangles @ North and South Elevations: $\mathbf{G_{DL}}$ = 4.6 plf

Garage: Gable End Triangle @ East Elevation: $\mathbf{G_{DL}}$ = 2.9 plf

10. Determine the $\mathbf{R_{upnet}}$ at the foundation for the gable end wall locations. The house portion does not have its floor framing bearing on the north/south gable end wall foundation. Equation 10(b) on page 144 is the correct one to use for the house. Given the information collected in step 9: $\mathbf{R_{upnet}} = \mathbf{R_{up}} - \mathbf{Wall_{DL}} - \mathbf{G_{DL}}$. Thus, $\mathbf{R_{upnet}}$ = 386 – 25.6 – 4.6 = 355.8 plf along the foundation wall.

    The garage portion has a floating slab-on-grade and contributes no weight to the uplift resistance at the gable end wall foundation. Equation 10(b) on page 144 is also the correct one to use for the garage. Given the information collected in step 9: $\mathbf{R_{upnet}} = \mathbf{R_{up}} - \mathbf{Wall_{DL}} - \mathbf{G_{DL}}$. Thus, $\mathbf{R_{upnet}}$ = 280 – 25.6 – 2.9 = 251.5 plf.

    House: Net Uplift at North and South Foundation Walls: $\mathbf{R_{upnet}}$ = 355.8 plf

    Garage: Net Uplift at East Foundation Wall: $\mathbf{R_{upnet}}$ = 251.5 plf

11. The house sits on a crawl space. Start with the east/west foundation walls, where the $\mathbf{R_{upnet}}$ = 280.5 plf. Enter Table A.13 of Appendix A for crawl space conditions and assume that the contractor prefers to install concrete block foundations. Read

282 pounds per foot of length for a block depth of 3'-4" and grout spacing of 48" o.c. Also, an 8" x 16" concrete footing is assumed, along with an 8" projection of the foundation wall above grade. Verify if the depth of the foundation extends past the minimum depth for frost recommended for your geographic region. The total depth below grade is 3'-4" + 8" ftg – 8"above grade = 3'-4" or 40 inches. There are many other options if a greater or lesser depth for frost uplift protection is required. Assume 24" or 2'-0" is required for frost depth. Read from the Table at the "**C**" = 2'-0" and move horizontally until a value great than a $\mathbf{R_{upnet}}$ occurs. For this case there is no resistance value greater than 280.5 pounds per foot of length. A deeper foundation is required beyond that needed for frost protection. This takes us back to the original design. The Champaign/Urbana geographic area requires 3'-4". Initially, assume

House: **East and West Foundations:**
8" CMU grouted at 48 inches o.c. to a depth of 3'-4"

The north/south walls of the residence will use the same foundation depth as designed for the east/west walls for construction practicality. Verify that the foundation resistance will still be adequate. $\mathbf{R_{upnet}}$ = 355.8 plf along the north/south walls. Read from the Table where **C** = 3'-4" until a larger value than the net uplift force of 355.8 appears. Since none appears, drop down to a greater **C** depth. The value is 367 pounds per foot of building length, using 8" CMU grouted at 48" o.c. Thus, the north/south walls, where no floor dead load is available to reduce the uplift force, results in a greater required uplift resistance for the foundation design.

House: **North and South Foundations:**
8" CMU grouted at 48 inches o.c. to a depth of 4'-8"

The first logical simplification is to increase the depth of the foundation walls for the east/west condition to match the north/south-controlling situation. Thus the entire foundation of the residence would extend to a depth of 48" below grade. Note that one #4V reinforcing bar must extend to the footing at each grouted core and have a footing dowel in alignment with the vertical bar.

The north foundation wall of the garage has a net wind uplift $\mathbf{R_{upnet}}$ = 254.4 plf.

The garage gable end east foundation wall has a net wind uplift $\mathbf{R_{upnet}}$ = 251.5 plf.

The garage south foundation wall has a net wind uplift $\mathbf{R_{upnet}}$ = 280.0 plf due to the garage door opening. Hold the south garage door elevation for now. The largest $\mathbf{R_{upnet}}$ = 254.4 plf for three of the foundation walls. Read from Table A.13 for **"C"**

= 3'-4" the value of 281.9 plf, which is greater than the required net uplift force. This solution uses 8" CMU grouted at 48" o.c. This meets frost depth requirements and the garage foundation can be shallower than the foundation for the residence.

Garage: All Foundations:
8" CMU grouted at 48 inches o.c. to a depth of 3'-4"

Figure 4.11a illustrates the summary of net uplift forces at the foundation, vertical reinforcement in the 8" CMU walls and the footing depth.

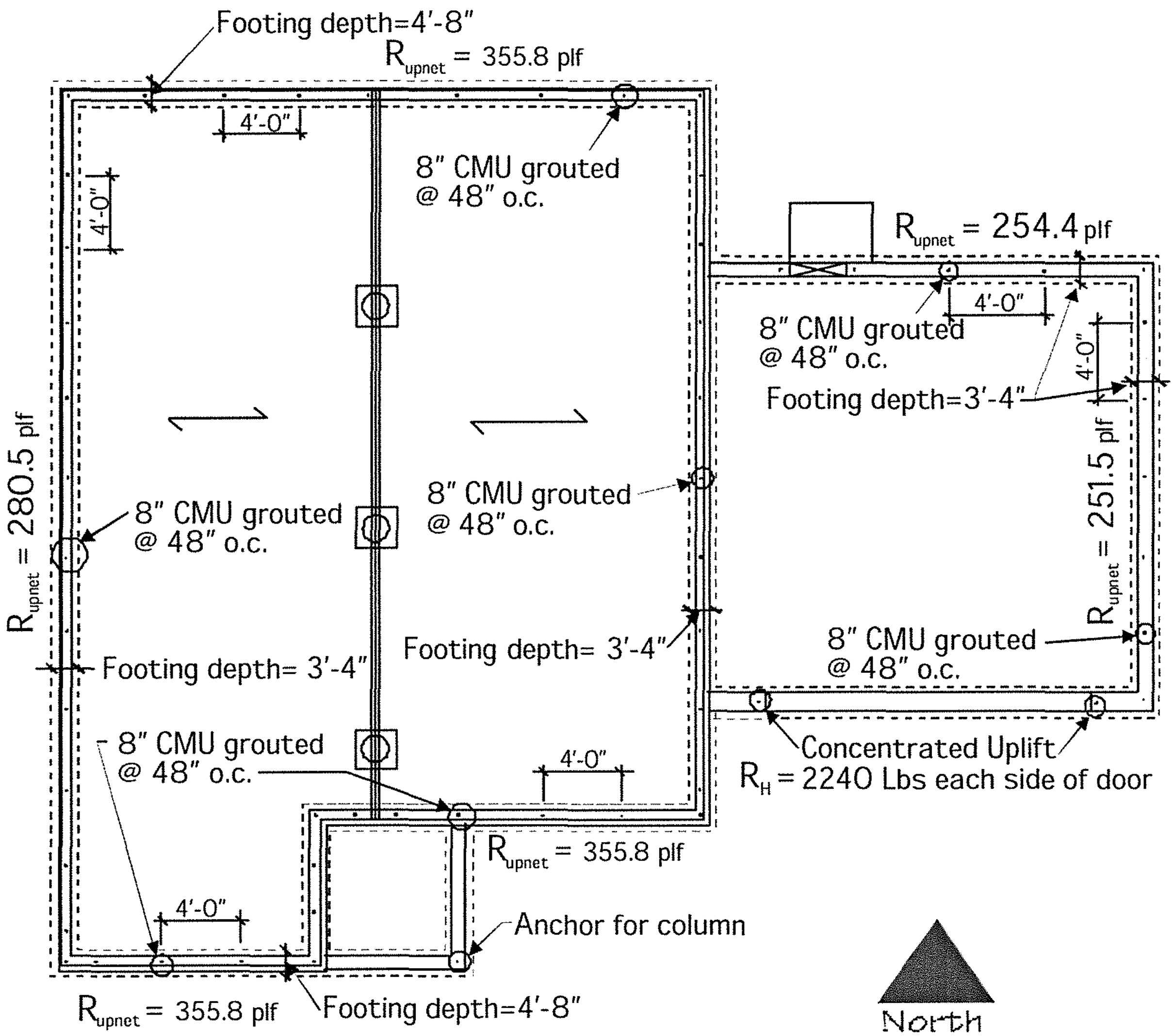


Figure 4.11a - Summary of Net Uplift Forces at Foundation, Vertical Reinf. & Footing Depth

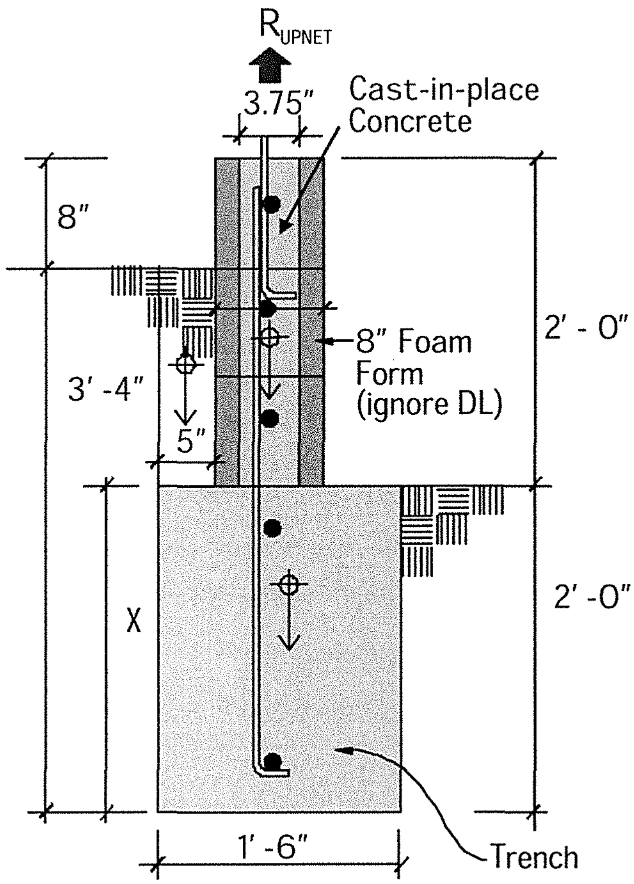
ALTERNATE FOUNDATION DESIGN

Next assume that the contractor has a backhoe with a shovel width of 18 inches. He suggests for economy that he will dig trench-footings, since concrete is the least costly way to achieve uplift resistance. See Figure 4.8 for a section illustration of the trench-footing option. To achieve the 3'-4" depth for frost protection, the footing depth must be 2'-0" with 3 courses of concrete filled foam forms. Calculations to arrive at the uplift resistance for a trench footing approach are shown in Figure 4.12.

A logical first choice for a footing depth is 2'-0" so as to achieve a 3'-4" frost depth. The calculations illustrate an uplift resistance of 362.5 plf. The west foundation wall $\mathbf{R_{upnet}}$ = 280.5 plf. Thus, the desired condition is that: $\mathbf{FDTN_{DL}} \geq \mathbf{R_{upnet}}$ or in essence 362.5 plf ≥ 280.5 plf. The largest $\mathbf{R_{upnet}}$ = 355.8 plf found along the north and south walls of the house. Here again, 362.5 plf ≥ 355.8 plf, so the trench-footing depth still works. It is possible to keep changing trench depth around the house and garage as the $\mathbf{R_{upnet}}$ value changes; however, this is not practical nor desirable as it also changes anchor bolt spacing around the four elevations of the house and garage. Plus, the frost depth of 3'-4" is required anyway. Figure 4.9 illustrates the actual required trench footing depth and anchor bolt spacing to resist the net uplift. Obviously, errors will occur in coordination and layout. It is generally best to select one footing size and one anchor bolt spacing. Use Table A.17 in Appendix A to select trench-footing depth to produce: $\mathbf{FDTN_{DL}} \geq \mathbf{R_{upnet}}$

House and Garage: Trench Footing Depth = 2'-0" to produce a 3'-4" frost depth from the grade to the bottom of the footing.

Figure 4.11b illustrates the resulting foundation uplift forces, anchor bolt forces and minimum size of trench-footings, while Figure 4.11c illustrates the final practical results for design.



$$\text{Wall: } \left(\frac{3.75}{12}\right)\left(150^{PCF}\right)\left(2^{FT}\right) = 93.75^{PLF}$$

$$\text{ftg: } \quad 150^{PCF}\left(2^{FT}\right)\left(1.5^{FT}\right) = 450^{PLF}$$

$$\text{Total DL: } \quad = 543.75^{PLF}$$

Reduced DL = 2/3 (543.75) $= 362.5^{PLF}$
(For Code Uplift)

Ignore 5" soil wedge above ftg ledge

Another approach to determine trench footing depth if frost depth quite minimal:

since $R_{UPNET} = 280.5^{PLF}$

Assume an A. B. Spacing, say 3' - 4" o.c.

ॐ $R_{UPNET_{3\text{-}4}} = 934^{LB}$/3' - 4" a.b. Spacing:

wall : 93.75 (3.33') $= 312.2^{LB}$

ftg : 1.5(150)(3.33')x $= 749.25x^{LB}$

set : $R_{UPNET} = 2/3\ (DL)$

$$934^{LB} = (312.2 + 749.25\,x)\ 2/3$$

$$x = \frac{3/2\ (934) - 312.2}{749.25}$$

$$\boxed{x = 1.5^{FT}}$$

Req'd footing depth

Figure 4.12 - Trench Footing Weight

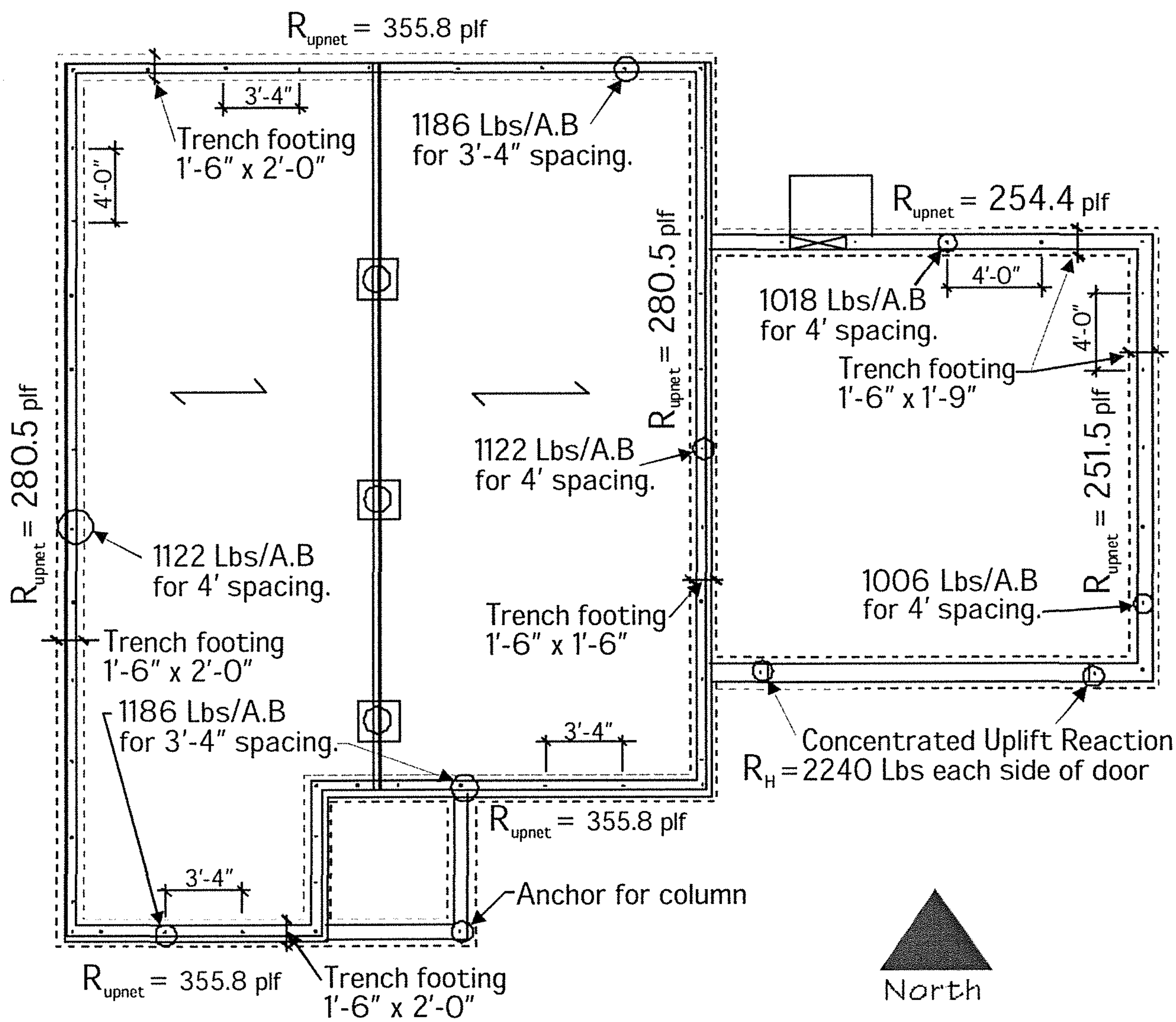


Figure 4.11b - Summary of Net Uplift Forces at Foundation, A. B. Spacing & Minimum Trench Footing Sizes

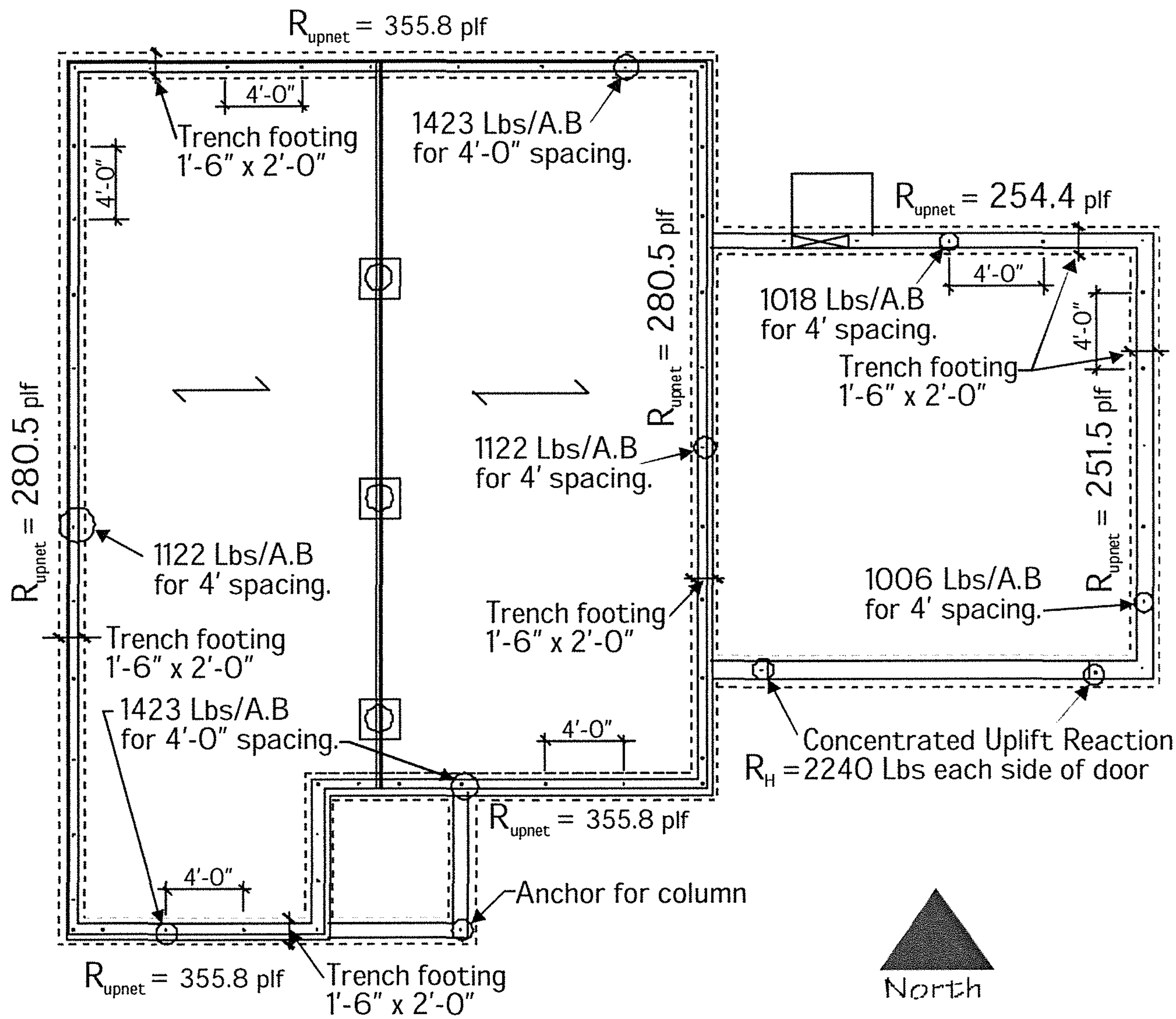


Figure 4.11c - Summary of Final Net Uplift Forces at Foundation, A. B. Spacing & Trench Footing Sizes

12. The selection of anchor bolt spacing and washer type for the residence should be based on the largest $\mathbf{R_{upnet}}$ = 355.8 plf for the north/south foundation locations. If we place the anchor bolts at 4'-0" on center to match the grouted core spacing, the anchor bolts should start at the same spacing. Thus each anchor bolt must resist 355.8 plf x 4 ft. = 1423.2 pounds. Figure 4.11c illustrates that spacing and force uplift on the anchor bolt. Enter Table A.15 of Appendix A from the right column heading "spacing" and move down to 4'-0". Read across horizontally to the first three columns. Select the washer that provides a capacity greater than the required force of 1423.2 pounds. Note that none of the washers provide the required capacity. Move to the next smaller spacing of 3'-4" and revise the required uplift force to 355.8 plf x 3.333 ft. = 1186.0 pounds of net uplift at each anchor bolt. Re-enter the Table at 3'-4" spacing and select the square washer with an uplift resistance capacity of 1483 pounds.

House: North/south Foundation: Revise 1/2" ϕ A.B. spacing from 4'-0" to 3'-4"

The largest net uplift force at the garage is along the north elevation. The value of $\mathbf{R_{upnet}}$ = 254.4 plf. Based on 4'-0" spacing of anchor bolts, the uplift force at each anchor bolt will be 254.4 plf x 4 ft. = 1018 pounds. Figure 4.11c illustrates that spacing and force uplift on the anchor bolt. Enter Table A.15 again with a 4'-0" anchor bolt spacing and read across to a value that is larger than 1018 pounds. Select the square washer with an uplift resistance of 1236 pounds.

Garage: All Foundations: Use 1/2" ϕ A.B. spacing @ 4'-0"

13. The garage door opening is H_L = 16'-0" long. The wind uplift force at the south elevation roof R_{up} = **280** pounds per foot of length. Thus, using these two values and the equation: $\mathbf{R_H = R_{up} \times H_L \div 2}$, the uplift reaction is $\mathbf{R_H}$ = 280 plf x 16 ft. ÷ 2 = 2240 pounds. Ignore the weight of the header over the door to be conservative, since this is a critical wind tie-down location adjacent to a large door. The connection between the wood header and the doubled wood studs must also assist in the tension chain of resistance. Figure 2.19 illustrates the top, middle and bottom conditions requiring connectors or structural paneling with appropriate nailing. Metal connectors will be selected from Simpson Strong-Tie Company's Catalog. Reference Figure 4.13 for garage details.

The connection of each truss to the double top plate and then to the header should be done with one type of connector. From step 3 in the solution, the uplift force at each truss end is 560 lbs. Figure 2.5 illustrates a twisted strap anchor type (U), which would be appropriate for this situation. Select a LTS20 with (6)-10d common nails attached to the truss and to the header will provide an allowable [4.5] uplift resistance of 720 lbs., which is greater than the required uplift resistance of 560 lbs.

It is recommended that structural paneling cover this detail to the top of the double top plate.

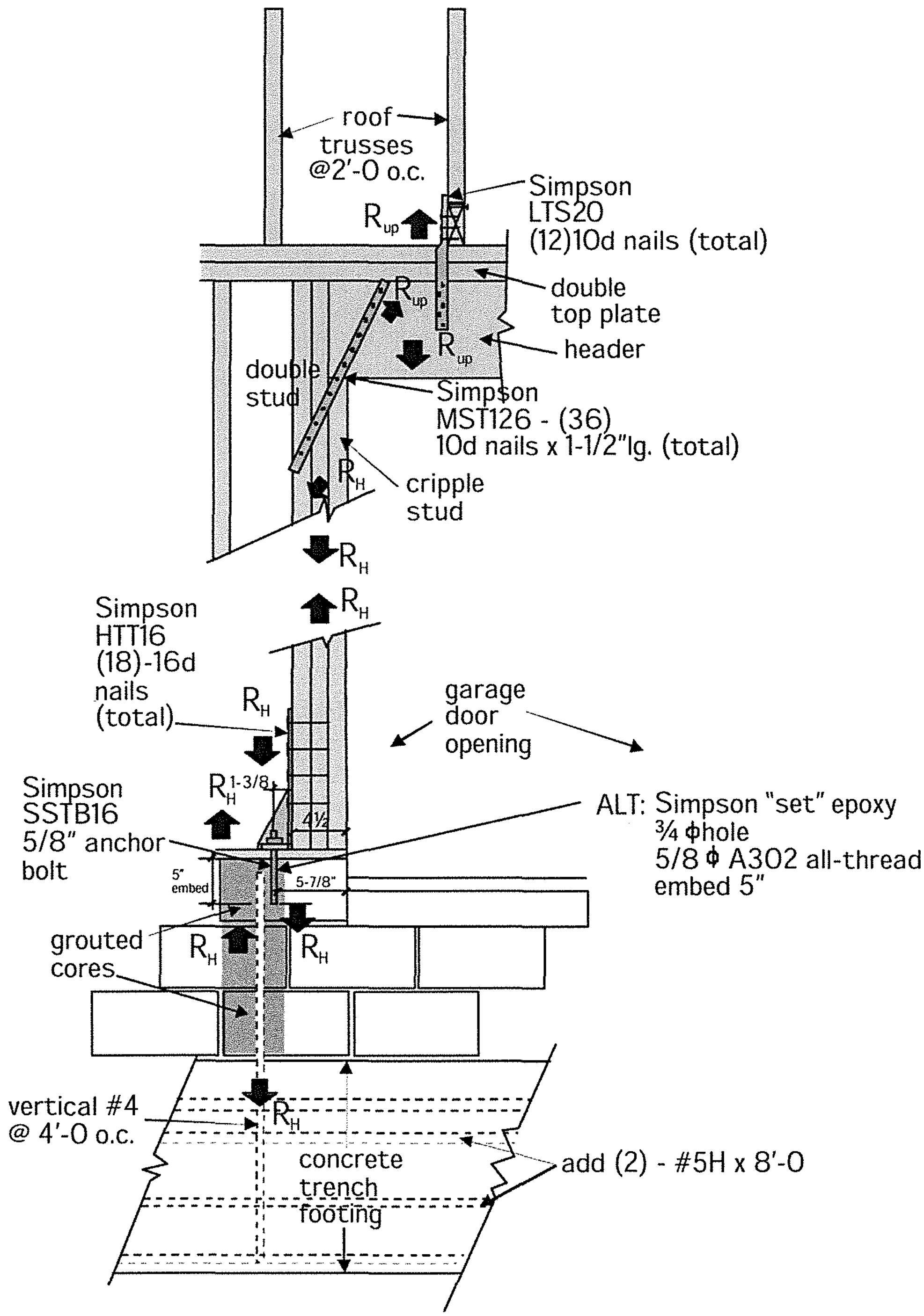
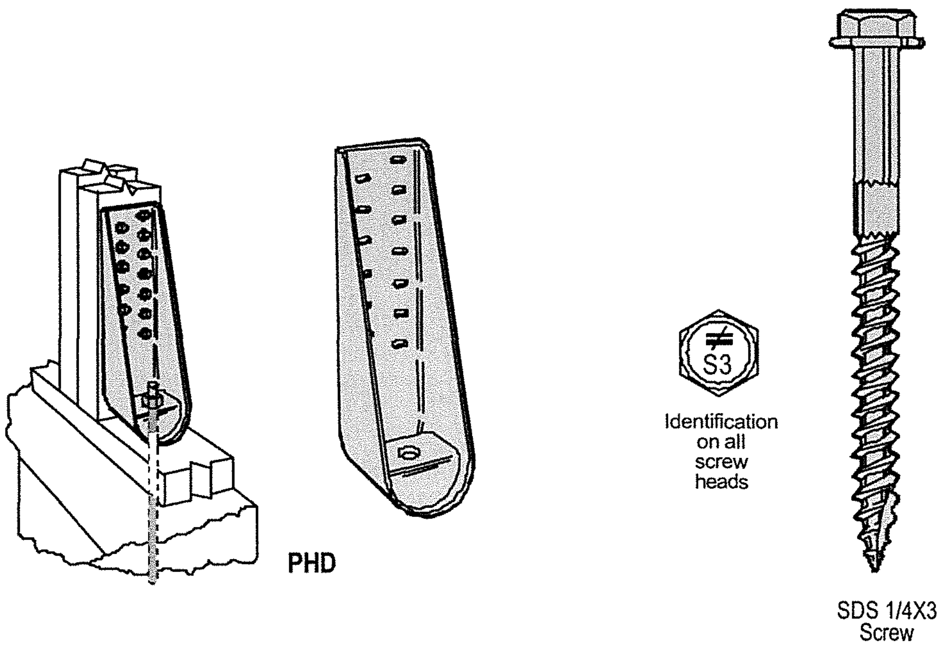


Figure 4.13 - Garage Tension - Chain @ Door

A metal strap tie on a slight diagonal from the end of the header to the double stud will suffice to connect the header to the doubled stud. Select a MST126 [4.6] metal strap tie with (18)-10d x 1-1/2" long nails to the header and the doubled stud. Be sure that nine of the nails hit each stud. The allowable uplift capacity is 2505 lbs., which is greater than the 2240 lbs. of required resistance. It is recommended that structural paneling cover this detail.

There are two types of tie-down anchors to the foundation: (1) using thru-bolts as illustrated in Figure 14 of Chapter 14 in the *WMM*; or (2) using nails as illustrated in Figure 2.7b(z), or another type using screws as shown below in Figure 4.14 [4.7].



Model No.	Ga	Dimensions W	H	B	CL	Fasteners Anchor Diameter	No. of Simpson SDS1/4 X 3 Wood Screws	Average Ult	Allowable Loads 2-2x and Greater Vertical Wood Member	Holdown Deflection at highest Allowable Design Load
PHD2	14	3	9-5/16	2-7/8	1-3/8	5/8	10	12,520	3610	.003
PHD5	14	3	11-9/16	2-7/8	1-3/8	5/8	14	15,670	4685	.047
PHD6	12	3-1/8	13-13/16	2-7/8	1-3/8	7/8	18	18,250	5860	.045
PHD8	10	3-1/8	17-3/16	2-7/8	1-3/8	7/8	24	21,243	6730	.051

1. Allowable loads have been increased 33% for earthquake or wind loading with no further increase allowed; reduce where other loads govern.
2. The anchor embedment and configuration must be specified. See the SSTB Anchor Bolts.

Model No.	Description	Metric Equivalent (mm)	Finish	Fasteners per carton	Doug Fir-Larch/So. Pine Allowable Loads[1] Light Guage Shear (100)	Light Guage Guage	3 Guage Shear (100)	Spruce-Pine-Fir Allowable Loads[1] Light Guage Shear (100)	Light Guage Guage	3 Guage Shear (100)
SDS 1/4X3	¼" x 3" Wood Screw	6.1 x 76.2	ZINC	950	303	10	327	261	10	284

1. Allowable loads are based on the 1997 NDS. Adjustments are made for use with metal side plates, Fes = 45 ksi. Loads under light gauge are for gauges listed through 22 gauge. Allowable loads for gauges not indicated must be calculated according to the code.

Figure 4.14 - Tie-Down Anchor Type Without Bolts to the Studs

Contractors state that bolt nuts and extended threads are in the way of the finish wood for the trim of the opening. Thus, screwed or nailed connectors are preferred at the ends of garage door openings. The "Z" type of Figure 4.7b will be used. Select the tension tie HTT16 [4.8] with (18) - 16d nails into the double stud, which has an allowable tension capacity of 3480 lbs. A 5/8"ϕ anchor bolt type SSTB16 [4.9] embedded 12" into the concrete of the insulating foam formwork, which has an allowable tension capacity of 4085 lbs. Both capacities are greater than the 2240 lbs. of required uplift resistance.

An alternate solution to the pre-set anchor bolt is desirable, since alignment of the anchor bolt in wet concrete has to accurately match that of the tension tie hole location. Drilled holes, after the stud wall is set and the concrete has hardened, are a simpler solution. Select the "SET" High Strength Epoxy [4.10] method, using a 5/8"ϕ threaded A307 Rod. A 3/4"ϕ drill is required for the selected rod diameter. A 5" embedment into the concrete is required for an allowable uplift capacity of 3140 lbs.

The dead weight of the foundation must provide the last resistance to the tension chain. The trench footing plus wall weight = 362.5 plf as calculated above in Figure 4.12. The required concentrated uplift resistance =2240 lbs. each side of the garage door opening. The length of foundation to be engaged to resist that concentrated uplift: 2240 lbs. ÷ 362.5 plf = 6.2 ft. Given that the trench footing is reinforced horizontally and that vertical bars exist at 4'-0" o.c., it is likely that over six feet of foundation could be engaged. Review Figure 4.13 again.

Trench Footing: **Add (2) - #5 bars x 8'-0" long horizontally and centered under the concentrated uplift load.**

There are no large windows that would require the anchorage system just described for the garage door location.

14. The entry porch has a protective roof. See Figure 4.4 at the beginning of Example 1. Two baked aluminum half cylinders join to create a hollow decorative column. The column is loadbearing and can handle the gravity load. A 4 x 4 wood column could fit inside and be used as a tension tie to keep the porch roof from being broken by wind uplift. Metal anchors at the top and bottom are an easy solution.

An alternate solution is to use a metal chain connected to an eyebolt embedded into the concrete of the porch and another eyebolt drilled through the porch beam to a square washer and nut above. This was the preferred approach of the contractor who built this home. See Figure 4.15.

The calculations involve the 20-foot span trusses at the front extension of the house. These trusses are also at a 4 in 12 slope. Select Table A.3 in Appendix A

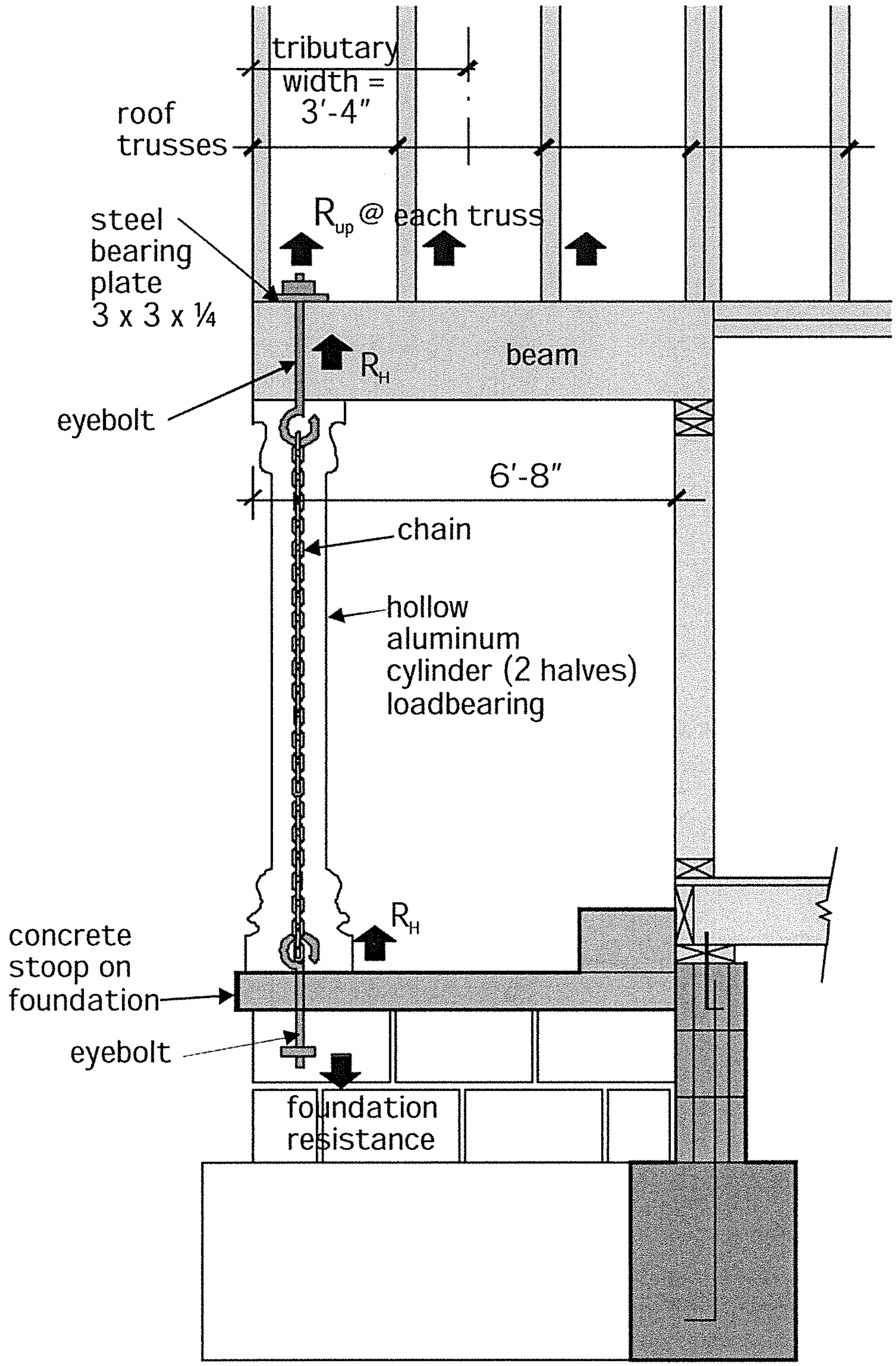


Figure 4.15 - Entry Column - Tension Chain

and read $\mathbf{R_{up}}$ = 280 lbs./ft of house length. The overhang is 6'-8", so the tributary width of wind at the column is $\mathbf{R_{up}}$ **x** 6'-8"/2 = 933.0 lbs. Consideration should be given to wind under the overhang pushing upward. Assume 20 psf x 0.8 = 16.0 psf. The tributary width of the ceiling of the overhang is 7'-0" ÷2 = 3.5 ft. The additional wind load to be carried by the column is 16 psf x 3.5' x6'-8"/2 =93 lbs. The total uplift force is 933 + 93 = 1026 lbs.

The anchors described for the chain must provide sufficient uplift resistance. Manufacturer's catalogs will assist in the choice of these items.

Example 2 - All Metal Connector Solution

Given:

Repeat Example 1, except that this time it is desired to use non-structural insulation sheathing wherever possible. Structural Panels will be used only for stability shear walls. Figures 4.4 through 4.8 still apply. Note that studs do not align with the roof trusses, except by chance. Many of the steps in the procedure will repeat. Only those steps that are new will be described. Simpson Strong-Tie's Catalog [4.1] will be used to select metal connectors and determine their allowable capacities.

Solution:

Follow the procedure outlined in the previous section:

2. Same as Example 1:

Use 90-MPH wind speed

2. Geometric parameters are same as Example 1:

House: **Roof slope is 4 in 12**
Roof Truss Span = 32 feet.

Garage: **roof slope of 4 in 12.**
Roof Truss Span = 20 feet.

3. Same as Example 1:

House: East and West walls: R_{up} = 386 plf
Uplift at each roof truss end = 772 Lbs. @ 2'-0"o.c.

Garage: North and South walls: R_{up} = 280 plf
Uplift at each roof truss end = 560 Lbs. @ 2'-0"o.c.

5. Same as Example 1:

House: East and West wall connection to *every* Truss:
Simpson: (2) H2.5: Capacity = 930 Lbs./Truss

Garage: North and South wall connection to *every* Truss:
Simpson: H1: Capacity = 585 Lbs./Truss

5. The solution in progress assumes that non-structural insulation board will be used as sheathing from the top of the double top plate to the bottom of the mudsill. Structural panels will only be used at stability shear wall locations.
Figure 2.6b illustrates several types of metal connectors for use between the double top plate and the studs. Connector type "B" is a common choice and can be potentially used both at the top double plate of the stud wall and at the bottom plate. The Simpson SP2 connector [4.11] is used at the double top plate and has an allowable tension capacity of 1065 lbs. The house east and west walls have $\mathbf{R_{up}}$ = 386 plf. The spacing of the metal connector (**S**) by equation will be:

$$\mathbf{S} \text{ (in inches)} = \text{allow. Tension capacity of one connector} \div \mathbf{R_{up}} \times 12$$

Thus, S = 1065 ÷ 386 x 12 = 33.1 inches. Since the studs are at 16" o.c., use the connector at every other stud (32 inches).

House: East and West Elevations: Stud to Double Top Plate:
Use SP2 metal connector @ 32" o.c.

The north and south elevations of the garage have $\mathbf{R_{up}}$ = 280 pounds per foot of length and an uplift of 560 lbs. per truss at 2'-0" o.c. Even though this is smaller than the uplift per truss on the house, use the same connector SP2 to use a minimum of connector types.

Garage: North and South Elevations: Stud to Double Top Plate:
Use SP2 metal connector @ 32" o.c.

Note: The truss layout and spacing is random with respect to the stud layout and spacing. Therefore, the uplift force at the metal connector, between truss and double top plate, does not align with the stud to double top plate connector. The distance between the two connectors (referred to as eccentricity "**e**") produces bending and shears in the double top plate as shown in Figure 4.16(a).

The double top plate will be subjected to bending from the uplift at each truss connection. The double top plate must behave as a continuous beam with the double top plate/stud metal connector locations being the supports. The maximum bending and shear locations in the double top plate are shown in Figure 4.16(b). When the truss uplift is adjacent to a stud/double top plate connection a maximum shear condition occurs in the double top plate. When the truss uplift is centered between two stud/double top plate connections and no metal connector exists to the stud directly below the truss uplift, a maximum bending condition occurs. A second location of maximum bending occurs near a building corner where the double top plate ceases to be continuous.

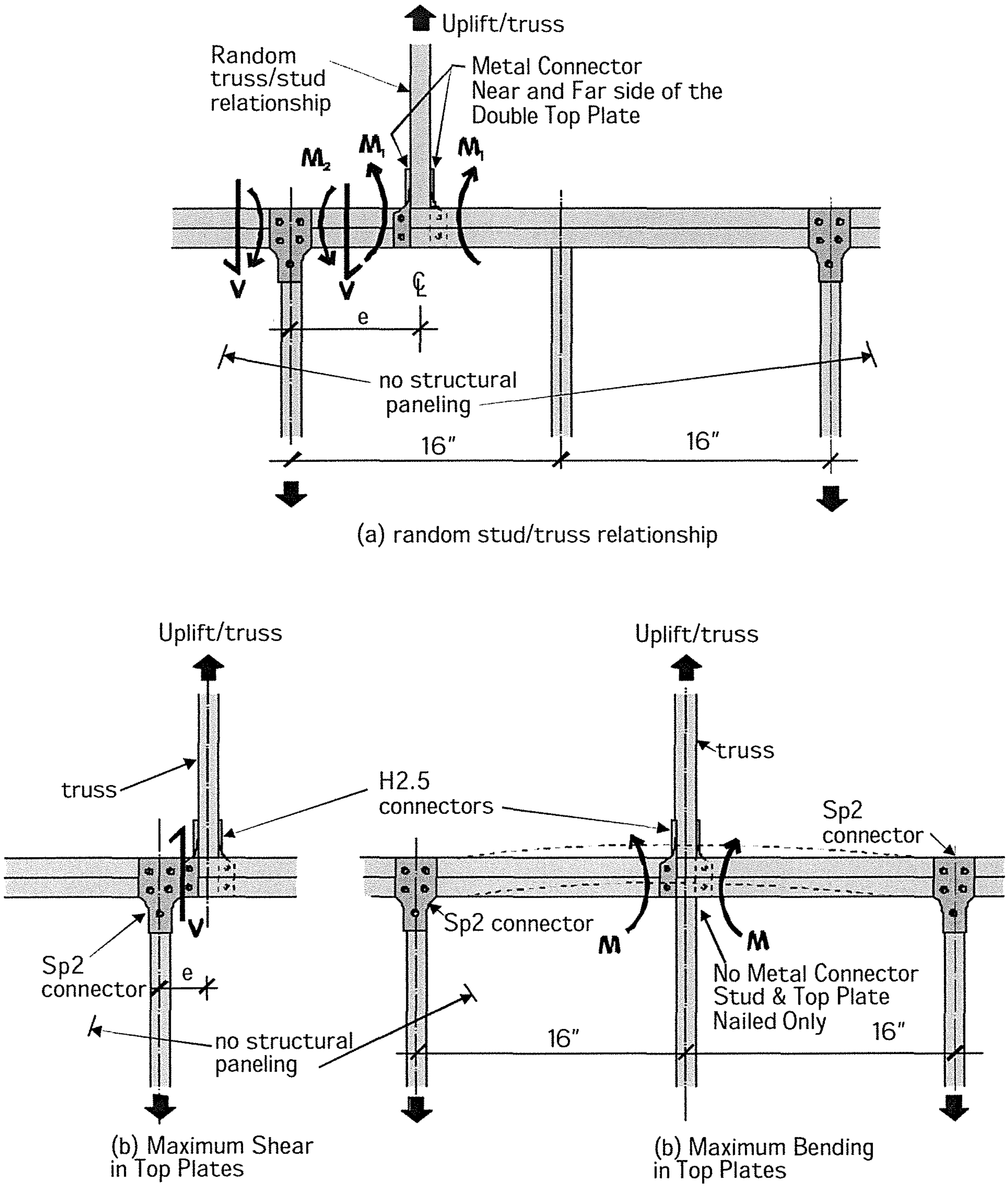


Figure 4.16 - Stud/Truss Relationships

The double top plate has its own Code requirements, which are illustrated in Figure 2 of Chapter 6 in the *WMM*:

a. Butt splices shall occur over a stud, never between studs;
b. a 4'-0" minimum stagger shall occur between butt splices in the top and bottom plates. These butt splices must also align with a stud below.

Figure 4.17 illustrates this author's recommendations for the relationship between truss and stud connectors to the double top plate, based on the above (a) and (b) Code criteria. The assumptions used for these recommendations are:

(1) That S-P-F, Douglas Fir-Larch or Southern pine 2 x 4-dimension lumber is used;
(2) that the lumber is at least "Construction Grade"; and
(3) that the wind speed does not exceed 90 MPH.

The recommendations are as follows:

1. Where the double top plate is continuous, metal connections between stud and double top plate can be spaced 32" o.c. maximum. This means that if the studs are spaced on 16" o.c. the metal connectors can be spaced at 32" o.c. maximum.

   Note: If the wind speed is greater than 90 MPH and/or a lesser species and grade of lumber is used, connectors between studs and double top plate shall occur at *every* stud.

2. When a butt splice in the double top chord occurs directly over a stud, that stud and the stud on either side of the butt splice shall have a metal connector between stud and double top plate. That means three metal-plate connected studs in a row with the middle connector at a butt splice in the double top plate.

3. House corner stud walls should have the outer two studs metal-plate connected prior to spacing metal-plate connectors at every other stud.

6. Step #6 is not applicable to the all metal connector solution.

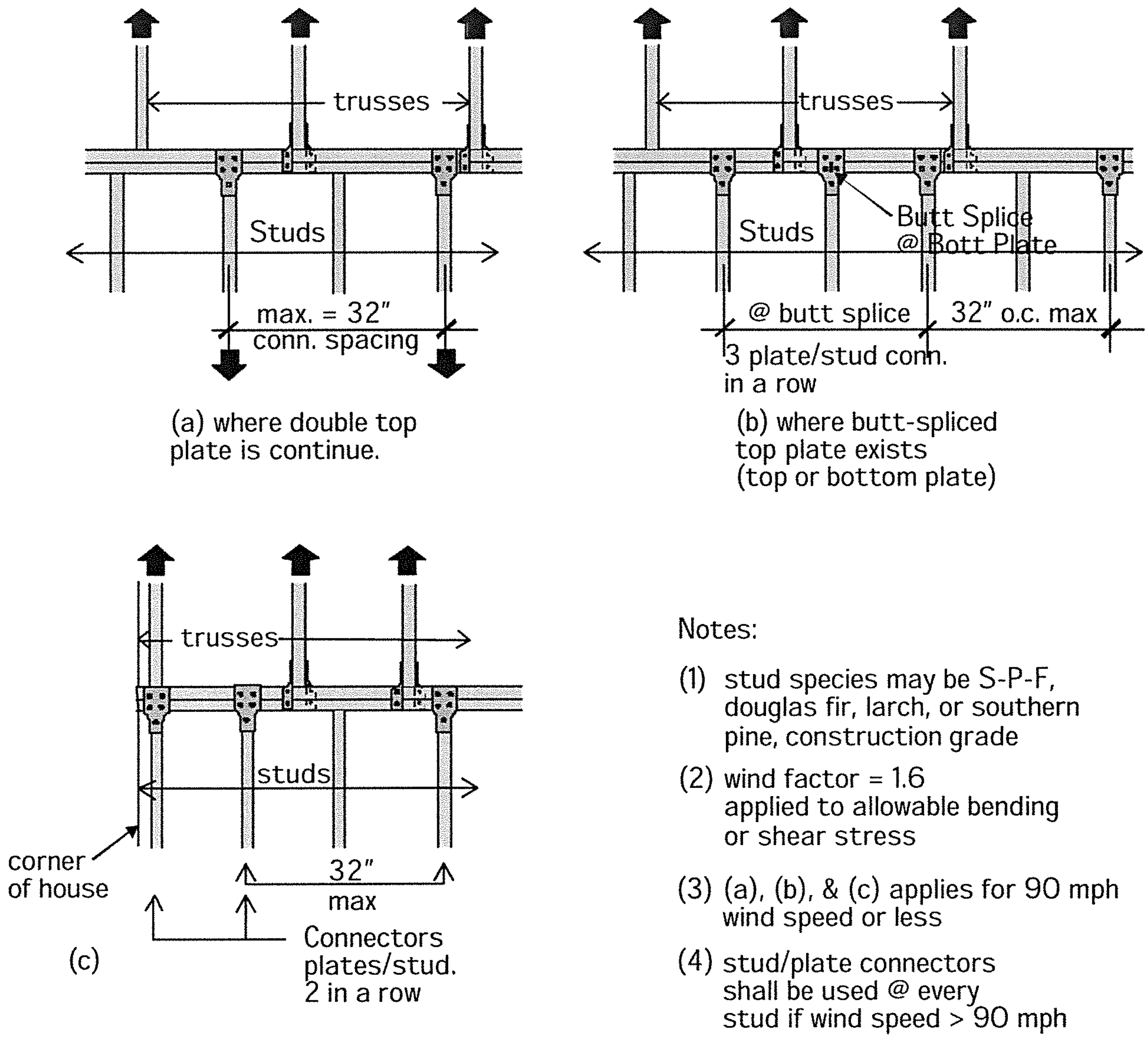


Figure 4.17 - Acceptable Metal Connector Arrangements Between Stud & Double Top Plate

7. Enter Table A.12 in Appendix A and select the gable end weight with structural panel only on one side. This is very close to the weight of interior 1/2" drywall and assumes zero weight for the 1" insulation board. Multiply the 1.7 psf times 8' to obtain the weight of wall per foot of building length, thus 1.7 psf x 8 feet = 13.6 lbs. per foot of length:

House and Garage: Exterior stud wall weight: $Wall_{DL}$ = 13.6 plf

The house uplift at the bottom of the stud wall is $\mathbf{R_{up}}$ = 386 plf - $\mathbf{Wall_{DL}}$ per foot of wall length. $\mathbf{R_{up}}$ = 386 plf - 13.6 = 372.4 plf. Then the net uplift force at the bottom of each stud = 372.4 x 16" o.c./12 in/ft = 496.5 lbs. per stud. The garage uplift at the bottom of the stud wall is $\mathbf{R_{up}}$ = 280 plf - $\mathbf{Wall_{DL}}$ per foot of wall length. $\mathbf{R_{up}}$ = 280 plf - 13.6 = 266.4 plf. Then the net uplift force at the bottom of each stud = 266.4 x 16" o.c./12 in/ft = 355.2 lbs. per stud.

House: **East and West walls:** $\mathbf{R_{up}}$ **= 372.4 plf**
Net Uplift at bottom of each stud = 496.5 Lbs.

Garage: **North and South walls:** $\mathbf{R_{up}}$ **= 280 plf**
Net Uplift at bottom of each stud = 355.2 Lbs.

The house stud walls requires reference to Figure 2.7a for examples of connectors applied to locations where a bottom plate, floor band board and mudsill must all be considered as part of the tension chain. Connector types **Q** and **E** seem most appropriate, as they engage the bottom plate of the stud wall and the bandboard, without needing to be placed at precise locations in the foundation construction.

Simpson's **H6** connector (Manual's E type) provides an allowable uplift capacity of 950 lbs. [4.3]. This connector bypasses the bottom plate and ties the stud to the bandboard. The spacing of the metal connector ($\mathbf{S}$) by equation will be:

$\mathbf{S}$ (in inches) = allow. Tension capacity of one connector ÷ $\mathbf{R_{upnet}}$ x 12

Thus, $\mathbf{S}$ = 950 ÷ 372.4 x 12 = 30.6 inches. Since the studs are at 16" o.c., this connector must be used at every stud.

Simpson's **LSTA18** strap connector (Manual's Q type) with (14) 10d nails total provides an allowable uplift capacity of 1275 lbs. [4.6]. This connector nails to the bottom plate, the stud and to the bandboard. The spacing of the metal connector ($\mathbf{S}$) by equation will be:

Thus, $\mathbf{S}$ = 1275 ÷ 372.4 x 12 = 41 inches. Since the studs are at 16" o.c., this connector can be used at every other stud (32" o.c.).

House: **East and West stud walls:** **use LSTA18 at 32" o.c. with (14) 10d common nails. Align with connectors at top of stud wall.**

The garage only has a bottom plate and the foundation concrete below. Connector type B for only one plate is called **SP1** [4.11] with (6) 10d nails into the stud and (4) 10d nails into the plate. The allowable capacity for this connector is 585 lbs. Thus **S** = 585 ÷ 355.2 x 12 = 19.7 inches, thus this connector must attach to every stud. Another choice is Simpson's **LTT19** [4.8]; similar to the Manual's type **Z** illustrated in Figure 2.7b. The allowable uplift capacity is 1350 lbs., but requires drilling and epoxy bolts which is more work and expense for the contractor.

Garage: **North and South stud walls: use SP1 at every stud to mudsill with (6) 10d nails into the stud and (4) 10d nails into the plate (mudsill in this case)**

The completion of wood part of the tension chain involves connection between the bandboard and the mudsill of the house. Simpson's **LTP4** with (12) 8d nails total has an allowable capacity of 645 lbs. [4.12].

S (in inches) = allow. Tension capacity of one connector ÷ $\mathbf{R_{upnet}}$ x 12

Thus, **S** = 645 ÷ 372.4 x 12 = 20.8 inches. Since the bandboard and mudsill are continuous, this connector could be used at 20" o.c.

House: **East and West stud walls: use LTP4 with (12) 8d nails at 20" o.c. between bandboard and mudsill**

8. Enter Table A.11 in Appendix A and read the floor dead load $\mathbf{DL_{fl}}$ = 106.6 pounds per foot of wall length. Floor dead load $\mathbf{DL_{fl}}$ reduces the $\mathbf{R_{up}}$ value. The net uplift at the foundation for the *east and west walls of the house* is found from the formula in step 8(c.) of the procedure: $\mathbf{R_{upnet}} = \mathbf{R_{up}} - \mathbf{Wall_{DL}} - \mathbf{3/4 \times DL_{fl}}$. Thus, $\mathbf{R_{upnet}}$ = 386 – 13.6 – (3/4) x 106.6 = 292.5 lbs. per foot of length. Note the reduced wall weight.

House: Net Uplift at East and West Foundation walls: $\mathbf{R_{upnet}}$ **= 292.5 plf**

The garage wall weight for the north wall reduces the uplift force $\mathbf{R_{up}}$ = 280 pounds per foot of length. Use formula (b) from step 8 above: $\mathbf{R_{upnet}} = \mathbf{R_{up}}$ **- Wall$_{DL}$** = 280 – 13.6 = 266.4 plf. This is the net uplift for selection of structural metal connectors.

Garage: Net Uplift at North foundation wall: $\mathbf{R_{upnet}}$ = 266.4 plf

The *south garage wall* is the same as Example 1.

Garage: Net Uplift at South door header: $\mathbf{R_{upnet}}$ = 280.0 plf

9. Gable end wall triangles. Same as Example 1.

House: Gable End Triangles @ North and South Elevations: $\mathbf{G_{DL}}$ = 4.6 plf

Garage: Gable End Triangle @ East Elevation: $\mathbf{G_{DL}}$ = 2.9 plf

9A. Select metal connectors for gable end walls. Figure 4.18 illustrates the connection locations. Start with the gable end wall connections on the north and south elevations of the house. The gable end wall truss is composed of 2x4's laid flat; therefore 4x2's. These are used for the sloping top chords, bottom chords and the vertical web members. There usually are no diagonal web members in gable end trusses. The members of the truss are connected with pressed metal plates. The truss comes prefabricated and does not need any additional connections. The connection between the bottom chord of the gable end truss to the double top plate and to the stud is labeled connection 1 in a circle. The required net uplift force is $\mathbf{R_{upnet}} = \mathbf{R_{up}} - \mathbf{G_{DL}}$ = 386 – 4.6 = 381.4 lbs. per foot of wall length. Simpson's strap tie **ST9** with (8) 16d nails total is a logical choice. The allowable capacity of the connection is 850 lbs. [4.6]. The usual spacing equation is use

$\mathbf{S}$ (in inches) = allow. Tension capacity of one connector ÷ $\mathbf{R_{upnet}}$ x 12

$\mathbf{S}$ = 850 ÷ 381.4 x 12 = 26.7 inches o.c. This spacing will not permit placing a strap tie every other stud, thus use at every stud.

House: North and South Elevations: Gable end truss bottom chord to double top plate and to the stud: ST9 with (8) 16d nails @ every stud. Connection 1

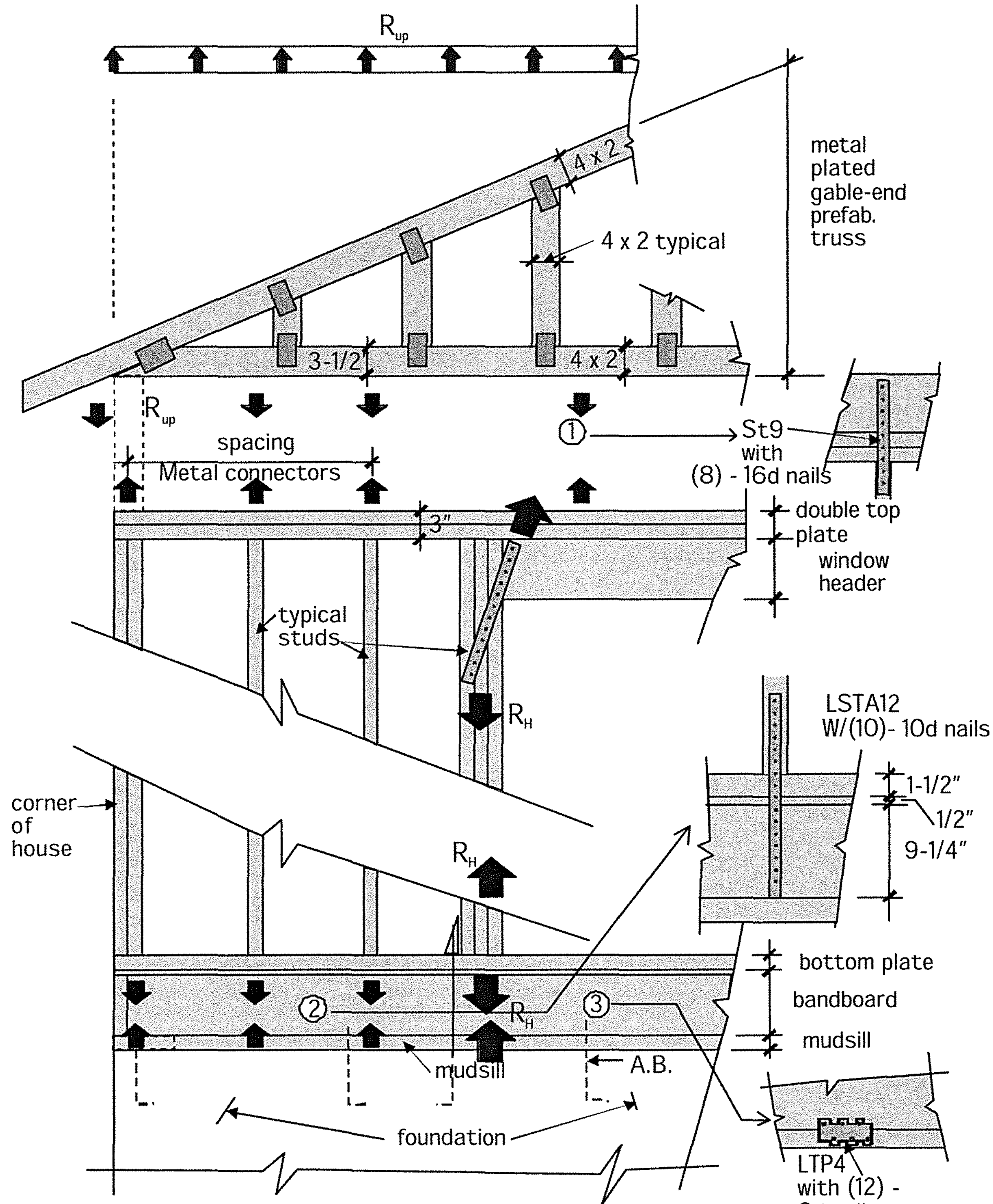


Figure 4.18 - Gable End Stud Wall Elevation
Location of Metal Connectors

The next connection is at the bottom of the studs to the bottom plate and the bandboard. The net uplift force will subtract the wall weight, besides the triangle of the gable end. $\mathbf{R_{upnet}} = \mathbf{R_{up}}$ - **Wall**$_{\mathbf{DL}}$ - $\mathbf{G_{DL}}$. Thus, $\mathbf{R_{upnet}}$ = 386 – 13.6 – 4.6 = 367.8 lbs. per foot of length. Note the reduced wall weight. Another strap tie is also appropriate at this location labeled connection 2 in Figure 4.18. Simpson's **LSTA12** with (10) 10d nails total, half in the stud and half in the bandboard. The allowable capacity of this connector is 905 lbs. [4.6]. Again, using the spacing equation:

S = 905 ÷ 367.8 x 12 = 29.5 inches o.c.

House: North and South Elevations: Stud to bottom plate and bandboard: LSTA12 with (10) 10d nails @ every stud.
Connection 2

The next wood connection is between the bandboard and the mudsill. Since the north and south elevations are non-loadbearing, the same net uplift applies $\mathbf{R_{upnet}}$ = 367.8 plf. The same framing anchor as used on the east and west elevations will apply, since the $\mathbf{R_{upnet}}$ is almost the same.

House: North and South Elevations: use LTP4 with (12) 8d nails at 20" o.c. between bandboard and mudsill

Note: It is assumed that wind perpendicular to the surface of any gable end wall will be braced with a ceiling plane and wood diagonals as shown in Figure 21 of Chapter 7 in the *WMM*.

Any window header should be designed for uplift and anchorage requirements to the foundation as described in step 13 of Example 1. Similar metal connectors will be required.

10. Determine the $\mathbf{R_{upnet}}$ at the foundation for the gable end wall locations. The difference in wall weights is minimal between Example 1 and Example 2, thus the net uplift forces to the foundation are quite similar and need not be repeated. Use the values from Example 1.

House: Net Uplift at North and South Foundation Walls: $\mathbf{R_{upnet}}$ = 367.8 plf

Garage: Net Uplift at East Foundation Wall: $\mathbf{R_{upnet}}$ = 263.5 plf

11. Same as Example 1

12. Same as Example 1

13. Same as Example 1

14. Same as Example 1

Two Story Residences

The Uplift resistance chain for all two-story residences will utilize the selection items found in Figure 4.19. The appropriate Design Tables in Appendix A are referenced in Figure 4.19 to provide a simplified approach to design of the Wind Uplift Resistance Chain.

Notes

The procedure detailed for Single Story Residences is similar in most respects to that for Two Story Residences; however there are a few comments to aid the user:

1. Tables A.7 to A.10 in Appendix A are the appropriate Tables for two story residences. The Tables are easily found by the upper right hand label $\mathbf{R_{up2}}$ to indicate two story Tables. And the span is labeled just below the $\mathbf{R_{up2}}$ as 20, 24, 28, 32 feet.

2. $\mathbf{R_{upnet2}}$ is the symbol for the net uplift after subtraction of the gravity dead load from the exterior wall above and the second floor dead load, *when the floor joists span to the exterior wall* below. See Figure 4.19(a).

3. $\mathbf{R_{upnet1}}$ is the symbol for the net uplift after subtraction of the gravity dead load from the two exterior walls above and the two floor dead loads, *when the floor joists span to the exterior wall at both levels*. See Figure 4.19(a).

4. $\mathbf{R_{upnet2}}$ is the symbol for the net uplift after subtraction of the gravity dead load from the exterior gable-end wall above, *when the second floor joists span parallel to the exterior wall below.* See Figure 4.19(b).

Thus, only the wall weight can be subtracted from the uplift load $\mathbf{R_{up}}$. Some engineers may ignore the wall weight as insignificant, particularly when the

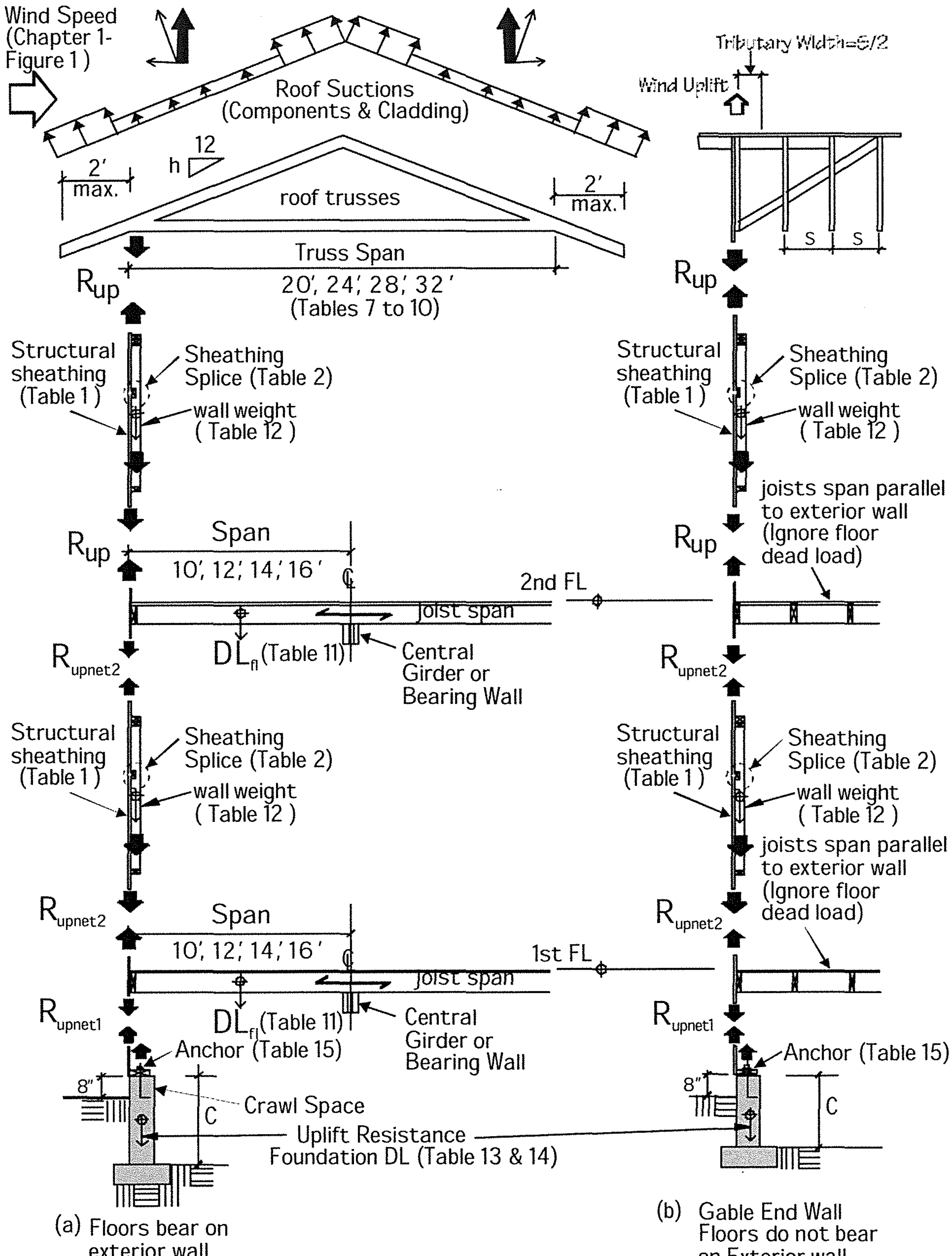


Figure 4.19 - Wind Uplift Design-2 Stories

floor framing does not bear on the exterior gable-end wall. This would simplify the uplift load to $\mathbf{R_{up}}$ at all floor levels and also for the foundation resistance required. See Figure 4.19(b).

5. All other Tables found in Appendix A can be used for two story residences just as for one story residences.

Conclusions

This Chapter concentrated on the engineering process to design the Uplift Resistance System from the roof to the foundation. A one-story building was used as an example for the tension chain design for two design approaches:

1. Structural panel sheathing across the entire residence, and

2. Metal connectors used exclusively, sheathing the walls with non-structural insulation board.

A two story example would repeat many items of the procedure, except for the second floor location. Notes are provided for that case.

NOTE: The use of a single manufacturer for the two example problems was to maintain consistency during the solution process. It is clear that other manufacturers can provide equivalent or similar metal connectors to those selected.

Nomenclature

The following variables will be used to relate Figure 1 in Chapter 4 to the Tables found in Appendix A:

R_{up} = Uplift (tension) force per foot of building length, *based on direct uplift of wind across the gable roof.* This uplift value exists between the roof trusses and the top of the double top plate of the stud wall. Variables include roof slope and wind speed in MPH for the geographic location where the residence is located. Appendix A Tables A.3 through A.6 are for one story building Tables A.7 through A.10 are for two story buildings of widths of 20', 24', 28' and 32' respectively. Interpolation between Tables is permissible for actual building widths between those cited. Roof dead load is assumed to be 15 psf and is described in each Table's footnotes. Roof trusses spaced at 24" o.c. with a maximum overhang of 2 feet are assumed in the design Tables. Only 2/3ths of the dead load is assumed to counter the wind uplift force, based on all three model Codes.

DL_{fl} = The floor dead load in pounds per foot of building length. This value is to be used only when the floor joists span perpendicular to the exterior wall. Only 2/3rds of the actual dead load is found in the Tables, since this is the maximum dead load that can be counted on to resist wind uplift. The span is measured from the exterior face of the exterior wall to the centerline of the first interior bearing for these joists, whether a steel beam, a wood girder or a wood stud wall exists.

T_{all} = The allowable tension capacity of structural panels, either OSB board or plywood for its face grain running parallel or perpendicular to the vertical direction of tension wind uplift. The tension is measured in pounds per foot of building length.

$Wall_{DL}$ = The stud wall weight with choices of interior and exterior sheathing materials. The values are in pounds per foot of wall length. Only 2/3rds of the actual wall weight s are found in the Tables since this is the maximum dead load that can be counted on to resist wind uplift to comply with the three model Codes.

R_{upnet} = The net uplift force at the foundation level after all applicable dead load subtractions have been made to R_{up}. The magnitudes are in units of pounds per foot of building length.

H_{avg} = The average height in feet of a rectangle producing an equivalent area to the triangle formed at the gable end wall.

G_{DL} = The gable end wall weight in pounds per foot of gable end wall length.

H_L = The span (in feet) of any wall opening; door, window or garage door.

R_H = The concentrated uplift reaction at the end of an opening in pounds.

h = Roof slope in inches per each foot of horizontal roof run.

t = Gross thickness of structural panel in inches.

S = Spacing of metal connectors or other fasteners in inches.

C = Depth of foundation from the top of footing to the top of the foundation wall in feet.

Selected References

4.1 "Wood Construction Connectors", *Simpson Strong-Tie Company, Inc.*, Catalog C-99, January 1999.

4.2 "Lumber Connectors - USP Full Line Catalog", *United Steel Products Company*, Copyright, 1998, formerly Kant-Sag -Silver Lumber Connectors.

4.3 Simpson, p. 62.

4.4 Ibid., pp. 61-62.

4.5 Ibid., p. 57.

4.6 Ibid., p. 57-58.

4.7 Ibid., p. 14.

4.8 Ibid., p. 16.

4.9 Ibid., p. 17.

4.10 Ibid., p. 22.

4.11 Ibid., p. 61.

4.12 Ibid., p. 60.

Chapter 5 - Numerical Examples - The Stability System

Introduction

The stability issues of **Overturning and Sliding** comprise the last, but most important of the structural design issues for the lateral load resistance system. This Chapter will define the procedure step-by-step to deal with roof and floor diaphragms, shear walls and shear wall anchorage by the traditional shear wall design method, as well as the perforated shear wall empirical design method. This will be followed by a complete numerical example.

Overview of the Stability System

The stability of the total residence involves its resistance to **overturning** and **sliding** from wind as a total building. This involves the selection of structural panels for the roof and floor diaphragms, and the location and quantity of structural panels for shear walls. The user has many options to satisfy these resistance requirements. The structural planning issues were described in Chapter 3. Although many choices are available, there are better and worse choices that will influence the stiffness of the building and thus determine the building's sidesway movement and torsional twisting characteristics. Chapter 3 should be read before attempting to work the numerical example in this Chapter.

The wind loads used for the stability design are those of category two, the ***Main Wind-Force Resistance System***, described earlier. Wind pressures, both positive and negative, hit large tributary surface areas perpendicular to the wind direction. These surfaces transfer the wind pressures and suctions to the roof and floor planes and then to the vertical wall planes parallel to the wind direction.

Now, rather than use the vertical wind components, as done for uplift, the horizontal components of the wind will be applied to the building. The user should review Figures 10 and 11 of Chapter 1 in the *WMM,* along with all the Figures and text of Chapter 5 in the *WMM*.

Wind resistance for stability is generally an engineering review in two primary directions:

(a) Perpendicular to the long dimension of the building, and
(b) Perpendicular to the short direction of the building.

One Story Residences

The appropriate Design Tables in Appendix **B** are referenced in the Figures presented in this Chapter to help simplify the approach to design of the Wind Stability system. The nomenclature and definitions are at the end of the Chapter. The Stability issues for all one-story residences will utilize the following expanded-version procedure. A short form procedure is also included for frequent users who no longer need the elaborate explanations.

General Design Procedure - Expanded Version

1. Determine the geographic location for the residence. Use the Wind Speed map found in Chapter 1 Figure 1. Regions where a 70-MPH wind speed is shown should be increased to 80 MPH. The minimum wind speed recommended by this author is **80 MPH,** and that is the minimum wind speed assumed by the Tables found in the Appendices.

2. Establish the following parameters for a review of diaphragm selection and their associated shear walls for the residence:

   A. Potential Shear Wall Locations:

a. Shear walls are best located at the perimeter of the residence, since foundations exist below the walls. Also, shear walls should be located as far from the center of the building as possible to minimize twist due to unsymmetrical wind directions.

b. Shear walls are most efficient when they also function as bearing walls that carry gravity dead loads from the roof and/or floors.

c. Consider two wind directions:

(1) Wind perpendicular to the ridge of a gable or hip roof, and

(2) Wind parallel to the ridge of a gable, hip or monoslope roof. Some building configurations may require more than two mutually perpendicular directions of wind orientation, but that is beyond the scope of this Manual.

B. Diaphragm Proportions:

a. Once potential shear walls have been identified in both directions, assess the length to width ratio for each diaphragm. This aspect ratio is a rough way to control deflection of the diaphragm and should not exceed 4 [1] , [2]. The aspect ratio L/W, as it applies to sloped roof planes, is illustrated in Figure 1 on the next page.

The structural behavior of sloped diaphragm planes, as constructed for gable or hip roofs is assumed to approximate that of a folded plate roof. This implies that the diaphragm shears transfer from one sloped side to the other. This assumption is valid when the structural sheathing panels butt against each other at the ridge. However, continuous ridge vents are very common and their ability to function is dependent on the sheathing stopping short of the ridge peak to allow for an air space. This slight separation of the two sloped diaphragm planes influences the transfer of shear across the ridge. Most engineers ignore this issue. Figure 2, two pages ahead, illustrates the possible ridge diaphragm conditions. The recommended solution is to add steel straps across the ridge when rafters are used to frame the roof. When trusses frame the roof no straps are required.

b. Note openings in floor diaphragms for stairs, atriums or two story spaces. The limits on opening size and location [3] are also shown in Figure 1.

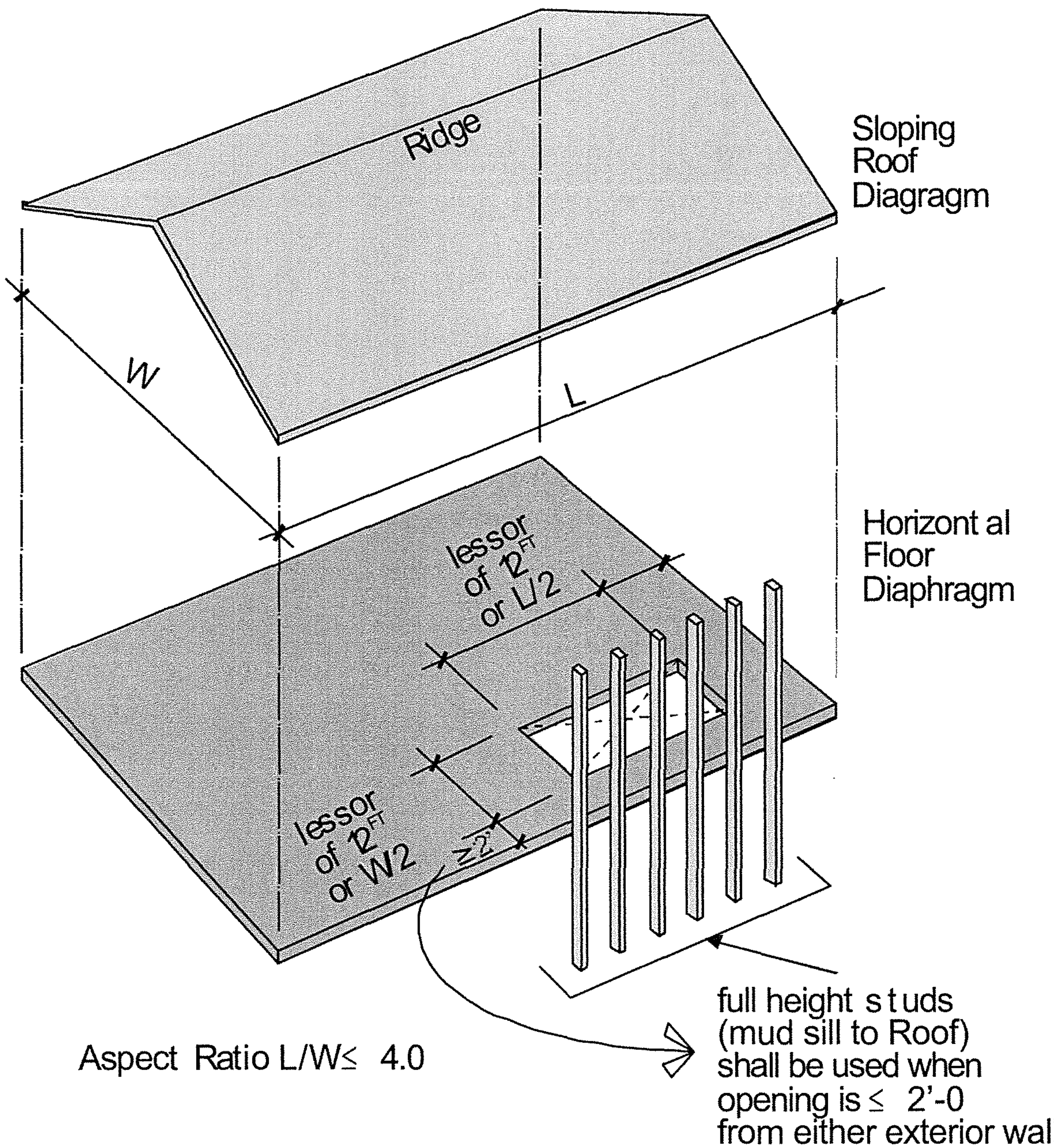
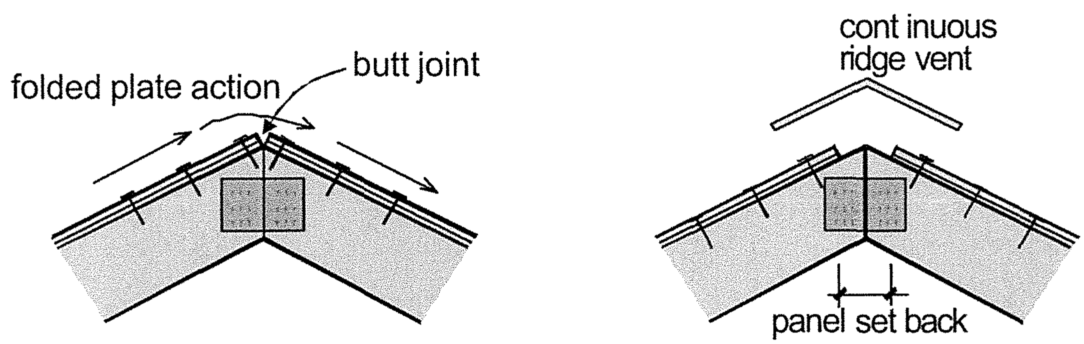
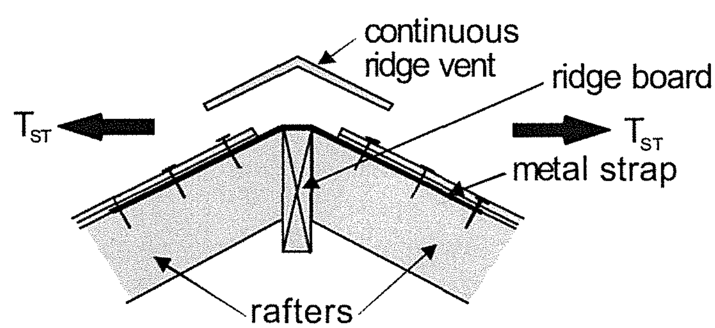


Figure 1 - Diaphragm Recommendat ions
from SBC & UBC



(a) Roof Trusses



(b) Roof Rafters

Figure 2- Diaphragm Continuity @ Ridge

C. **Roof & Floor Framing Components and wood Species:**

a. Determine the **type** of roof-framing components, i.e. trusses, dimension lumber rafters or solid web"I"joists and rafters.

b. Determine the **species** and **width** of lumber used for the members directly in contact with the structural sheathing panels.

Dimension lumber is graded either visually or by machine stress-rating. A corresponding grade stamp [4] [5] [6] is placed on each piece of lumber. Sample grade stamps for both techniques are shown in Figure 3 two pages ahead. Figure 3 explains how to read the information contained on these grade stamps. Such information should be readily available in existing residences from within the attic. Grade stamps on lumber for new construction is even easier to find. If the residence is in planning, your contractor will know what grades of lumber he typically uses. Stamps also appear on laminated wood, LVL lumber, parallam lumber, solid web "I" joists and roof or floor truss top chords.

c. Repeat steps (a) and (b) for the floor framing components.

3. Select structural panel thickness and grade for the roof and floor diaphragms based on gravity dead and live load.

Gravity loading will most often determine the structural panel thickness and the required diaphragm thickness for in-plane wind load will merely be a check. The BOCA Code [7] SBC [8] and The APA [9] all provide Tables for minimum thickness of structural panels based on framing spacing and gravity loads. Tables 25 and 26 in the Appendix **B** [9] should be reviewed. The span rating for the selected panel thickness should be noted for reference in later steps of this procedure. Both Tables include APA Rated Sheathing, which is the most common structural panel used for subfloors and roof sheathing. APA Rated Sturd-I-Floor is a combination subfloor-underlayment, providing a smooth surface for application of carpet and pad. APA Rated Sheathing is generally selected based on the spacing of structural framing below and the roof and floor live load to be supported.

Residential floor live loading is by all Codes 40 psf.

A typical roof live load for sheathing selection is conservatively based on snow loading of 30 psf, whether it exists or not.

Table 25 gives allowable live loads, while Table 26 gives nail size and spacing for the selected panel thickness and spacing.

Structural panels used for sheathing incorporate two types of grade stamps as illustrated in Figure 4 two pages ahead. The current stamp style [10] is shown in Figure 4(a), while the older stamp style [11] found in many existing homes from the 50's to the 80's is shown in Figure 4(b). The nomenclature within each stamp type is defined to assist the user in reading grade stamps. The "Span Rating" relates the center to center spacing of structural framing that supports the panels. For example, 32/16 means that the panel is rated for a maximum roof framing spacing of 32" on center and 16" on center for floor framing. These panels can carry the live loads indicated in Table 25 and 26 in Appendix **B**.

4. Select the appropriate roof and floor diaphragm force (plf) from Tables 1 through 4 in Appendix **B**.

A. Wind direction: Perpendicular to the ridge of a sloped roof.

Each respective Table is highlighted in large boldface type in the upper right corner ($\mathbf{F}_{\perp}$) with building width to the side. Enter the selected Table for roof (rafters or truss) span (**W**), roof slope "**h**" and **wind speed**. Read the lateral wind load to the roof diaphragm ($\mathbf{F_{RF}}$) and the first floor diaphragm force ($\mathbf{F_{F1}}$) in pounds per foot of building length (**L**).

Figure 5 on the next page illustrates the tributary areas of wind pressure and suction that determines the diaphragm force at the roof/ceiling plane and at the first floor plane for Tables 1 to 4 in Appendix **B**. It should be clear from summing horizontal forces from the roof down to the foundation that the roof force ($\mathbf{F_{RF}}$) will be smaller than the first floor ($\mathbf{F_{F1}}$) force in plf. Thus, the structural requirements of the diaphragms become larger as the user moves from the roof diaphragm, to the second floor diaphragm and to the first floor diaphragm.

Figure 5 requires further explanation to fully understand how the values of the diaphragm forces at the roof and floor are calculated.

The following discussion will start with a simple flat roof diaphragm and move to the more difficult sloped or "gabled" roof diaphragm, and finish with first floor diaphragms.

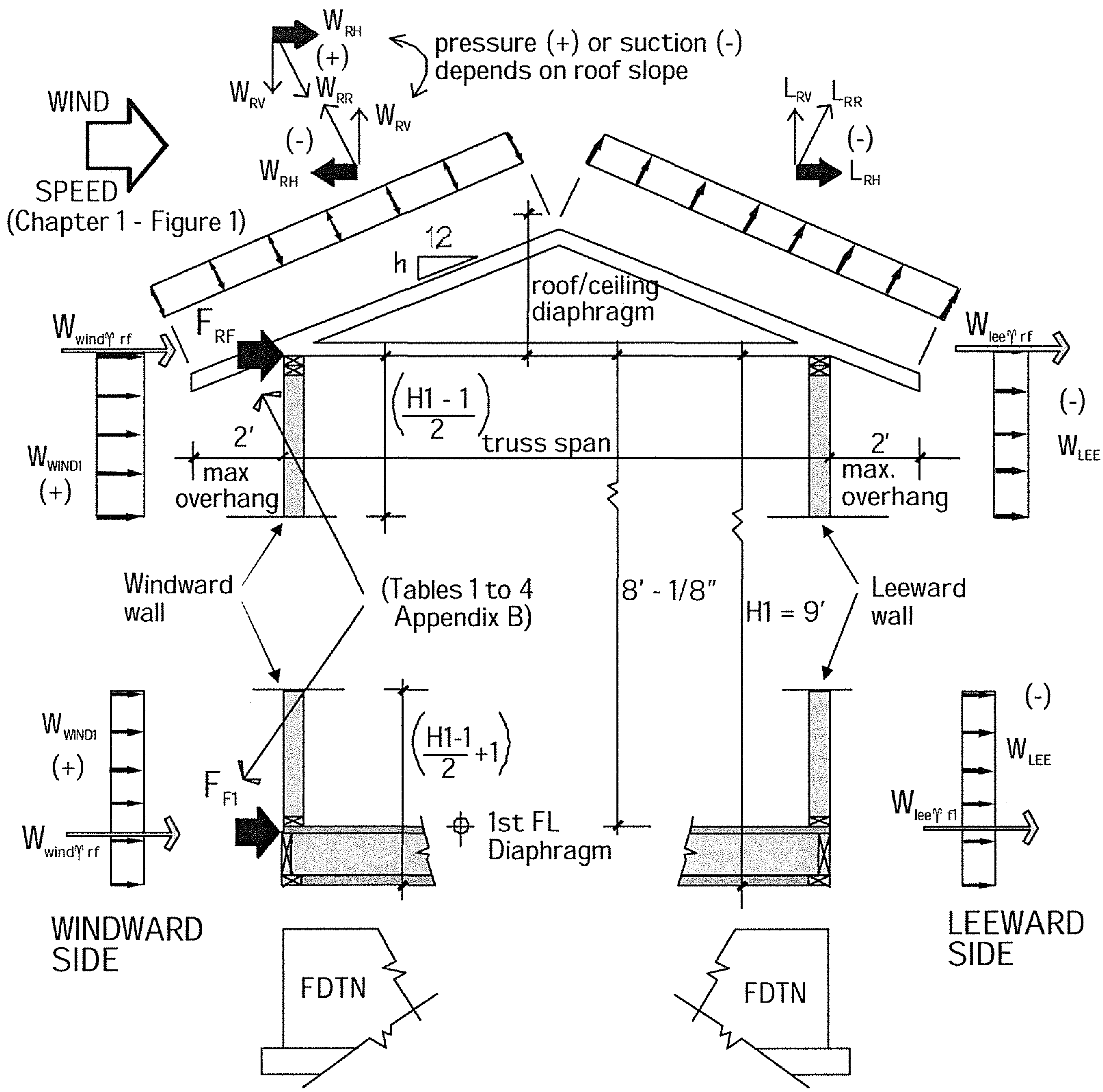


Figure 5.5 - Wind Load to Diaphragms Procedure for 1-Story Residence Wind Perpendicular to Ridge

Flat Roof Diaphragms

Figure 5.6 illustrates in perspective the structural behavior of flat roof diaphragms, the assumed model, the transfer of force between roof plane and shear wall and the uplift potential of shear walls. It is time to clarify the vast number of force arrows shown in Figures 3 through 8 in Chapter 5 of the *WMM*. All of those Figures attempt to graphically explain what will be stated now. Also, the nomenclature to be used throughout this Chapter requires being very specific. A thorough study of Figure 5.6 will now be presented.

The wind direction as shown is from left to right and is perpendicular to the surface of the long walls. The windward wall receives positive wind pressure and the leeward wall receives negative wind pressure (suction). These walls bend vertically in their plane and transfer a reaction to the double top plate and the bottom plate of each wall stud. Figure 3 in Chapter 5 of the *WMM* isolates one stud and illustrates this transfer of windward pressure to the top and bottom of the wall and then equal and opposite to the roof and floor diaphragms. This same description applies to the leeward studs.

Thus, the roof diaphragm receives a tributary width of wind pressure on the windward wall equal to the upper portion of the wall height, defined by the equation shown in Figure 5.5. This load to the diaphragm becomes a lineal load ($\mathbf{w_{wind\perp rf}}$) in the plane of the roof diaphragm along the windward edge. The wind transfer occurs similarly on the leeward side of the roof diaphragm, producing a lineal load ($\mathbf{w_{lee\perp rf}}$) in the plane of the diaphragm along its leeward edge. The flat roof surface receives a vertical negative pressure (suction) and therefore contributes no horizontal force component to the roof diaphragm. The total wind load ($\mathbf{F_{RF}}$) to the flat roof diaphragm is then the sum of the windward and leeward lineal in-plane loads in units of lbs. per foot of building length ($\mathbf{L}$).

$$\mathbf{F_{RF}} = \mathbf{w_{wind\perp rf}} - \mathbf{w_{lee\perp rf}}$$

The flat roof diaphragm spans between the exterior sidewalls in Figure 5.6, which are the potential shear walls. These walls will be sheathed with structural panels and anchored to the foundation, so they will be stiff walls and will receive the reaction ($\mathbf{R_{RF\perp}}$) of the linear wind load ($\mathbf{F_{RF}}$). This is true since the structural model for a diaphragm is a deep wide flange beam spanning between stiff supports - the shear walls. The structural model is

Important Connections
1. Web panel to panel
2. Web panel to shear wall
3. Flange to web
4. Flange splices
5. Foundation Anchorage

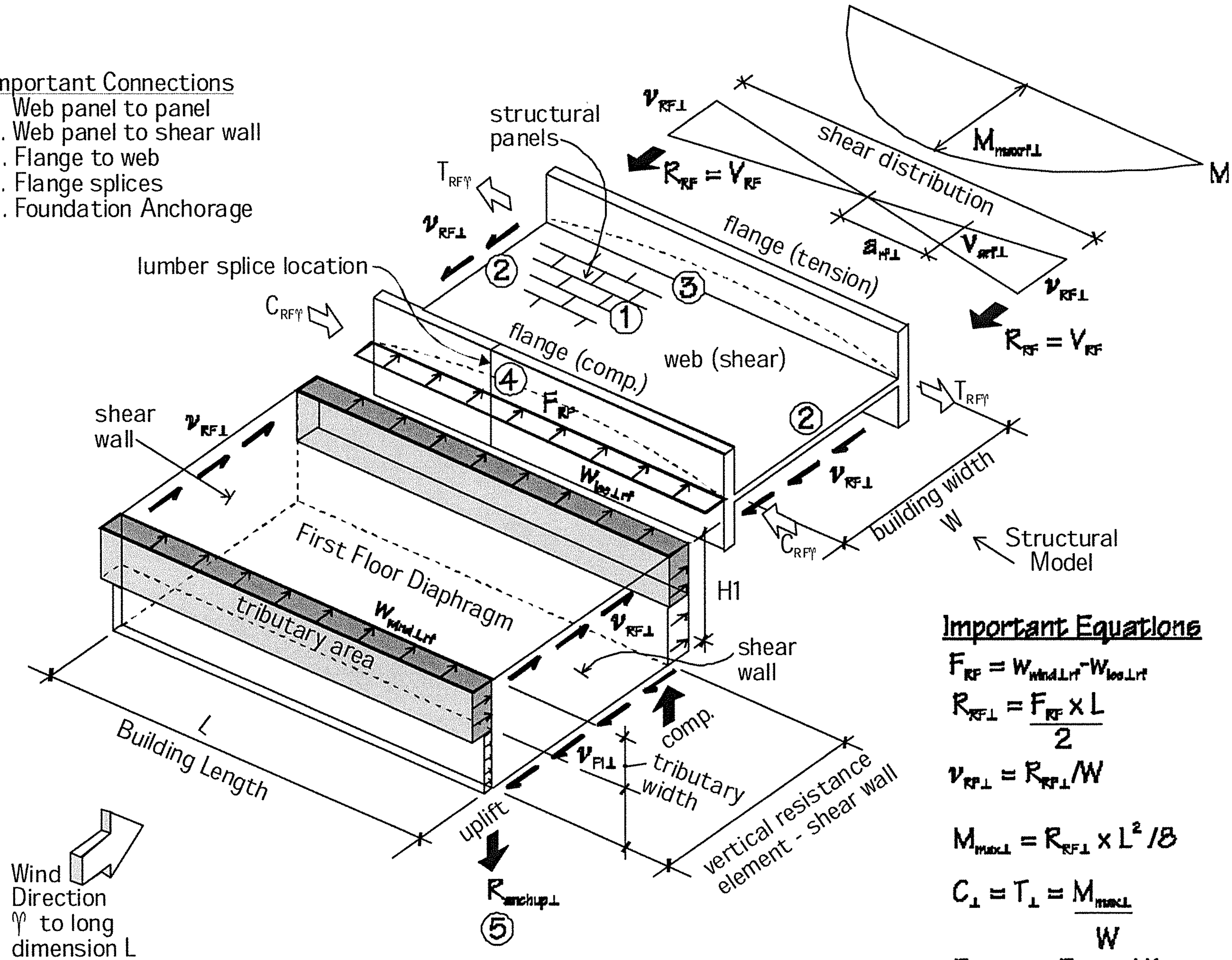


Figure 5.6 - Flat Roof Diaphragm Behavior

Important Equations

$$F_{RF} = w_{wind\perp rf} - w_{lee\perp rf}$$

$$R_{RF\perp} = \frac{F_{RF} \times L}{2}$$

$$v_{RF\perp} = R_{RF\perp}/W$$

$$M_{max\perp} = R_{RF\perp} \times L^2/8$$

$$C_\perp = T_\perp = \frac{M_{max\perp}}{W}$$

$$R_{anchup\perp} = \frac{R_{RF\perp} \times H1}{W}$$

illustrated in the center of Figure 5.6, where a wide flange beam is set on its side. The web of the beam model is the roof diaphragm. One must imagine, as illustrated, that there are structural panels of plywood or OSB in 4'x8' sheets across the surface of the web. The wind load ($\mathbf{F_{RF}}$) uniformly loads the top flange of the beam in the plane of the web. The beam bends and deflects, as shown by the dotted lines in the model. Each diaphragm of a residence is assumed to become a simply supported beam subjected to a uniform wind load. The reaction ($\mathbf{R_{RF\perp}}$) at each end of the diaphragm span is a force equal to the maximum shear ($\mathbf{V_{RF\perp}}$) in pounds as shown by the shear diagram. Conversion of the shear force to a unit shear ($\mathbf{v_{RF\perp}}$) in plf along the diaphragm edge is easily performed by the equations shown in Figure 5.6 and shown below:

Roof/Ceiling Diaphragm Reaction: $\mathbf{R_{RF\perp}} = \mathbf{V_{RF\perp}} = \mathbf{F_{RF}} \times \mathbf{L/2}$ lbs.

Roof Diaphragm shear: $\mathbf{v_{RF\perp}} = \mathbf{R_{RF\perp}} \div \mathbf{W}$ plf

It should now be clear that all the one-sided arrows in Figure 5.6 represent shear along the entire edge of the diaphragm. This edge shear ($\mathbf{v_{RF\perp}}$) at the roof and the edge shear at the floor ($\mathbf{v_{F1\perp}}$) are the required shears for which each diaphragm design is based. It should be noted that since the web is composed of discontinuous sheets, the nailing of those sheets to the structural framing below is critical to providing the proper shear strength.

Gable Roof Diaphragms

The above discussion focused on the roof diaphragm. It is now important to discuss sloped roof diaphragms, mainly gable and hip roof forms. Figure 5.7 is a mirror image of Figure 5. 6 with the exception of the roof form. Also, Figure 5. 7 illustrates that the structural wide flange beam model has a bent web. It is assumed that the bent web behaves as a folded plate and provides continuity across the ridge. Figure 5. 2 indicated all the possible ridge conditions one might confront. Even with continuous ridge venting, when the two slopes become independent diaphragms, they are assumed to sum and provide the shear capacity of one large diaphragm.

A larger wind load ($\mathbf{F_{RF}}$) occurs at the roof/ceiling plane, due to the additional horizontal components of wind from each sloping plane. As shown in Figure 5.5, there is a force in plf from the windward slope ($\mathbf{W_{RH}}$)

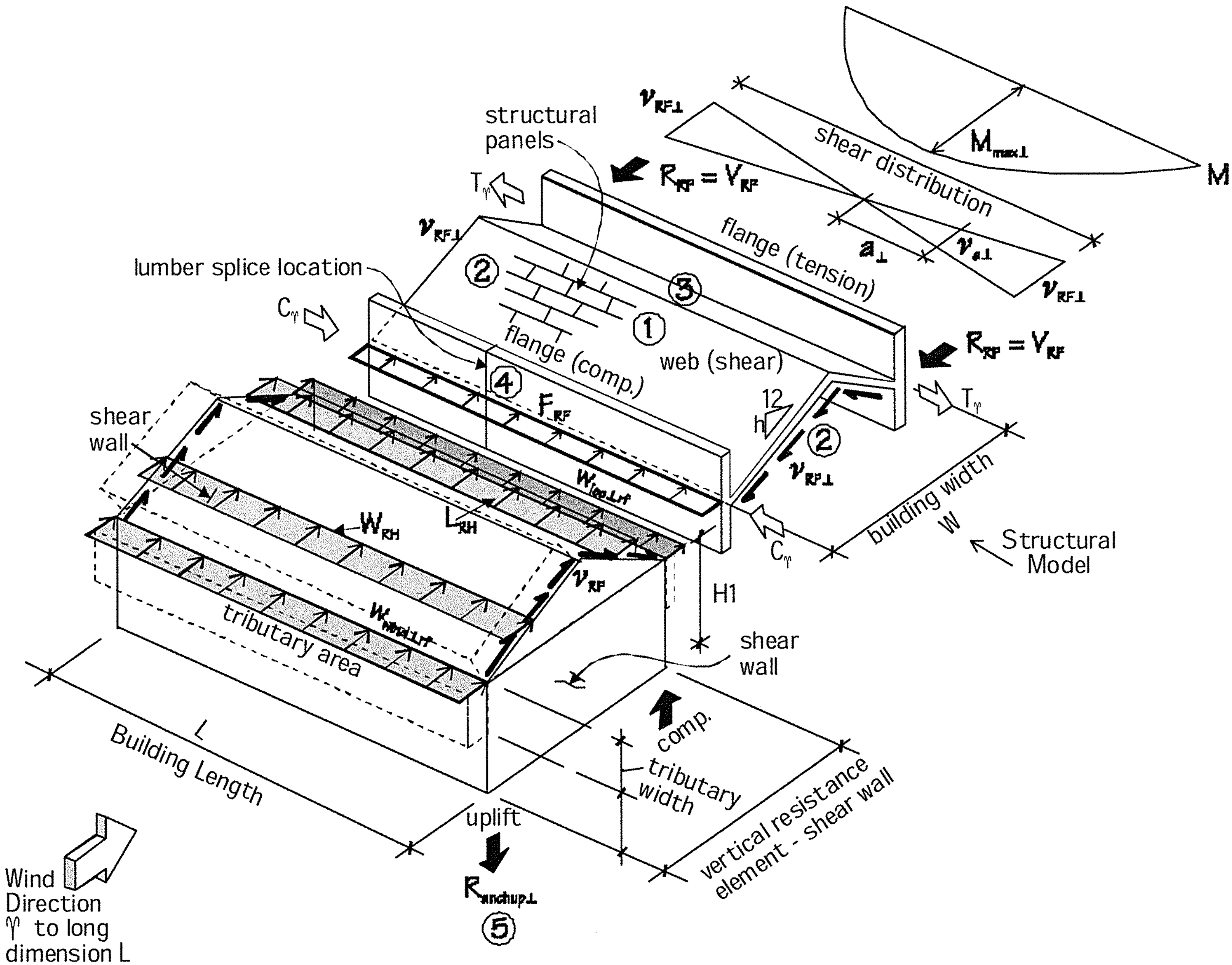


Figure 5.7 - Sloping Roof Diaphragms

that is either positive or negative, depending on the roof slope (h). Also, there is a force in plf from the leeward slope (W_{LEE}) that is always negative (suction). These two roof components add to the wall components ($w_{wind\perp rf}$ - $w_{lee\perp rf}$). Thus, the equation becomes:

Sloped Roof Diaphragm wind load: $F_{RF} = w_{wind\perp rf} - w_{lee\perp rf} + W_{RH} - L_{RH}$

NOTE: If the above answer is (+) then F_{RF} is a force in plf to the right, and if the answer is (-) then F_{RF} is a force in plf to the left. Positive is in the direction of the wind, while negative is in the direction opposite to the direction of the wind. Tables B.1 through B.8 in Appendix B are for sloped roof forms, and the tabled values of F_{RF} , F_{F1} and F_{F2} have taken signs into account in tabulating values.

First Floor Diaphragms

Figure 5.5 illustrates the first floor diaphragm load F_{F1}, but again requires further explanation regarding how the in-plane diaphragm force is determined. Figure 5.8 is an exploded perspective view of the same one-story residence with a flat roof found in Figure 5.6. The diaphragms are separated from the shear wall and the foundation. The fact that the illustration shows a flat roof is immaterial, since the same approach is used to define the first floor diaphragm force, regardless of roof form.

Again, the windward wall receives positive wind pressure and the leeward wall receives negative wind pressure (suction). The wall studs bend vertically in their plane and transfer a reaction to the bottom plate of the structurally sheathed stud wall.

Thus, the first floor diaphragm receives a tributary width of wind pressure on the windward wall equal to the lower portion of the wall height, defined by the equation shown in Figure 5.5. This load to the diaphragm becomes a lineal load ($w_{wind\perp f1}$) in the plane of the floor diaphragm along the windward edge. The wind transfer occurs similarly on the leeward side of the floor diaphragm, producing a lineal load ($w_{lee\perp f1}$) in the plane of the diaphragm along its leeward edge. The total wind load (F_{F1}) to the floor

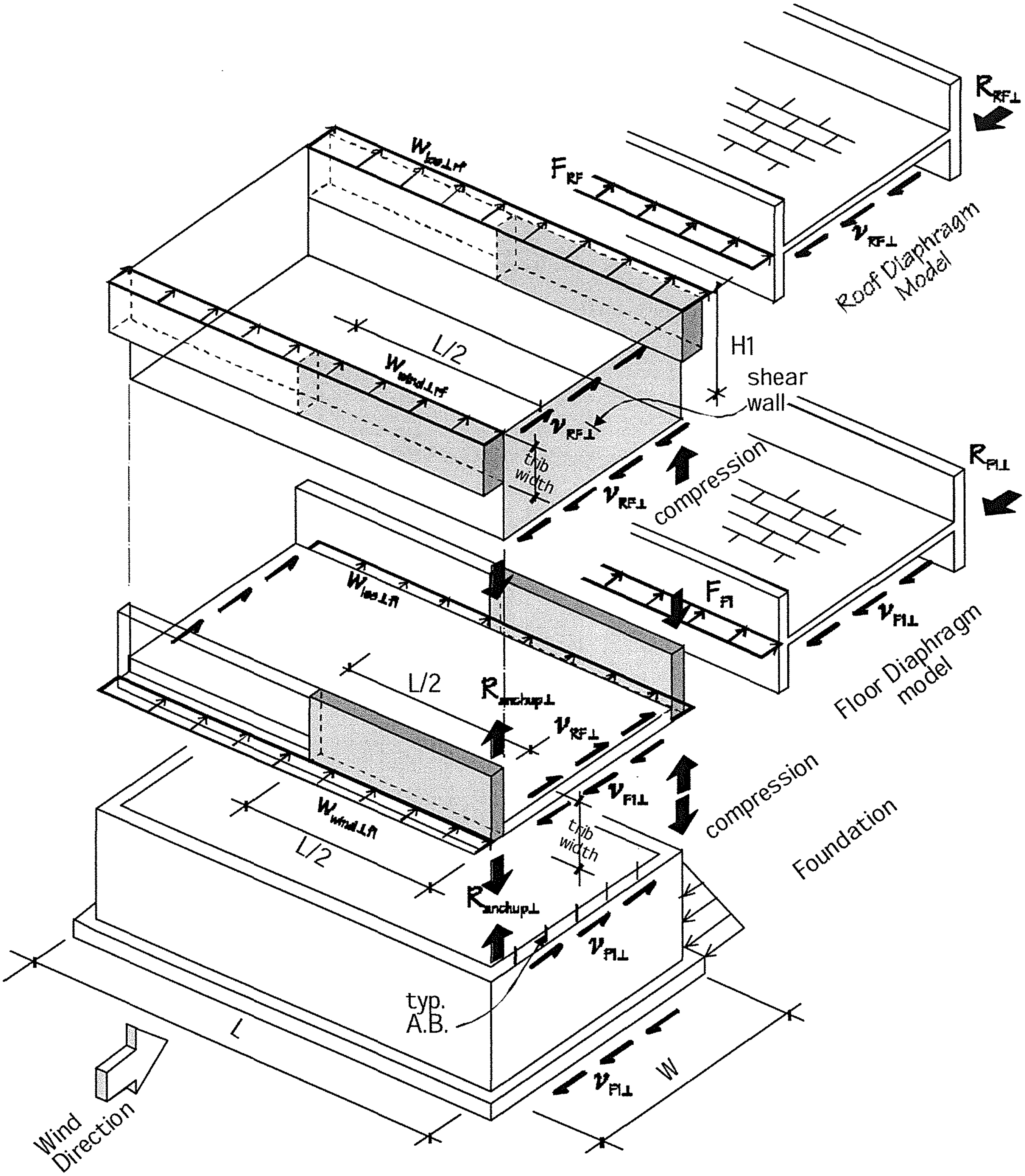


Figure 5.8 - Exploded View of One Story Diaphragms and Shear Wall

diaphragm is then the sum of the windward and leeward lineal in-plane loads plus the roof diaphragm load:

$$F_{F1} = (w_{wind\perp f1} + w_{lee\perp f1}) + F_{RF} \quad \text{(plf)}$$

The flat roof and first floor diaphragm span between the exterior shear walls; however, only one shear wall is shown in Figure 5.8 for clarity. The roof diaphragm must transfer its reaction ($R_{RF\perp}$) in shear ($v_{RF\perp}$) to the shear wall. The equations to calculate the reaction and shear at the roof diaphragm are shown above under the discussion of the flat roof diaphragm.

Note the single-sided arrowhead symbols for the diaphragm shear in Figure 5.8, and in turn the equal and opposite repeat of these arrows to illustrate the transfer of $v_{RF\perp}$ to the shear wall. This shear in plf must be transferred over the entire depth ($H1$) of the shear wall. This is the function of the structural sheathing panels. This topic will be discussed further on in the shear wall design part of the process.

The bottom of the shear wall, which rests on the first floor diaphragm, must be attached to the floor with a connection method sufficient to transfer the shear $v_{RF\perp}$ into the floor diaphragm. Simultaneously, the first floor diaphragm receives the uniform wind load (F_{F1}) in its plane. Thus, the first floor diaphragm reaction and unit shear becomes by equation:

First Floor Diaphragm Reaction: $R_{F1\perp} = F_{F1} \times L/2$ lbs.

First Floor Diaphragm shear: $v_{F1\perp} = R_{F1\perp} \div W$ plf

The shear $v_{F1\perp}$ not only is used to design the diaphragm, but also the shear transfer into the foundation. It should be clear from the elaborate discussion above that the shear at the first floor diaphragm and the resulting shear transfer to the foundation is significantly larger than ($v_{RF\perp}$).

This shear wall will be sheathed with structural panels and anchored to the foundation. Resistance to *sliding* will be resisted by anchor bolts in shear, while tie-down anchors will resist *overturning* of the shear wall.

5. Determine the maximum shear (v) in (plf) for each diaphragm section by the following equations repeated from step #4:

A. **Wind Direction:** *Perpendicular* to long dimension of diaphragm:

Roof/Ceiling Diaphragm Reaction: $R_{RF\perp} = F_{RF} \times L/2$ lbs.

Roof Diaphragm shear: $v_{RF\perp} = R_{RF\perp} \div W$ plf

First Floor Diaphragm Reaction: $R_{F1\perp} = F_{F1} \times L/2$ lbs.

First Floor Diaphragm shear: $v_{F1\perp} = R_{F1\perp} \div W$ plf

6. Select the appropriate APA rated structural sheathing panel grade and thickness, nail size and spacing, nail pattern across the diaphragm and blocking requirements from Table B.27 in Appendix B [5.12]. There is a significant amount of information in Table B.27 and one must be careful where to enter the Table to locate certain information. Also, there are many notes that add significantly to the choices made on any particular item. The following order is recommended to properly use Table B.27:

a. Select panel grade.

The Table is broken into panel grades "APA Rated Sheathing" and "APA Structural 1". It was recommended in Step 3 to use "APA Rated Sheathing" for economy and select a panel **Span Rating** for the sheathing that provided the required roof live load. Thus, enter Table B.27 with the thickness selected for gravity loading within the horizontal grouping of "APA Rated Sheathing".

b. Select framing lumber Species and Specific Gravity from Table B.30 in Appendix B [5.13].

Move horizontally along the row for the panel thickness and move all the way to the column labeled **Framing Species**. The wood species in this column group refer to those species with a **specific gravity** of **0.49 or greater**. These species would most likely include Southern Pine, Douglas Fir, Douglas Fir-Larch or "I" solid web joists. The structural-framing members that the structural panels would be nailed to must be of these species. Otherwise, move to the last column group, which is based on wood framing of species with a specific gravity less than 0.49 but greater than or equal to 0.42. These species would most likely include S-P-F or Hem-Fir.

c. Select nominal framing width.

There are two choices for nominal width of framing members below the structural panels. The **2" nominal width** refers to dimension lumber of an actual width of 1-1/2". This would also apply to roof trusses made from dimension lumber pieces metal plate stamped together. **The 3" nominal width** refers to 4"x2" floor trusses or "I" solid web joists that could have a width of 1-1/2", 1-3/4", 2-5/16" or 3-1/2". Both of these are illustrated in the notes of Table B.27.

d. Compare the required Roof diaphragm shear ($v_{RF\perp}$) in plf with the allowable shear values in plf listed.

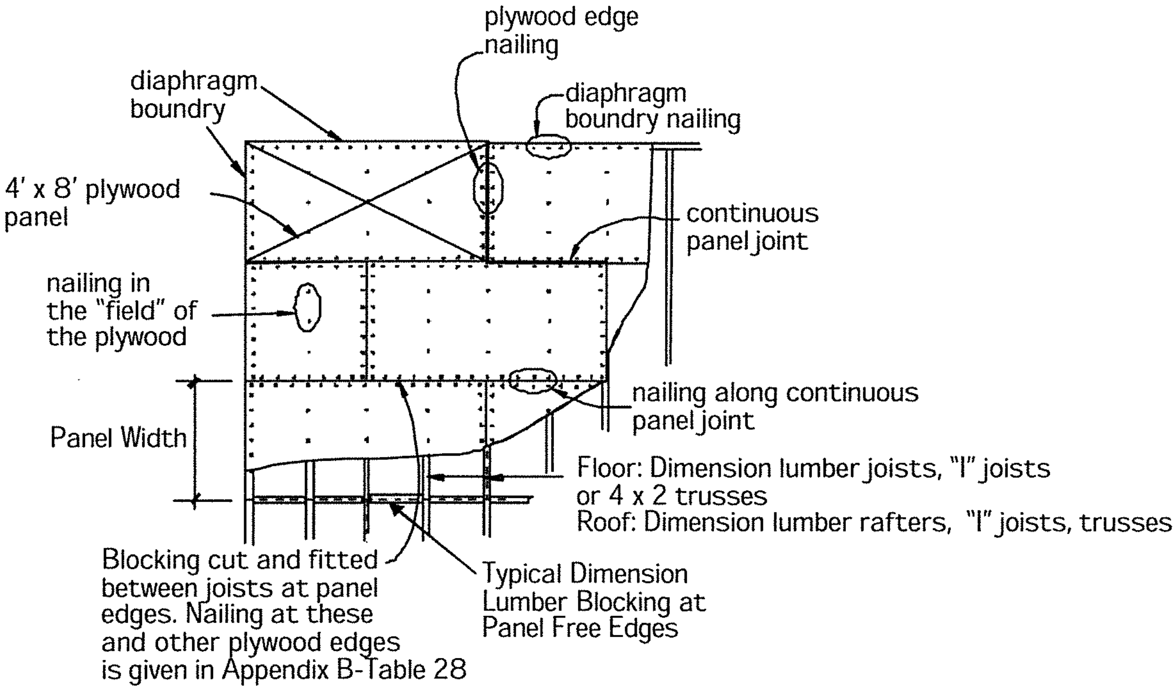
Read across the row for the choice of nominal framing width (2" or 3") until you reach an **allowable shear value** (v_{ALL}) just larger than the required shear.

e. Select nail size and spacing, and whether blocking is required or not.

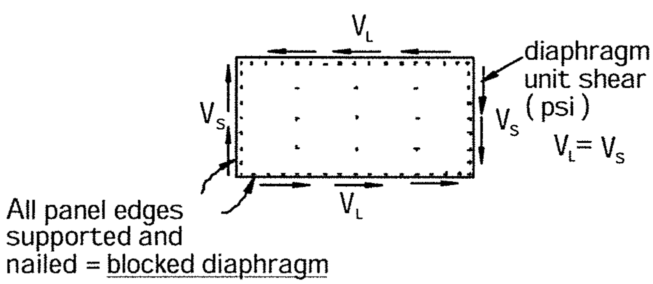
Move vertically from the selected allowable shear and read the **nail spacing** of 6", 4", 2-1/2" or 2". Directly above that it will indicate if a **blocked diaphragm** is required or a non-blocked diaphragm is required. A blocked diaphragm is illustrated in the notes of Table B.27. Again from the selected allowable shear, move to the left in the same row until the required size of common nail: 6d x 2" lg., 8d x 2-1/2" lg., or 10d x 3" lg. is found. In the next column to the right, read the **required nail penetration** into the framing member below the sheathing. The nail choice and the choice of panel thickness will automatically satisfy the required nail penetration.

f. Draw a sketch of panel nailing pattern.

The terminology is very confusing in the footnotes and the Table column headings with regard to nail pattern and spacing requirements. Figures 5.9 and 5.10 [5.14] illustrate use of the terminology found in Table B.27 for blocked diaphragms and unblocked diaphragms respectively. The panel pattern, whether staggered or not with respect to the wind direction and whether panel joints are continuous with respect to the wind direction are illustrated in the notes for Table B.27. It is always best to stagger the panels, so continuous panel joints run the longer dimension of the diaphragm and are perpendicular to the wind direction. This will maximize the strength of the diaphragm in the worst shear direction.

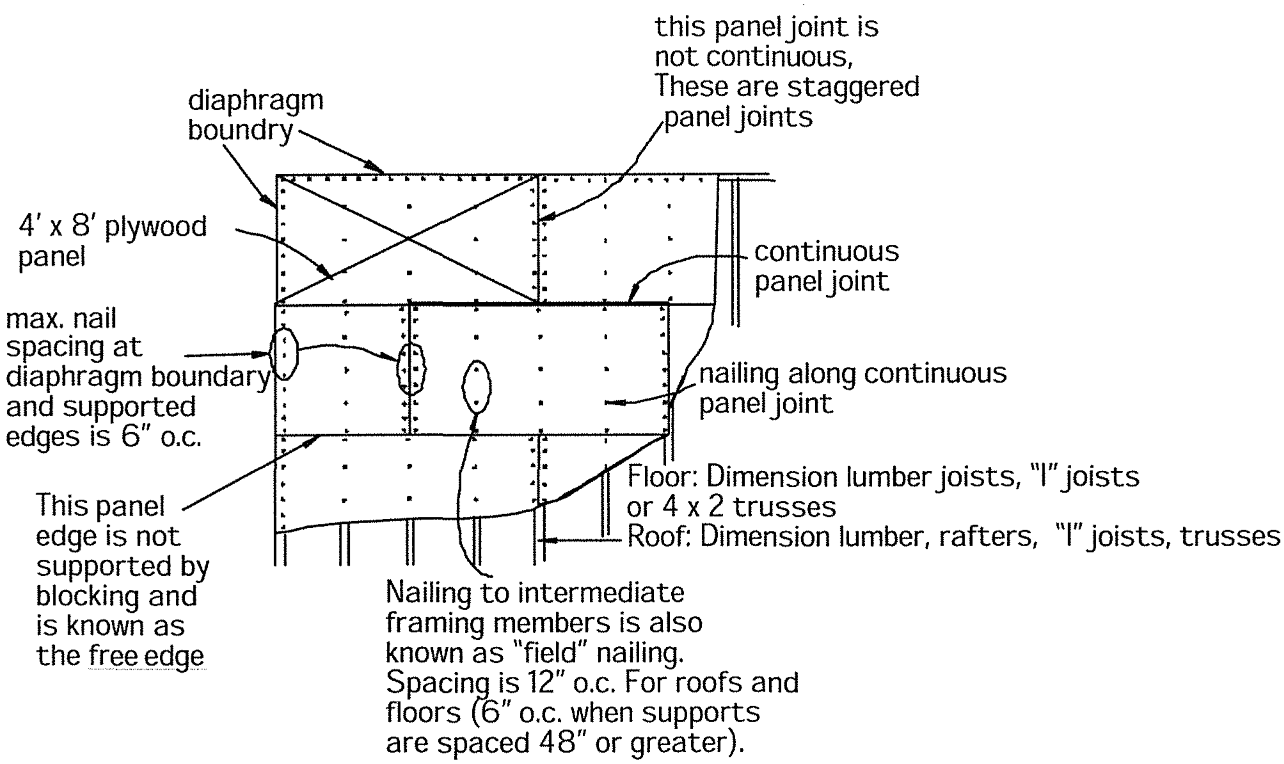


Partial Roof or Floor Diaphragm
in Plan View

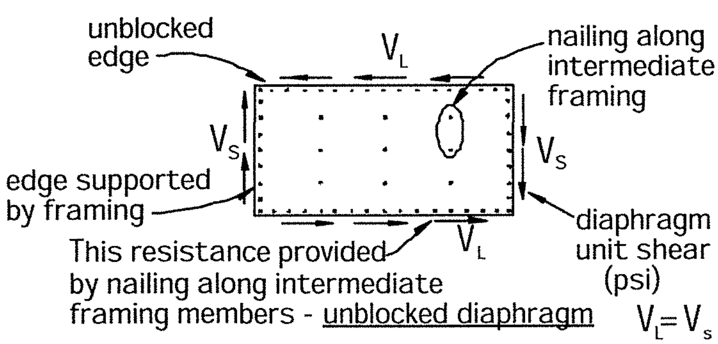


Single Panel

Figure 5.9- Blocked Diaphragm
Nail Pattern & Nomenclature



Partial Roof or Floor Diaphragm
in Plan View



Single Panel

Figure 5.10 - Unblocked Diaphragm
Nail Pattern & Nomenclature

7. Select the appropriate roof and first floor diaphragm force (plf) from Tables B.9 through B.12 in Appendix B.

Wind Direction: parallel to the ridge of a sloped roof.

Each respective Table is highlighted in large boldface type in the upper right corner ($\mathbf{F}_{II}$) with building width to the side. Enter the selected Table with roof slope "**h**" and **wind speed**. Read the wind load ($\mathbf{W}_{RF}$) to the roof diaphragm and the first floor diaphragm force ($\mathbf{W}_{F1}$) in pounds per foot of building width ($\mathbf{W}$).

NOTE: Diaphragm design strength is usually based on a wind direction perpendicular to the long dimension of the building for flat roofs and perpendicular to the ridge for gable roofs. Consider that wind perpendicular to the ridge or building length will create a greater wind area along the longest ($\mathbf{L}$) side of the building. The diaphragm reaction and edge shear will be large. Similarly, wind against a side wall of width ($\mathbf{W}$) will produce a smaller wind load area and therefore a smaller diaphragm reaction and edge shear. Review of diaphragm strength for a wind direction parallel to the long dimension for flat roofs and parallel to the ridge for sloping roofs is usually a check, rather than a design.

Figure 5.11 illustrates the wind components that mathematically will produce the values $\mathbf{W}_{RF}$ and $\mathbf{W}_{F1}$. Consider that wind pressures are exerted across the windward and leeward end walls.

Wind Load to End Walls Below the Triangular Gable End

Wind direction: parallel to the ridge of a gable roof.

The wind direction as shown is from left to right and is perpendicular to the surface of the end walls. The windward end wall receives positive wind pressure and the leeward end wall receives negative pressure (suction). These walls bend vertically in their plane and transfer a reaction to the double top plate and the bottom plate of each wall stud. Figure 5.3 in Chapter 5 of the *WMM* isolates one stud and illustrates this transfer of windward pressure to the top and bottom of the wall and then equal and opposite to the roof and floor diaphragms.

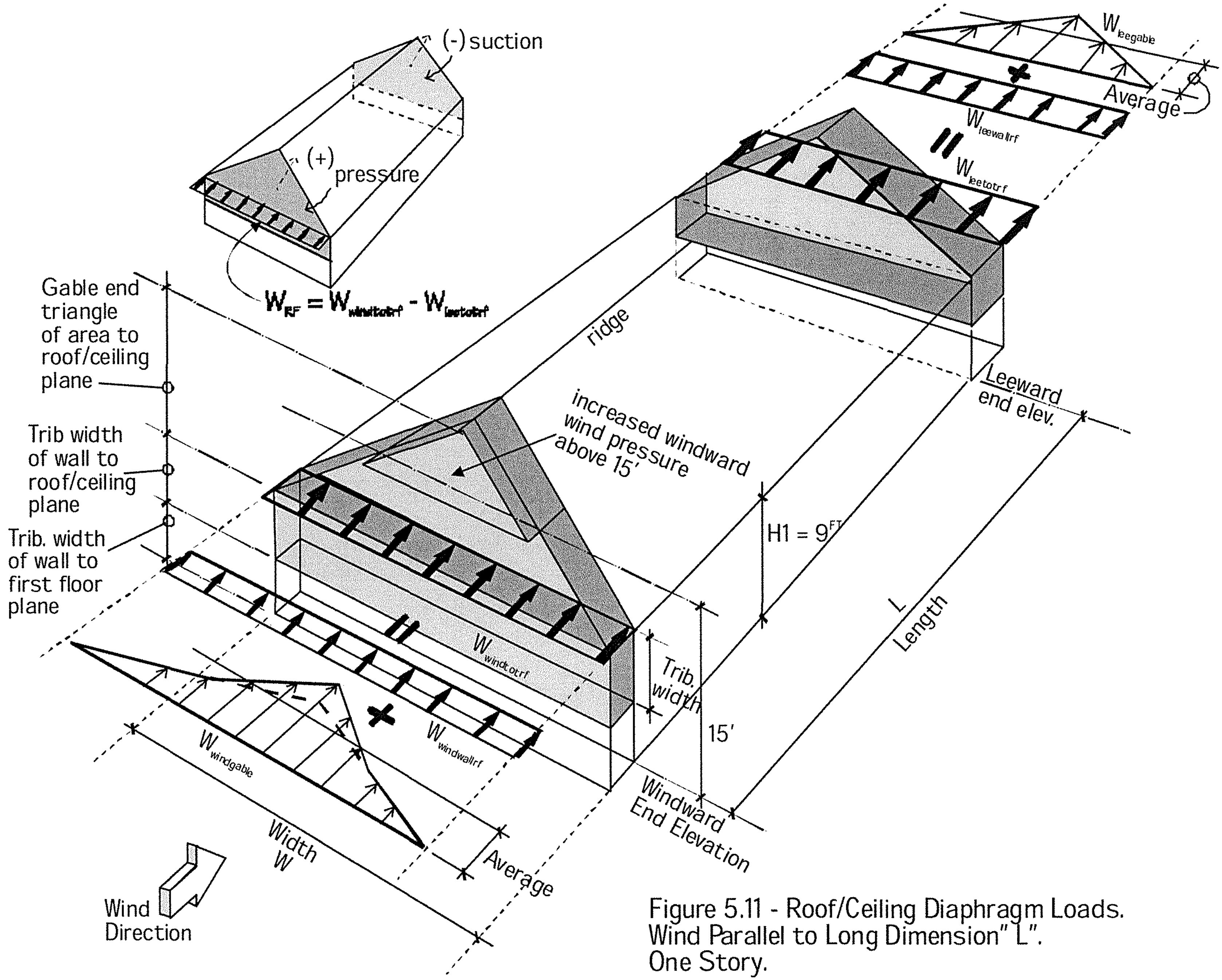


Figure 5.11 - Roof/Ceiling Diaphragm Loads.
Wind Parallel to Long Dimension" L".
One Story.

This same description applies to the leeward studs. Figure 4.3 illustrates the top of the wall and the option of a triangular gable end truss, or a triangular stud wall.

Thus, the roof/ceiling diaphragm receives a tributary width of wind pressure from the windward wall equal to the upper portion of the wall height, defined by the equation for tributary width shown in Figure 5.5. This load to the roof/ceiling diaphragm becomes a lineal load ($\mathbf{w_{windwallrf}}$) in the plane of the ceiling along the windward edge. The wind transfer occurs similarly on the leeward side of the roof/ceiling diaphragm, producing a lineal load ($\mathbf{w_{leewallrf}}$) in the plane of the ceiling along its leeward edge. The sloping roof planes receives a negative pressure (suction) perpendicular to their surface and therefore contribute no horizontal force component to the roof diaphragm.

Wind Load to the Triangular Gable Portion of Windward End Wall

The windward gable portion of the end wall receives positive wind pressure and the leeward gable portion of the end wall receives negative pressure (suction). These triangular walls may be composed of a truss without diagonals or a stud wall with studs of varying length. Either construction technique will bend vertically out of their plane and transfer a reaction to the roof structural panels and the bottom plate of each wall stud resting on top of the double top plate of the stud wall below. The entire triangular area of the gable on the windward side and leeward side will receive wind pressures.

The windward gable is of most interest, since wind pressure on this entire elevation varies with height. The ANSI 7-93 Loads Document considers that wind pressure is constant to a height of 15 feet and then increases in magnitude according to a power curve. This is illustrated in Figure 5.11 by the increased size of the wind pressure triangle against the end wall above 15 feet. The area of the trapezoid × the wind pressure between 0-15 ft. is added to the triangular "tip" area × the larger wind pressure above 15 ft. and then divided by the building width ($\mathbf{W}$) to arrive at an average linear load along the ceiling plane. This double sloped diaphragm edge load is illustrated in Figure 5.11 by ($\mathbf{w_{windgable}}$). Thus the total windward roof/ceiling diaphragm edge load along the ceiling plane is:

$$\mathbf{w_{windtotrf}} = \mathbf{w_{windwallrf}} + \mathbf{w_{windgable}} \qquad \text{plf}$$

Wind Load to the Triangular Portion of the Leeward Gable

The leeward gable has negative wind pressure (suction) on the entire elevation, which does not vary with height. This is illustrated in Figure 5.11 by the uniform size of the wind pressure triangle against the end wall. Thus, the entire triangle of wall above the ceiling plane acts on the roof/ceiling diaphragm.
The area of the entire gable triangle × the negative wind pressure and then divided by the building width ($\mathbf{W}$) to arrive at an average linear load along the ceiling plane. This triangular diaphragm edge load is illustrated in Figure 5.11 by ($\mathbf{w_{leegable}}$). Thus the total leeward roof/ceiling diaphragm edge load along the ceiling plane is:

$$\mathbf{w_{leetotrf}} = \mathbf{w_{leewallrf}} + \mathbf{w_{leegable}} \qquad \text{plf}$$

Total Roof/Ceiling Diaphragm In-Plane Wind Load

The total wind load in plf applied to the ceiling/roof diaphragm in the plane of the ceiling is:

$$\mathbf{W_{RF}} = \mathbf{w_{windtotrf}} - \mathbf{w_{leetotrf}} \qquad \text{plf}$$

First Floor Diaphragm In-Plane Wind Load

Wind Direction: Parallel to the Ridge of a gable roof.

Again, the windward end wall receives positive wind pressure and the leeward wall receives negative wind pressure (suction). The wall studs bend vertically in their plane and transfer a reaction to the bottom plate of the structurally sheathed stud wall.

Figure 5.12 illustrates that the first floor diaphragm receives a tributary width of wind pressure on the windward end wall equal to the lower portion of the wall height, defined by the equation shown in Figure 5.5. This load to the first floor diaphragm becomes a lineal load ($\mathbf{w_{windwall}}$) in the plane of the floor diaphragm along the windward edge. The wind transfer occurs similarly on the leeward side of the floor diaphragm, producing a lineal load ($\mathbf{w_{leewall}}$) in the plane of the diaphragm along its leeward edge. The total wind load ($\mathbf{W_{F1}}$) to the floor diaphragm is then the sum of the windward and leeward lineal in-plane loads plus the roof diaphragm load:

$$\mathbf{W_{F1}} = \left(\mathbf{w_{windwall}} - \mathbf{w_{leewall}}\right) + \mathbf{W_{RF}} \quad (\text{plf})$$

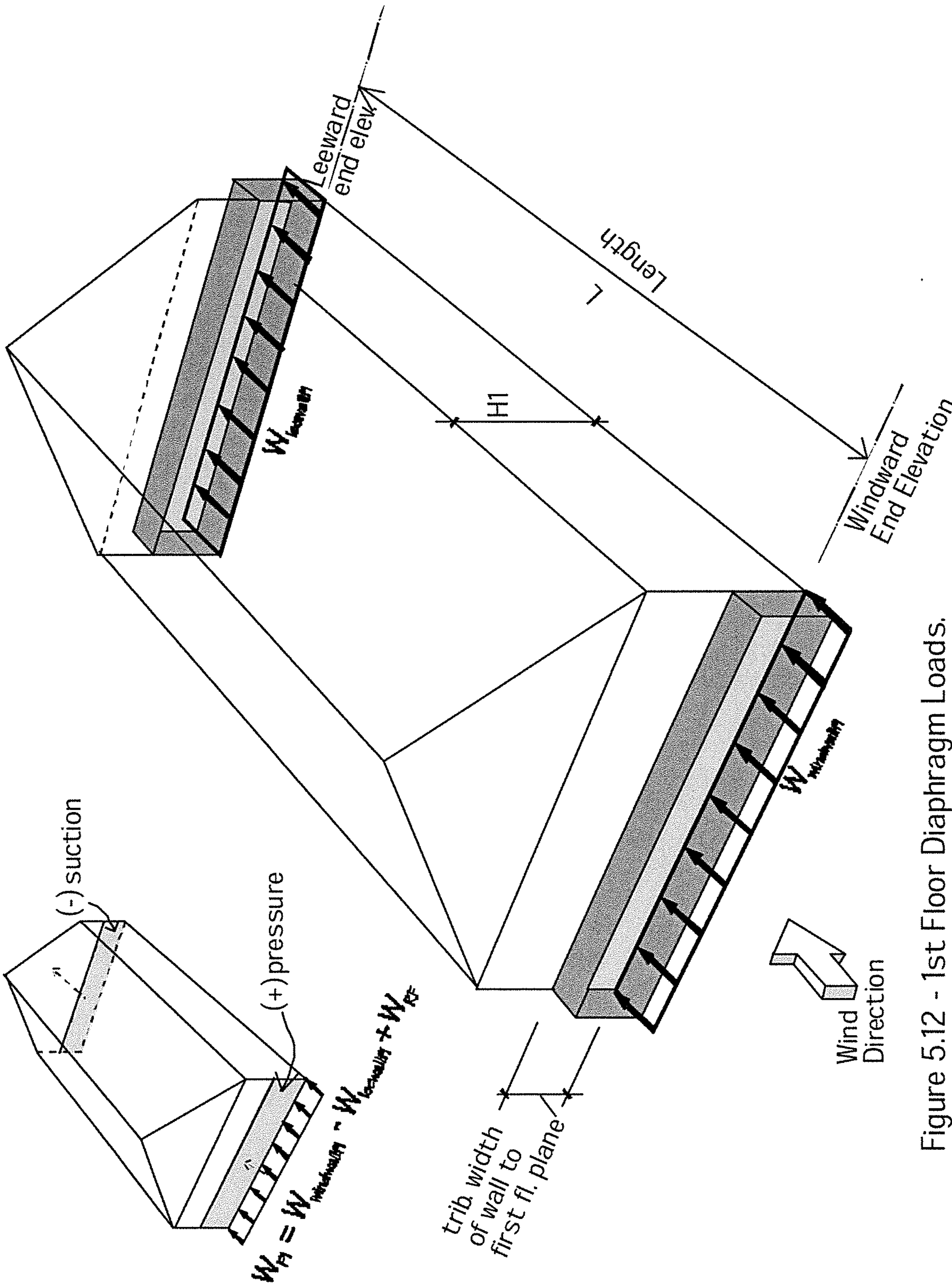


Figure 5.12 - 1st Floor Diaphragm Loads. Wind Parallel to Long Dimension "L". One Story.

Roof Diaphragm Behavior

Wind Direction: Parallel to the ridge of a gable roof.

Figure 5.13 illustrates in perspective the structural behavior of a gable roof diaphragm, the assumed model, the transfer of force between roof plane and shear wall and the uplift potential of shear walls. This illustration is very similar to Figure 5.6, except it is for wind parallel to the roof ridge. Figure 5.13 will permit a clarification of the force arrows related to reactions and shear. Also, the nomenclature for wind parallel to the ridge is somewhat different from that for wind perpendicular to the ridge. A thorough study of Figure 5.13 will now be presented.

The wind direction as shown is from left to right and is perpendicular to the surface of the long walls. The windward wall receives positive wind pressure that varies above a height of 15 feet as illustrated in Figure 5.11. The leeward wall experiences negative pressure that does not vary with height. The wall and gable of both windward and leeward walls produce a resultant linear load in the plane of the ceiling. Figure 5.11 designated the sum of these ceiling/roof diaphragm loads as $\mathbf{W_{RF}}$ in lbs. per foot of building width ($\mathbf{W}$).

The reaction ($\mathbf{R_{RFII}}$) at each end of the diaphragm span is a force equal to the maximum shear ($\mathbf{V_{RFII}}$) in pounds as shown by the shear diagram. Conversion of the shear force to a unit shear ($\mathbf{v_{RF\,II}}$) in plf along the diaphragm edge is easily performed by the equations shown in Figure 5.13 and shown below:

Roof/Ceiling Diaphragm Reaction: $\mathbf{R_{RFII}} = \mathbf{V_{RFII}} = \mathbf{W_{RF}} \times$ **W/2**

Roof Diaphragm shear: $\mathbf{v_{RFII}} = \mathbf{R_{RFII}} \div \mathbf{L}$ plf

It should now be clear that all the one-sided arrows in Figure 5.13 represent shear along the entire length of the diaphragm. This edge shear ($\mathbf{v_{RF\,II}}$) at the roof/Ceiling diaphragm is the important unit force for which the diaphragm is designed. It should be noted that for wind parallel to the ridge the length of the diaphragm will produce a rather small ($\mathbf{v_{RFII}}$) shear, compared to the shear ($\mathbf{v_{RF\perp}}$) for wind perpendicular to the ridge. This is why shear parallel to the ridge is generally only a check, and rarely becomes the shear required for strength. Also, note that the structural panels, for this direction of wind, have their joints continuous and parallel to the wind. This panel arrangement has a smaller allowable shear capacity than when the

Important Connectins
1. Web panel to panel
2. Web panel to shear wall
3. Flange to web
4. Flange splices
5. Foundation Anchorage

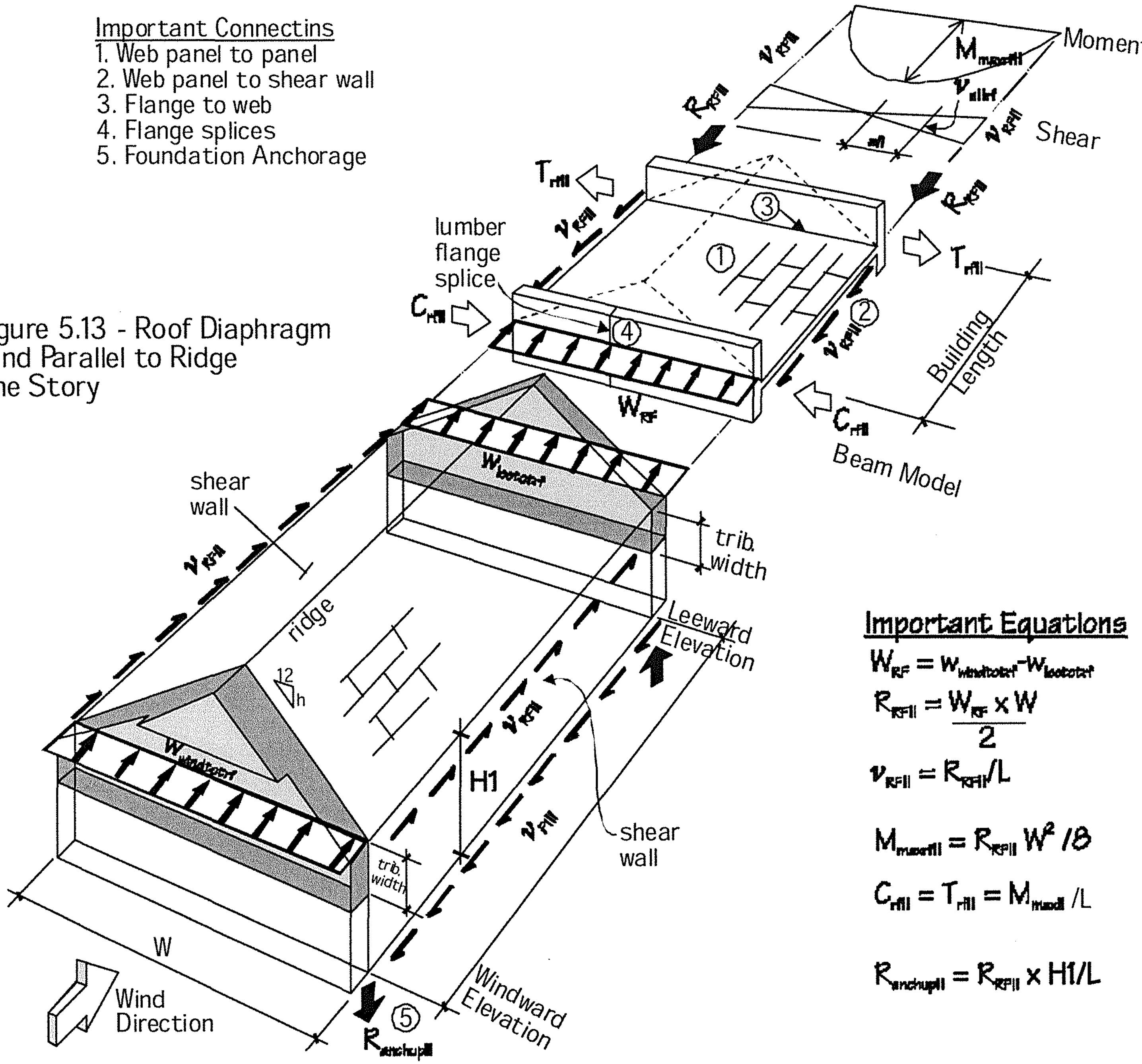


Figure 5.13 - Roof Diaphragm
Wind Parallel to Ridge
One Story

Important Equations

$$W_{RF} = w_{windtotal} - w_{leetotal}$$

$$R_{RFll} = \frac{W_{RF} \times W}{2}$$

$$v_{RFll} = R_{RFll}/L$$

$$M_{maxfll} = R_{RFll}\, W^2/8$$

$$C_{rfll} = T_{rfll} = M_{maxll}/L$$

$$R_{anchupll} = R_{RFll} \times H1/L$$

joints are staggered; however, this is of little consequence since the required shear (v_{RFII}) is so much smaller as well.

First Floor Diaphragm Behavior

Wind Direction: Parallel to the ridge of a gable roof

Figure 5.12 illustrates the first floor diaphragm load W_{F1}, but again requires further explanation regarding how the in-plane diaphragm force is determined. Figure 5.14 is an exploded perspective view of the same one-story residence with a gable roof found in Figure 5.13. The diaphragms are separated from the shear wall and the foundation.

Again, the windward wall receives positive wind pressure and the leeward wall receives negative wind pressure (suction).

The first floor diaphragm receives a tributary width of positive pressure on the windward end wall and a negative pressure on the leeward end wall equal to the lower portion of the wall height, defined by the equation shown in Figure 5.5. The sum of these two loads to the first floor diaphragm becomes a lineal load (W_{F1}) in the plane of the floor diaphragm along the windward edge:

$$W_{F1} = (w_{windwall} - w_{leewall}) + W_{RF} \quad (\text{plf})$$

The gable roof/ceiling diaphragm and first floor diaphragm span between the exterior shear walls; however, only one shear wall is shown in Figure 5.14 for clarity. The roof diaphragm must transfer its reaction (R_{RFII}) in shear ($v_{RF\,II}$) to the shear wall. The equations to calculate the reaction and shear at the roof/ceiling diaphragm are shown above under the discussion of the part F-roof diaphragm behavior.

Note the single-sided arrowhead symbols for the diaphragm shear in Figure 5.14, and in turn the equal and opposite repeat of these arrows to illustrate the transfer of (v_{RFII}) to the shear wall. This shear in plf must be transferred over the entire depth ($H1$) of the shear wall. This is the function of the structural sheathing panels. This topic will be discussed further on in the shear wall design part of the process.

The bottom of the shear wall, which rests on the first floor diaphragm, must be attached to the floor with a connection method sufficient to transfer the shear (v_{RFII}) into the floor diaphragm. Simultaneously, the first floor

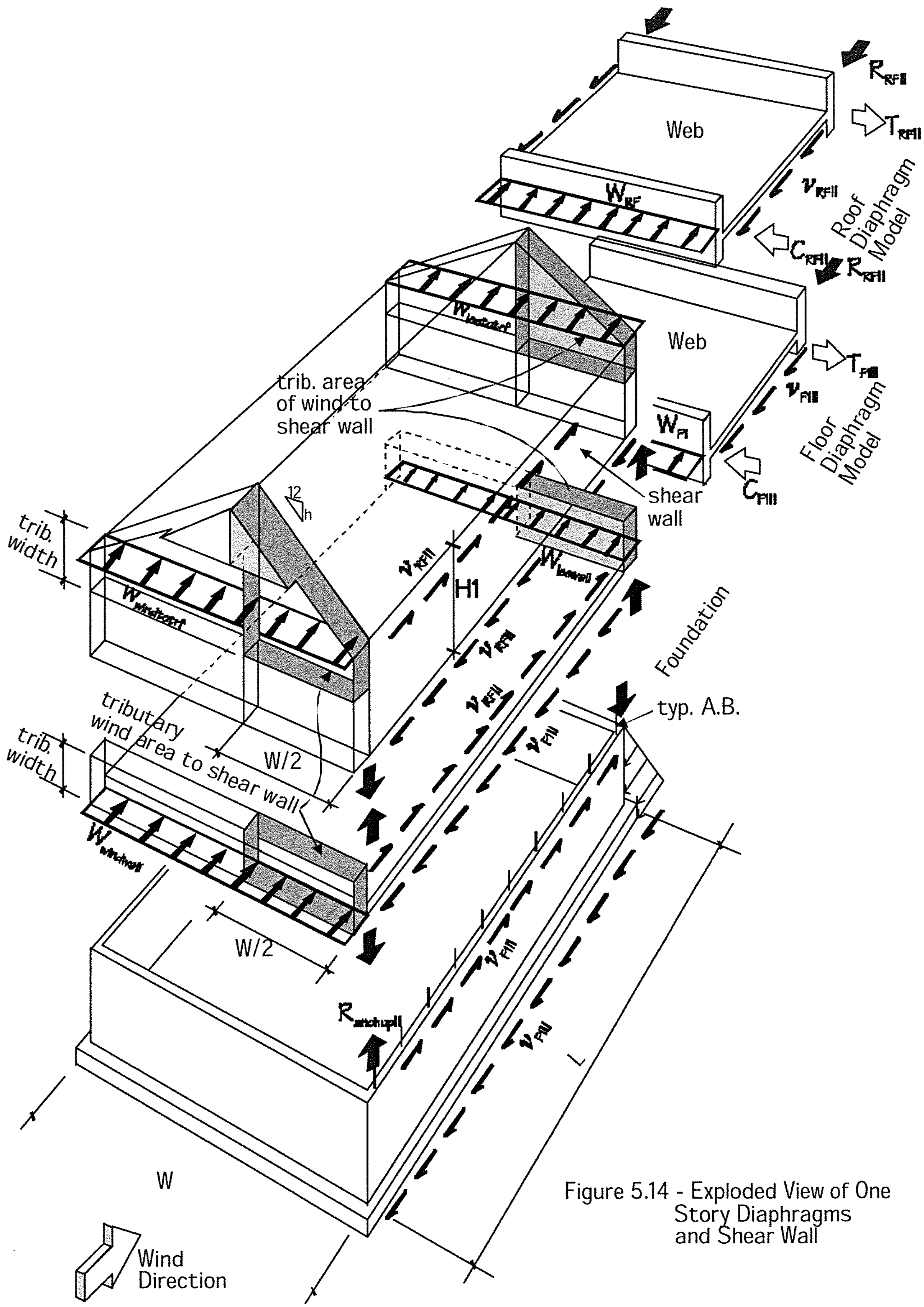


Figure 5.14 - Exploded View of One Story Diaphragms and Shear Wall

diaphragm receives the uniform wind load (F_{F1}) in its plane. Thus, the first floor diaphragm reaction and unit shear becomes by equation:

First Floor Diaphragm Reaction: $R_{F1II} = W_{F1} \times W/2$ lbs.

First Floor Diaphragm shear: $v_{F1II} = R_{F1II} \div L$ plf

The shear (v_{F1II}) is not only used to check the diaphragm, but must transfer the shear into the foundation by the use of the anchor bolts, already there to resist direct uplift. It should be clear from the elaborate discussion above that the shear at the first floor diaphragm, and the resulting shear transfer to the foundation, is significantly smaller than ($v_{RF\perp}$).

This shear wall will be sheathed with structural panels and anchored to the foundation, to resist sliding in shear and overturning, although for such a long shear wall as illustrated, overturning is unlikely.

8. Determine the maximum shear (v) in (plf) for each diaphragm section by the following equations repeated from step #5:

Wind Direction: Parallel to ridge of gable roof:

Roof/Ceiling Diaphragm Reaction: $R_{RFII} = V_{RFII} = W_{RF} \times W/2$

Roof Diaphragm shear: $v_{RFII} = R_{RFII} \div L$ plf

First Floor Diaphragm Reaction: $R_{F1II} = W_{F1} \times W/2$ lbs.

First Floor Diaphragm shear: $v_{F1II} = R_{F1II} \div L$ plf

a. Repeat step 6, using discussion from step 7.

9. Select **Shear Wall** locations for each roof diaphragm section. This is done visually, based on the orientation of the wind with respect to the long side or the gable roof ridge. Since diaphragms were selected, shear walls are at the ends of each diaphragm.

10. Calculate shear wall reactions from equations in Step #5 using diaphragm loads determined in step #4.

Wind Direction: *Perpendicular* to long dimension of flat roof diaphragm or *Perpendicular* to ridge of gable roof diaphragm.

Roof/Ceiling Diaphragm Reaction to shear wall:

$$\mathbf{R_{RF\perp}} = \mathbf{F_{RF}} \times \mathbf{L/2} \qquad \text{lbs.}$$

11. Develop Understanding of Structural Shear Wall Panel selection: panel grade, thickness and nail size and spacing from Table B.28 in Appendix B [5.15].

Use of Table B.28: Table B.28 for vertical shear walls functions much like Table B.27 for horizontal diaphragm shear plane design.

a. The Table is divided into three columns based on *"Framing Species"*. The most common species are listed in the three columns based on their specific gravity "G". For example, the largest shear strength for lateral loading of nails is found in species with a "G" > 0.49. These species are Southern Pine, Douglas Fir, or Douglas Fir-Larch. Any species with a "G" less than 0.49 will have a lower lateral nail load strength. Thus, for S-P-F (average grade for forest regions with the three species: spruce-pine-fir) with a "G" of 0.42, the basic shear values are multiplied by 0.82. This is the middle column of shears. Similarly for species with "G" less than 0.42, a factor of 0.65 is multiplied by the shear values in the left column. This has been done in the right column of shears. Refer to Table B.30 in Appendix B for species value of "G".

b. Each of the three major columns is further sub-divided by nail spacing: 6", 4", 3" and 2". It is best to use the largest nail spacing possible to avoid splitting of the panels and framing studs behind.

c. Table B.28 is divided into three horizontal rows based on structural *"panel grade"*. **APA Structural 1** grades have slightly higher allowable shear values, but are also the most expensive panels. The most commonly used sheathing for shear walls is **APA Rated Sheathing**, due to its general availability and lower cost. APA Rated Siding is the least costly, but also the least shear resistant, and not commonly used for shear walls.

d. Each of the three major rows is subdivided into more rows based on available panel thickness and appropriate nail choices.

Traditional Shear Wall Method

12. Traditional Shear Wall Method: Refer to Chapter 3 for an in-depth discussion of this method. Regardless of method, all shear wall design begins with the traditional method.

Wind Direction: *Perpendicular* to long dimension of flat roof diaphragm or *Perpendicular* to ridge of gable roof diaphragm.

a. Determine the location and height of openings in the shear walls. Determine the lengths of each full height structurally sheathed segment of the shear wall plane for each diaphragm. No segment's height to length ratio shall not exceed 3.5 [5.1] [5.2]. Thus for an 8'-0" tall wall, the minimum length = 2'-4". This author does not recommend using any wall segment less than 4'-0" long for the traditional method.

b. Calculate the total sum of the lengths of solid wall segments, those sheathed with structural panels, that extend from the bottom plate to the top of the double top plate. These segments occur between openings.

c. Calculate the shear (in plf) that the shear wall must transfer from the top of the wall down to the bottom of the wall.

Required shear to be transferred:

$$v_{RF\perp} = R_{RF\perp} \div \Sigma W_{solid} \quad \text{plf}$$

d. Determine stud wall framing lumber species and grade as described in step #3 and sample dimension lumber grade stamps in Figure 5.3.

e. Select panel grade, thickness, nail size and nail spacing for the required shear $v_{RF\perp}$. Based on the introduction to Table B.28, enter the Table with the known species column and the APA Rated Sheathing row. Generally start with the 3/8" thickness row and read the allowable shear values in plf within the species column until a value is reached that is *equal or larger than* the required shear $v_{RF\perp}$ calculated above in sub-step (c) above. Should no allowable shear satisfy the required shear, move to the next thickness, and so on until the desired shear is reached. Read up from that value to find the nail spacing.

NOTE: It is common today to include the shear wall capacity of the interior gypsum drywall. The UBC and the SBCC have a Table for the allowable shear capacity in plf for various interior paneling. This author has selected the smallest shear of 100 plf to include as an addition to the allowable capacity of the entire shear wall. Refer to Table B.29 in Appendix B [5.16] for shear values of various interior finishes.

f. Calculate the total length of solid full-height shear wall required using the allowable shear capacity selected, using the following equation:

$$W_{solidreq} = R_{RF\perp} \div v_{allow} \qquad \text{feet}$$

Compare with the available: $\Sigma W_{solid} \geq W_{solidreq}$

g. Calculate the uplift potential of the shear wall by the following equations:

Determine available superimposed dead load on the shear wall and its selfweight: Refer to Table A.11 in Appendix A for any floor framing dead load, and Table A.12 in Appendix A for the wall selfweight. Both of these Tables were explained in Chapter 4 and will not be repeated here. Refer to Figure 5.15, Figure 3.9a and the following equations:

$$W_{self} = wall_{DL} \times \text{length of shear wall segment} \qquad (\text{lbs.})$$

$$W_{floor} = DL_f \times \text{length of shear wall segment} \qquad (\text{lbs.})$$

$$W_{roof} = DL_{rf} \times \text{length of shear wall segment} \qquad (\text{lbs.})$$

$$W_{totalDL} = W_{self} + W_{floor} + W_{roof} \qquad (\text{lbs.})$$

$$R_{anchup\perp} = (R_{RF\perp} \times y - W_{totalDL} \times x/2)/x \quad (\text{lbs.})$$

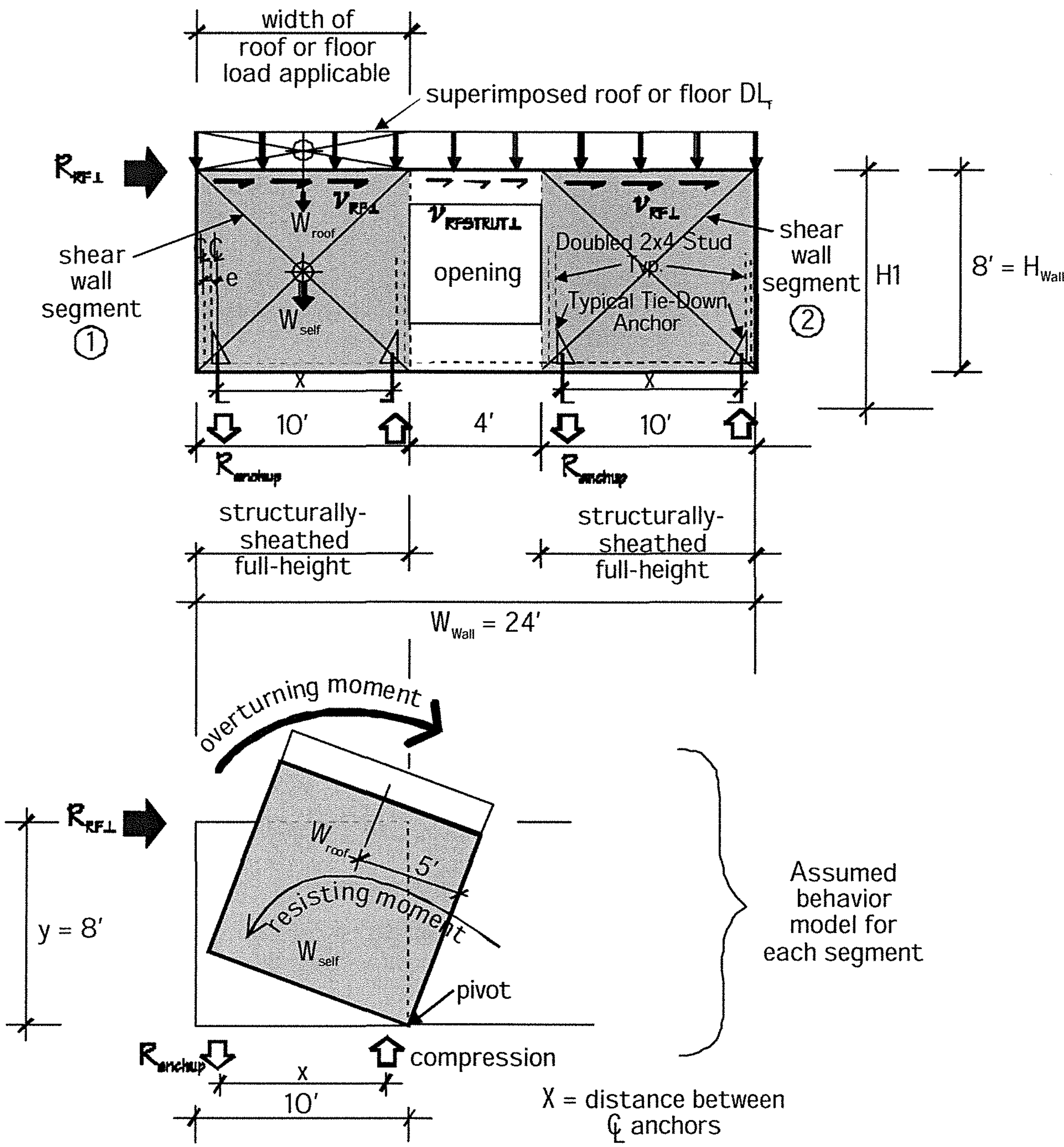


Figure 5.15 - Traditional Shear Wall

NOTES: 1. if answer is (-) no uplift occurs and **no** anchor is required. If answer is (+) select anchor from manufacturer's catalog, Similar to Figure 14 in the *WMM.*
2. Distance "x" is between centerlines of anchor tie-downs. This is between 6" to 8" less than $\mathbf{W_{wall}}$.

h. Calculate Sliding Resistance of Shear Walls

The **sliding Resistance of shear walls** is a function of the connection of the base of the shear wall to the supporting medium below the wall. The choice of connector and the magnitude of the shear to be resisted will determine the spacing between connectors. The shear the connectors must resist is $\mathbf{v_{ACT}}$, which is either $\mathbf{v_{RF}}$, $\mathbf{v_{F1}}$ or $\mathbf{v_{F2}}$ depending on the level of the shear wall and the sum of the length of the shear wall segments. The spacing of connectors below a shear wall or first floor diaphragm is determined by the following equation:

$$\mathbf{S_{SLIDEWALL}} = \text{Shear capacity of one fastener} \times 12 \div \mathbf{v_{ACT}}$$

1. Shear wall attached to floor framing.

Wood subfloor and TJI bandboard or Dimension Lumber bandboard is the common floor framing an exterior shear wall will attach to. Use of 10d Com nails, as the fastener, is typical. The nail is in single shear.

Assume **10d-Box nails,** and conservatively using **S-P-F lumber** with **"G" = 0.42** and a wind load factor of **1.33**, gives a shear allowable = **104 lbs.**

2. Shear wall attached directly to foundation of grouted concrete block or cast-in-place concrete, such as at a garage or patio enclosure. The connector type is likely an anchor bolt.

For penetration through bottom plate of wall into grouted block core with f'm = 1500 psi:

Assume 1/2" diameter anchor bolt = **1130 lbs.**

For penetration through the bottom plate of shear wall into cast-in-place concrete with f'c = 3000 psi:

Assume 1/2" diameter anchor bolt = **1660 lbs**.

3. Shear Connection of First Floor Diaphragm to a crawl space or basement foundation wall of grouted concrete block or cast-in-place concrete. A mudsill will be the typical intermediate member between the floor framing and the foundation.

The connector type is likely an anchor bolt. See item 2 above for anchor bolt capacity.

i. Calculate Drag Strut tension force over openings.

$$\mathbf{v_{RFSTRUT\perp}} = \mathbf{R_{RF\perp}} \div \mathbf{W} \qquad \text{plf}$$

$$\mathbf{T_{RFSTRUT\perp}} = \mathbf{v_{RFSTRUT\perp}} \times \text{length of opening} \quad \text{lbs.}$$

Select an appropriate metal connector to tension tie the drag strut to the shear wall at each side of the opening. See details from Figure 3.6c.

Wind Direction: Parallel to long dimension of flat roof diaphragm or Parallel to ridge of gable roof diaphragm.

a. Repeat step a. through i. by the traditional method for shear walls related to diaphragms in this wind direction.

Perforated Shear Wall Empirical Method

13. Perforated Shear Wall Empirical Method [5.17] [5.18]**:** Refer to Chapter 3 for an in-depth discussion of this method. Regardless of method, all shear wall design begins with the traditional method.

Wind Direction: *Perpendicular* to long dimension of flat roof diaphragm or *Perpendicular* to ridge of gable roof diaphragm.

The required total length of shear wall $\mathbf{W_{solidreq}}$ found by the Traditional Method will be merely multiplied by an increase factor $\mathbf{R_{\beta}}$ to account for openings and arrive at the minimum required length with anchors only at the ends of the elevation. Refer to Figure 5.16.

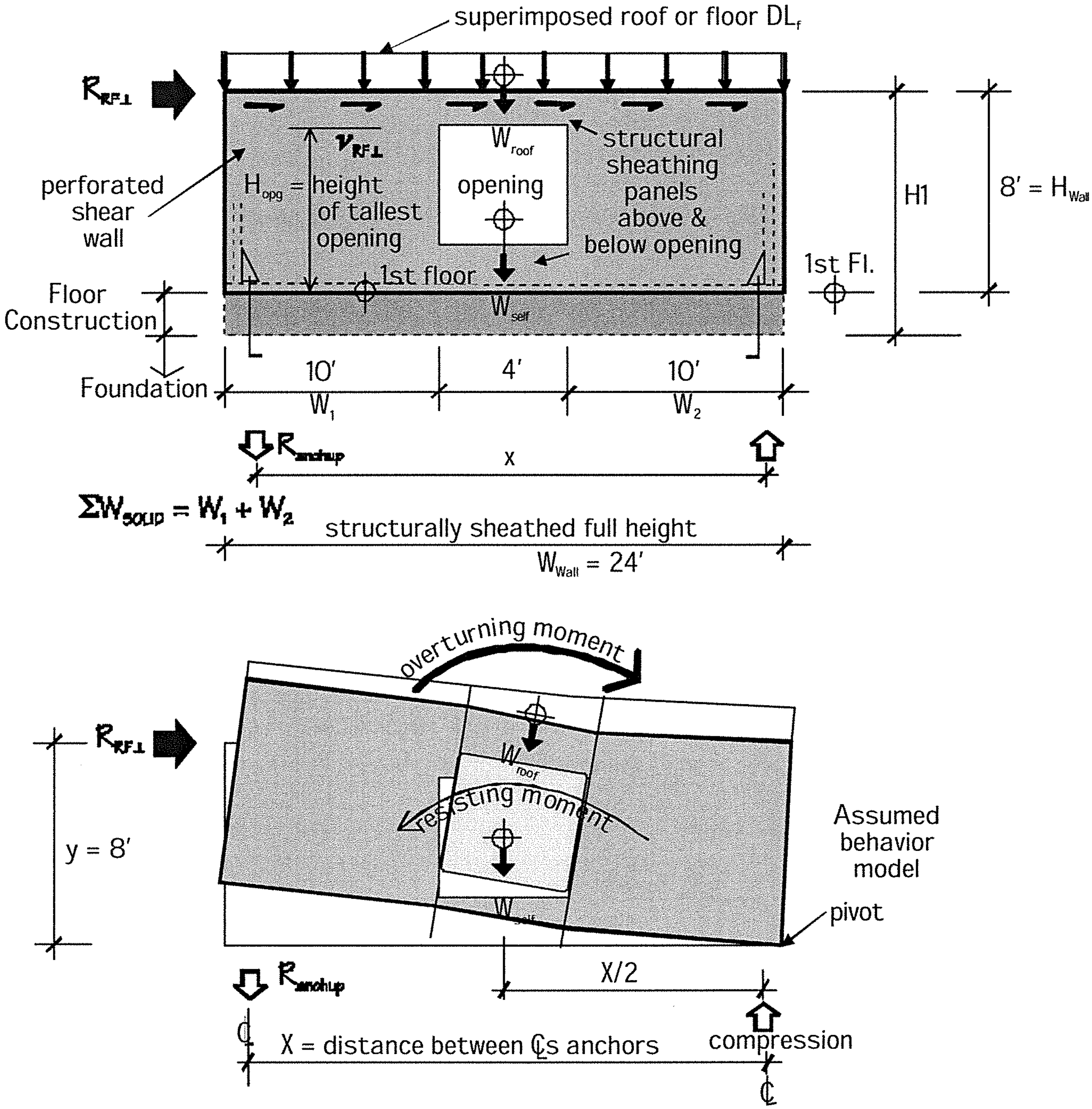


Figure 5.16 - Perforated Shear Wall

NOTES: 1. The entire wall must be sheathed with structural panels, including above and below openings.

2. This technique permits **H/3.5** as the minimum width of shear wall segment. Thus, for 8 foot high walls, the minimum width is 2'-4", while for a 10 foot wall height the minimum wall width is 2'-11".

a. Determine the location and height of openings in the shear walls of each diaphragm.

b. Calculate the total sum of the lengths of solid wall segments, those sheathed with structural panels, that extend from the bottom plate to the top of the double top plate across the total length of the elevation $\mathbf{W_{wall}}$.

$$\beta_1 = \Sigma \mathbf{W_{solid}} / \mathbf{W_{wall}}$$

c. Calculate the height of the tallest opening $\mathbf{H_{OPG}}$ and ratio it to the height of the wall **H**.

$$\text{Ratio} = \mathbf{H_{OPG}} / \mathbf{H}$$

Since most openings are 6'-8" tall and most walls are 8 foot tall, the ratio is usually **5H/6** or 0.833.

d. Calculate the length increase factor, which has been determined empirically to be:

$$R_\beta = \frac{1}{\dfrac{(1 - \beta_1)}{3 \text{ x} \left[\dfrac{H_{OPG}}{H_{wall}}\right]} + \beta_1}$$

The $\mathbf{R_?}$ factor indirectly reduces the allowable shear in plf to account for increased flexibility due to the openings in the wall. This is similar to increasing the length of total shear wall required, which the $\mathbf{R_?}$ factor accomplishes.

e. Calculate the minimum required length of perforated shear wall from the following equation:

$$W_{perf} = W_{solidreq} \times ?_?$$

Compare: $W_{perf} \leq W_{wall}$

Should the minimum required perforated wall length exceed the actual length of the elevation, the traditional method should be selected, or select a sheathing and nail arrangement with greater strength to reduce the traditional total shear wall length.

f. Repeat steps (a) through (e) for all shear walls associated with each diaphragm for a wind direction perpendicular to the ridge of the gable roof.

Wind Direction: Parallel to long dimension of floor diaphragm or Parallel to ridge of gable roof diaphragm.

Repeat Steps #9 through #12 for all shear walls associated with diaphragms receiving wind as described in the sentence above.

a. Procedure is the same but subscripting uses the parallel symbol (II).

Stability of Gable End Walls

14. Determine gable end wall stability. A one-story residence will provide this potential stability at the ceiling plane. Either the ceiling plane is capable of providing the stability or diagonal wood struts must provide the stability. Refer to Tables B.17 through B.20 in Appendix B for windward, leeward and total wind loads in plf. The wind Direction is always parallel to the ridge of the gable roof, and generally the windward and leeward wind pressures will be treated separately.

Stability Medium: Ceiling Diaphragm:

Windward Ceiling Diaphragm Reaction: $R_{CWIN} = W_{CWIN} \times W/2$

Ceiling Diaphragm shear: $v_{CWIN} = R_{CWIN} \div L_{ACT}$ plf

Leeward Ceiling Diaphragm Reaction: $R_{CLEE} = W_{CLEE} \times W/2$

Ceiling Diaphragm shear: $\nu_{CLEE} = R_{CLEE} \div L_{AVAIL}$
plf

Stability Medium: Diagonal braces:

Windward Horizontal Force per brace:

$$R_{CWINBR} = W_{CWIN} \times W/\text{no. of braces}+1$$

Brace Force at 45°: $F_{45} = R_{CWINBR} \times 1.414$

Leeward Horizontal Force per brace:

$$R_{CLEEBR} = W_{CLEE} \times W/\text{no. of braces}+1$$

Brace Force at 45°: $F_{45} = R_{CLEEBR} \times 1.414$

15. Design critical connections of diaphragm, diaphragm to shear wall and shear wall to foundation to resist sliding and overturning. Garage door framing should be designed as well.

General Design Procedure - Short Version

This version is for the experienced user

1. Determine the geographic location for the residence. Use the Wind Speed map found in Figure 1.1. Regions where a 70-MPH wind speed is shown should be increased to 80 MPH.

2. Establish the following parameters for a review of diaphragm selection and their associated shear walls for the residence:

   A. **Wind Direction**: **East/West Direction**: Perpendicular to the ridge of the house gable; parallel to the ridge of the garage gable

      a. Potential Shear Wall Locations and Diaphragm Proportions

      b. Roof and Floor Framing Components and wood Species:

   B. **Wind Direction**: **North/South Direction**: Parallel to the ridge of the house gable; perpendicular to the ridge of the garage gable

Repeat a. and b. above

3. Select structural panel thickness and grade for the roof and floor diaphragms based on gravity dead and live load. . Tables B.25 and B.26 in the Appendix B should be reviewed.

4. Select the appropriate roof and floor diaphragm force (plf) from Tables B.1 through B.4 in Appendix B.

A. **Wind Direction**: **East/West Direction**:

a. Roof diaphragms either perpendicular or parallel to ridge

B. **Wind Direction**: **North/South Direction**:

a. Roof diaphragms either perpendicular or parallel to ridge

Figure 5.5 illustrates the tributary areas of wind pressure and suction that determines the diaphragm force at the roof/ceiling plane and at the first floor plane for Tables B.1 to B.4 in Appendix B.

5. Determine the maximum shear (ν) in (plf) for each diaphragm section by the following equations:

A. **Wind Direction**: **East/West Direction**: Equations shown below are for wind direction perpendicular to the ridge:

Roof/Ceiling Diaphragm Reaction: $\mathbf{R_{RF\perp} = F_{RF} \times L/2}$ lbs.

Roof Diaphragm shear: $\boldsymbol{\nu_{RF\perp} = R_{RF\perp} \div W}$ plf

First Floor Diaphragm Reaction: $\mathbf{R_{F1\perp} = F_{F1} \times L/2}$ lbs.

First Floor Diaphragm shear: $\boldsymbol{\nu_{F1\perp} = R_{F1\perp} \div W}$ plf

B. **Wind Direction:** **North/South Direction:** Equations shown below are wind direction *Parallel* to ridge of gable roof:

Roof/Ceiling Diaphragm Reaction: $\mathbf{R_{RFII} = V_{RFII} = W_{RF} \times W/2}$

Roof Diaphragm shear: $\mathbf{v_{RFII} = R_{RFII} \div L}$ plf

First Floor Diaphragm Reaction: $\mathbf{R_{F1II} = F_{F1} \times L/2}$ lbs.

First Floor Diaphragm shear: $\mathbf{v_{F1II} = R_{F1II} \div L}$ plf

6. Select the appropriate APA rated structural sheathing panel grade and thickness, nail size and spacing, nail pattern across the diaphragm and blocking requirements from Table B.27 in Appendix B.

A. Wind Direction: East/West Direction: Roof

a. Select panel grade.

b. Select framing lumber species.

c. Select nominal framing width.

d. Compare the required Roof diaphragm shears ($\mathbf{v_{RF\perp}}$), ($\mathbf{v_{F1\perp}}$), ($\mathbf{v_{RFII}}$), ($\mathbf{v_{F1II}}$) in plf with the allowable shear values in plf listed.

e. Select nail size and spacing, and whether blocking is required or not.

f. Draw a sketch of panel nailing pattern.

B. Wind Direction: North/South Direction: Roof

Repeat steps a. through f. for the mutually perpendicular direction

A. Wind Direction: East/West Direction: First Floor

Repeat steps a. through f.

B. Wind Direction: North/South Direction: First Floor

Repeat steps a. through f. for the mutually perpendicular direction

7. Select shear wall locations for each roof diaphragm section. This is done visually, based on the orientation of the wind with respect to the long side or the gable roof ridge.

8. Calculate shear wall reactions from equations in step #5 using diaphragm loads determined in step #4.

   A. Wind Direction: East/West Direction

   a. All shear walls associated with diaphragms in this direction

   B. Wind Direction: North/South Direction:

   a. All shear walls associated with diaphragms in this direction

9. **Traditional Shear Wall Method:** Select Structural Panels for sheathing shear walls. Select Panel grade, thickness and nail size and spacing from Table B.28 in Appendix B.

   A. Wind Direction: East/West Direction

   a. Determine the location and height of openings in the shear walls of each diaphragm.

   b. Calculate the total sum of the lengths of solid wall segments, those sheathed with structural panels, that extend from the bottom plate to the top of the double top plate.

   c. Calculate the shear (in plf) that the shear wall must transfer from the top of the wall down to the bottom of the wall.

$$v_{RF\perp ACT} = R_{RF\perp} \div \Sigma W_{solid} \qquad \text{plf}$$

   d. Determine stud wall framing lumber species and grade as described in step #3 and sample dimension lumber grade stamps in Figure 5.3.

   e. Select panel grade, thickness, nail size and nail spacing for the required shear $v_{RF\perp}$.

   NOTE: It is common today to include the shear wall capacity of the interior gypsum drywall. Refer to Table B.29 in Appendix B for shear values of various interior finishes.

f. Calculate the total length of solid full-height shear wall required using the allowable shear capacity selected, using the following equation:

$$W_{solidreq} = R_{RF\perp} \div v_{allow} \qquad \text{feet}$$

Compare with the available: $\Sigma W_{solid} \geq W_{solidreq}$

B. Wind Direction: North/South Direction:

Repeat step a. through f. for shear walls in this direction.

10. Perforated Shear Wall Empirical Design Method:

A. Wind Direction: East/West Direction

a. Determine the location and height of openings of the shear walls associated with each diaphragm.

b. Calculate the total sum of the lengths of solid wall segments, those sheathed with structural panels, that extend from the bottom plate to the top of the double top plate across the total length of the elevation W_{wall}.

$$\beta_1 = \Sigma W_{solid} / W_{wall}$$

c. Calculate the height of the tallest opening H_{OPG} and ratio it to the height of the wall H.

$$\text{Ratio} = H_{OPG} / H$$

d. Calculate the length increase factor, which has been determined empirically to be:

$$R_{\beta} = \frac{1}{\dfrac{(1 - \beta_1)}{3 \times \left[\dfrac{H_{OPG}}{H_{wall}}\right]} + \beta_1}$$

e. Calculate the minimum required length of perforated shear wall from the following equation:

$$W_{perf} = W_{solidreq} \times ?_?$$

Compare: $W_{perf} \leq W_{wall}$

f. Determine available superimposed dead load on the shear wall and its selfweight: Refer to Table A.11 in Appendix A for any floor framing dead load, and Table A.12 in Appendix A for the wall selfweight.

$W_{self} = wall_{DL} \times$ length of shear wall segment (lbs.)

$W_{floor} = DL_f \times$ length of shear wall segment (lbs.)

$W_{roof} = DL_{rf} \times$ length of shear wall segment (lbs.)

$W_{totalDL} = W_{self} + W_{floor} + W_{roof}$ (lbs.)

g. Determine the uplift potential of the shear wall by the following equations:

$R_{anchup\perp} = (R_{RF\perp} \times y - W_{totalDL} \times x/2)/ x$ (lbs.)

h. Determine the required size and spacing of fastener to resist sliding of the shear wall under the full-height segments only. Use the following spacing S equation:

$S_{SLIDEWALL}$ = individual fastener shear capacity $\times 12 \div \nu_{ACT}$ in. o.c.

Where ν_{ACT} = shear in plf unique to each fully sheathed segment of a shear wall,

Where fastener selection is based on whether fastener penetrates wood, masonry or concrete.

i. Calculate Drag Strut tension force over openings.

$$v_{RFSTRUT\perp} = R_{RF\perp} \div W \quad \text{plf}$$

$$T_{RFSTRUT\perp} = v_{RFSTRUT\perp} \times \text{length of opening} \quad \text{lbs.}$$

Select an appropriate metal connector to tension tie the drag strut to the shear wall at each side of the opening. See details from Figure 3.6c.

Wind Direction: *Parallel* to long dimension of flat roof diaphragm or *Parallel* to ridge of gable roof diaphragm.

a. Repeat step a. through i. by the traditional method for shear walls related to diaphragms in this wind direction.

B. Wind Direction: **North/South Direction**:

Repeat step a. through h. for shear walls in the mutually perpendicular direction.

11. Determine gable end wall stability. Either the ceiling plane is capable of providing the stability or diagonal wood struts must provide the stability. Refer to Tables B.17 through B.20 in Appendix B for windward, leeward and total wind loads in plf.

Stability Medium: Ceiling Diaphragm:

Windward Ceiling Diaphragm Reaction: $R_{CWIN} = W_{CWIN} \times W/2$

Ceiling Diaphragm shear: $v_{CWIN} = R_{CWIN} \div L_{ACT}$ plf

Leeward Ceiling Diaphragm Reaction: $R_{CLEE} = W_{CLEE} \times W/2$

Ceiling Diaphragm shear: $v_{CLEE} = R_{CLEE} \div L_{AVAIL}$

plf

Stability Medium: Diagonal braces:

Windward Horizontal Force per brace:

$$R_{CWINBR} = W_{CWIN} \times W/\text{no. of braces+1}$$

Brace Force at 45°: $\mathbf{F_{45} = R_{CWINBR}} \times 1.414$

Leeward Horizontal Force per brace:

$$\mathbf{R_{CLEEBR} = W_{CLEE} \times W/\text{no. of braces+1}}$$

Brace Force at 45°: $\mathbf{F_{45} = R_{CLEEBR}} \times 1.414$

12. Design critical connections of diaphragm:
    - A. Tension chord splices for diaphragm tension force **T**.
    - B. Diaphragm to shear wall connection to transfer $\mathbf{v}$ in plf.
    - C. Garage Door Framing

    These items are beyond the scope of this Manual

Example - One Story - Structural Panel Sheathed Walls

Given:

Same information as for one story example #1 as found in Chapter 4.

Solution:

This solution follows the **General Design Procedure - Expanded Version** for a one-story residence outlined at the beginning of this Chapter.

1. The Residence is located in Urbana, IL. The ANSI 7-93 Wind Map indicates a 70-MPH wind speed. This is less than the minimum recommended by this author. the building department in Urbana has adopted the 1996 BOCA Code, which also advocates a 70-MPH wind speed; however they advocate and promote a 90-MPH wind speed

   Use 90-MPH wind speed

2. Establish the following parameters, referring to Figure 5.17 Roof Plan:

   A. **Wind Direction**: **East/West Direction**: Perpendicular to the ridge of the house gable; parallel to the ridge of the garage gable

   a. Potential Shear Wall Locations and Diaphragm Proportions

   (1) Consider the house gable roof **Diaphragm A** with north and south exterior **shear walls A and B** at the ends of its span **L** = 37'-4". The *wind direction is perpendicular to the ridge* of this gable diaphragm. The diaphragm proportions are 37.33/32 = 1.17< 4 as a recommended limit. This is a rather squarish diaphragm that will produce a small bending + shear deflection. There is a foundation directly under the shear walls, which is quite desirable for overturning resistance. There are no openings in this diaphragm.

   (2) Consider the small section of house gable roof that is a continuation of **Diaphragm A**. The diaphragm span **L** = 6'-8" and it has a width **W** = 20'-0". This small roof section will be referred to as **Diaphragm AA**, with **Shear Walls B and C** at each end of the span, but not continuous across the width of the

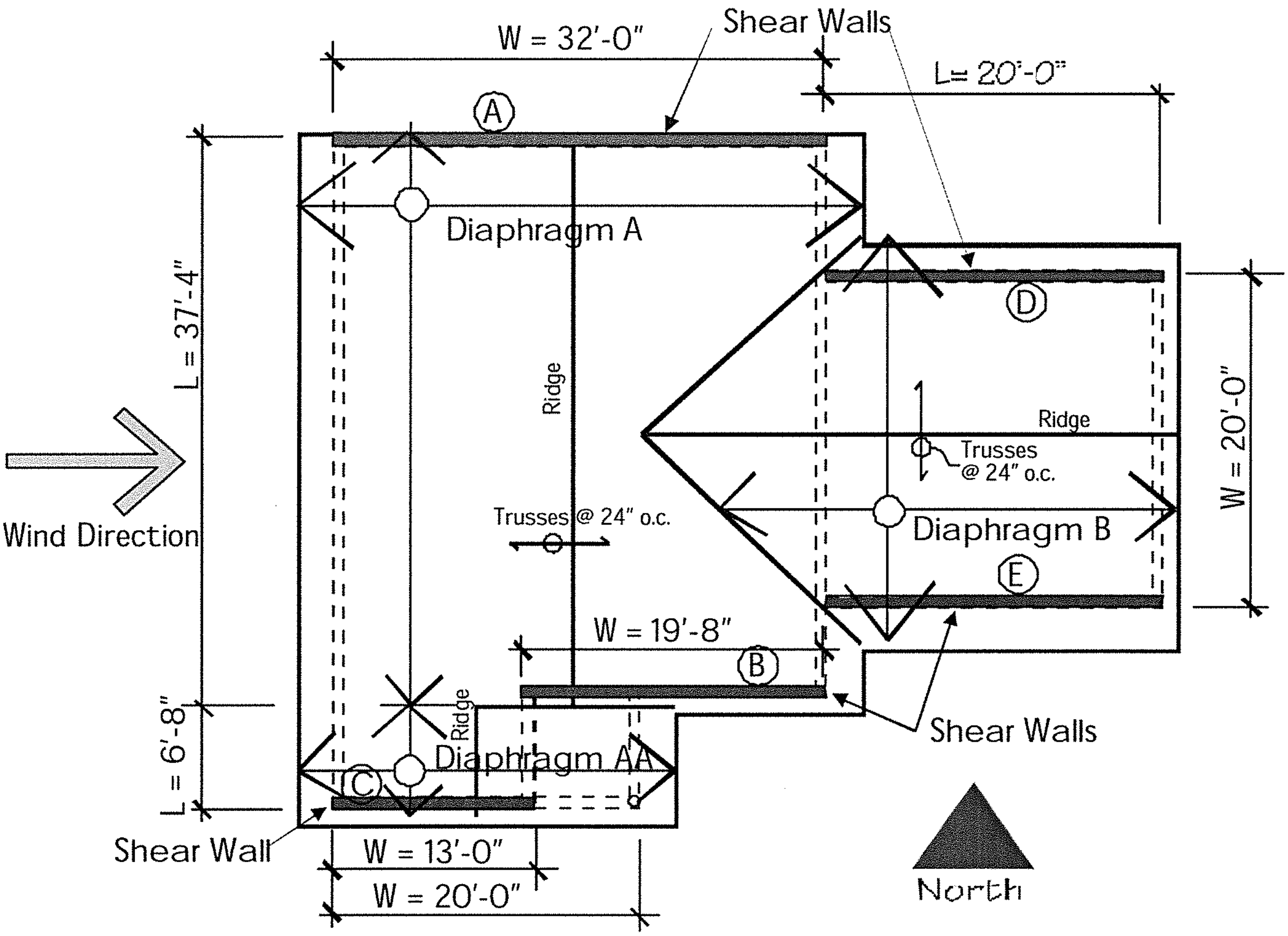


Figure 5.17 - East/West Direction for Roof Diaphragms and Shear Walls

diaphragm. Drag struts may be useful to transfer shear to the shear walls. Note that the interior **Shear Wall B** receives shear from two adjacent diaphragms.

(3) Consider the garage gable roof **Diaphragm B** with north and south exterior **Shear Walls D and E** at the ends of its truss span **W** = 20'-0". The *wind direction is parallel to the ridge* of this gable diaphragm. The diaphragm proportions are 20/20 = 1.0 < 4 as a recommended limit. This is a square diaphragm that will produce a small bending + shear deflection. There is a foundation directly under the shear walls, which is quite desirable for overturning resistance. There are no openings in

this diaphragm. Note that the east wall contains the garage door opening, which will require special consideration as a shear wall.

B. **Wind Direction**: **North/South Direction**: Parallel to the ridge of the house gable; perpendicular to the ridge of the garage gable

(1) Consider the house gable roof **Diaphragm C** shown in the roof plan view of Figure 5.18. The diaphragm spans between east and west exterior **Shear Walls G and F** of its span **W** = 32'-0". The *wind direction is parallel to the ridge* of this gable diaphragm. The diaphragm proportions are 44/32 = 1.4 < 4 as a recommended limit. This is a deeper diaphragm that it's span and will produce a very small bending + shear deflection. There is a foundation directly under the shear walls, which is quite desirable for overturning resistance. There are no openings in this diaphragm.

(2) Consider the garage gable roof **Diaphragm D** with east and west exterior/interior **Shear Walls H and G** at the ends of its span **L** = 20'-0". The *wind direction is perpendicular to the ridge* of this gable diaphragm. The diaphragm proportions are 20/20 = 1.0 < 4 as a recommended limit. This is a square diaphragm that will produce a small bending + shear deflection. There is a foundation directly under the shear walls, which is quite desirable for overturning resistance. There are no openings in this diaphragm. Note that **Shear Wall G** is mutually used as the shear wall for the **Diaphragm C**, besides **Diaphragm D**.

b. Framing components, species, lumber width and grade

(1) **Roof**

Pre-engineered **roof trusses** with pressed metal plates, composed of all 2x4 dimension lumber and spaced at 24" on center.

Top chord grade stamped: machine stress-rated:
Species = **Southern Pine**
Nominal Width = **2" (1-1/2" act.)**

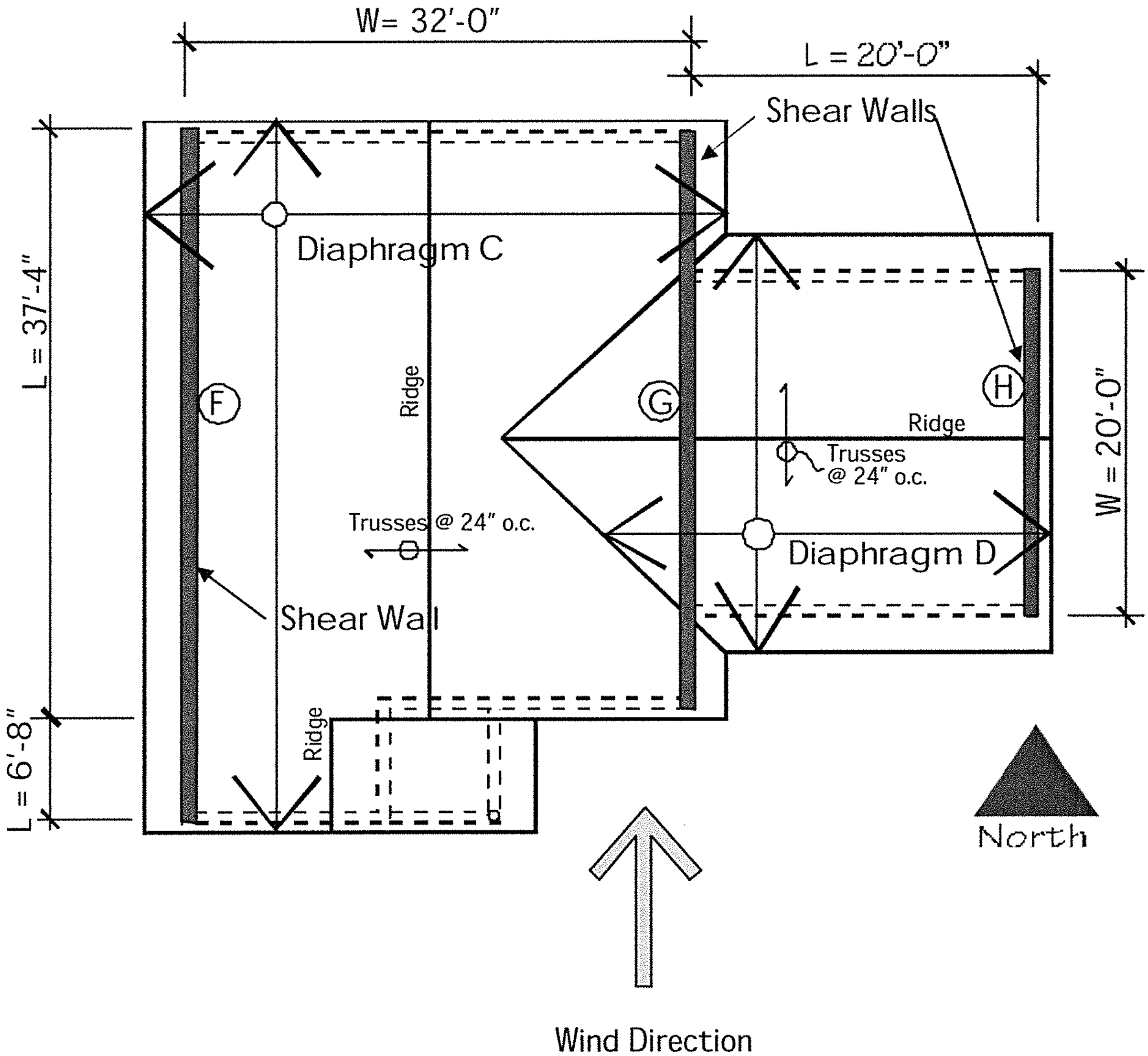


Figure 5.18 - North/South Direction for Roof Diaphragms and Shear Walls

(2) **First Floor**

Pre-engineered **solid web "I" joists** 9-1/2" deep by TJI or GPI2S, Spacing at 19.2" on center.

Species = equiv. **Southern Pine**
Nominal flange width: **2" (1-1/2" act.)**

3. Select Structural Panel for floor and roof based on gravity loads. Use Table B.25 and B.26 in Appendix B:

   a. **Roof** - use minimum live load = 30 psf.

      APA Rated Sheathing = **7/16"** thick
      Grade stamp: **Exposure 1**
      24/16 roof/floor ratio

   b. **Floor** - Subfloor: use minimum live load = 40 psf

      APA Rated Sheathing = **19/32"** thick
      Grade Stamp: **Exposure 1**
      32/16 roof/floor ratio

   Selections are conservatively chosen to minimize sag between framing below. Roof shingles appear to "oil can", meaning to show sag with moisture on roof or light snow layer, when structural panels are too thin.

4. Select roof and floor diaphragm force (plf) from Tables B.1 through B.4 and B.9 through B.12 in Appendix B.

   A. **Wind Direction**: **East/West Direction**:

   Diaphragm A: Perpendicular to the ridge of the house gable

   Enter Table B.4: Roof truss span **W** = 32ft
   Roof slope of 4 in 12
   Wind Speed = 90 MPH
   Load to the roof diaphragm: F_{RF} = **166 plf**
   Load to the floor diaphragm: F_{F1} = **309 plf**

   Diaphragm AA: Perpendicular to the ridge of the house gable

   Enter Table B.1: Roof truss span **W** = 20ft
   Roof slope of 4 in 12
   Wind Speed = 90 MPH
   Load to the roof diaphragm: F_{RF} = **105 plf**

   Diaphragm B: Parallel to the ridge of the garage gable

   Enter Table B.9: Roof truss span **W** = 20ft
   Roof slope of 4 in 12

Wind Speed = 90 MPH
Load to the roof diaphragm: W_{RF} = **130 plf**
Load to the floor diaphragm: W_{F1} = **245 plf**

B. Wind Direction: North/South Direction:

Diaphragm C: Parallel to the ridge of the house gable

Enter Table B.12: Roof truss span W = 32ft
Roof slope of 4 in 12
Wind Speed = 90 MPH
Load to the roof diaphragm: W_{RF} = **165 plf**
Load to the floor diaphragm: W_{F1} = **288 plf**

Diaphragm D: Perpendicular to the ridge of the garage gable

Enter Table B.1: Roof truss span = W = 20ft
Roof slope of 4 in 12
Wind Speed = 90 MPH
Load to the roof diaphragm: F_{RF} = **105 plf**

5. Determine the maximum Reaction R in (lbs.) and shear v in (plf) for each diaphragm section.

A. Wind Direction: East/West Direction:

Diaphragm A: Perpendicular to the ridge of the house gable

Refer to Figure 5.19 for a perspective illustration of the location of these values.

Roof/Ceiling Diaphragm Reaction: $R_{RF\perp} = F_{RF} \times L/2$ lbs.

$$R_{RF\perp} = 166 \times 37.33/2 = \mathbf{3098} \quad \textbf{lbs.}$$

Roof Diaphragm shear: $v_{RF\perp} = R_{RF\perp} \div W$ plf

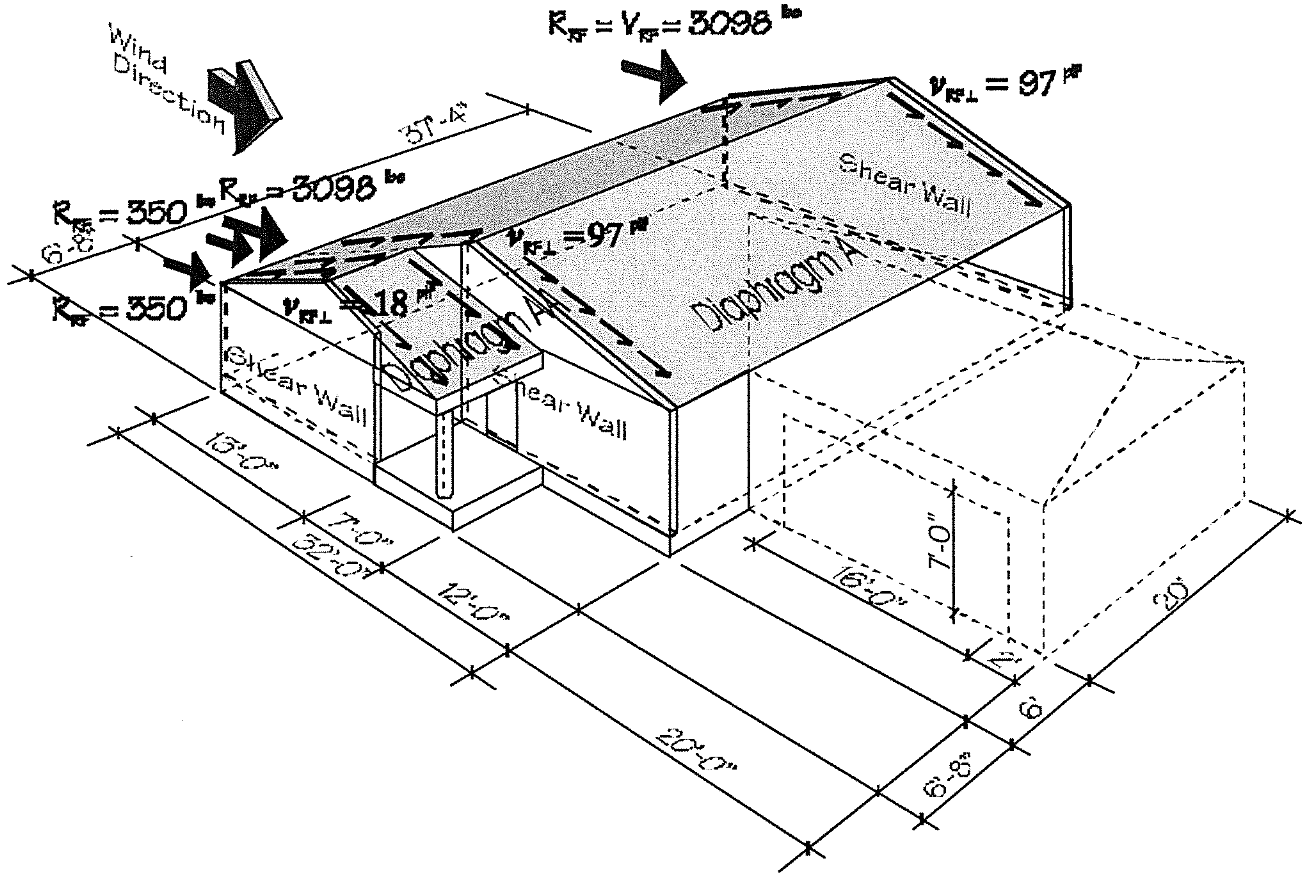


Figure 5.19 - Perspective of One Story Residence
Diaphragm A & AA Reactions and Shears

$$v_{RF\perp} = 3098 \div 32 = \mathbf{97\ \ plf}$$

First Floor Diaphragm Reaction: $\mathbf{R_{F1\perp}} = \mathbf{F_{F1}} \times \mathbf{L/2}$ lbs.

$$\mathbf{R_{F1\perp}} = 309 \times 44/2 = \mathbf{6798\ \ lbs.}$$

First Floor Diaphragm shear: $\mathbf{v_{F1\perp}} = \mathbf{R_{F1\perp}} \div \mathbf{W}$ plf

$$\mathbf{v_{F1\perp}} = 6798 \div 32 = \mathbf{212\ \ plf}$$

Diaphragm AA: Perpendicular to the ridge of the house gable

Refer to Figure 5.19 for a perspective illustration of the location of these values.

Roof/Ceiling Diaphragm Reaction: $\mathbf{R_{RF\perp} = F_{RF} \times L/2}$ lbs.

$$\mathbf{R_{RF\perp}} = 105 \times 6.67/2 = \mathbf{350 \ \ lbs.}$$

Roof Diaphragm shear: $\mathbf{v_{RF\perp} = R_{RF\perp} \div W}$ plf

$$\mathbf{v_{RF\perp}} = 350 \div 20 = \mathbf{18 \ \ plf}$$

Diaphragm B: Parallel to the ridge of the garage gable

Refer to Figure 5.20 for a perspective illustration of the location of these values.

Roof/Ceiling Diaphragm Reaction: $\mathbf{R_{RFII} = V_{RFII} = W_{RF} \times W/2}$

$$\mathbf{R_{RFII} = V_{RFII}} = 130 \times 20/2 = \mathbf{1300 \ \ lbs.}$$

Roof Diaphragm shear: $\mathbf{v_{RFII} = R_{RFII} \div L}$ plf

$$\mathbf{v_{RFII}} = 1300 \div 20 = \mathbf{65 \ \ plf}$$

B. **Wind Direction**: **North/South Direction**:

Diaphragm C: Parallel to the ridge of the house gable

Refer to Figure 5.21 for a perspective illustration of the location of these values.

Roof/Ceiling Diaphragm Reaction: $\mathbf{R_{RFII} = V_{RFII} = W_{RF} \times W/2}$

$$\mathbf{R_{RFII} = V_{RFII}} = 165 \times 32/2 = \mathbf{2640 \ \ lbs.}$$

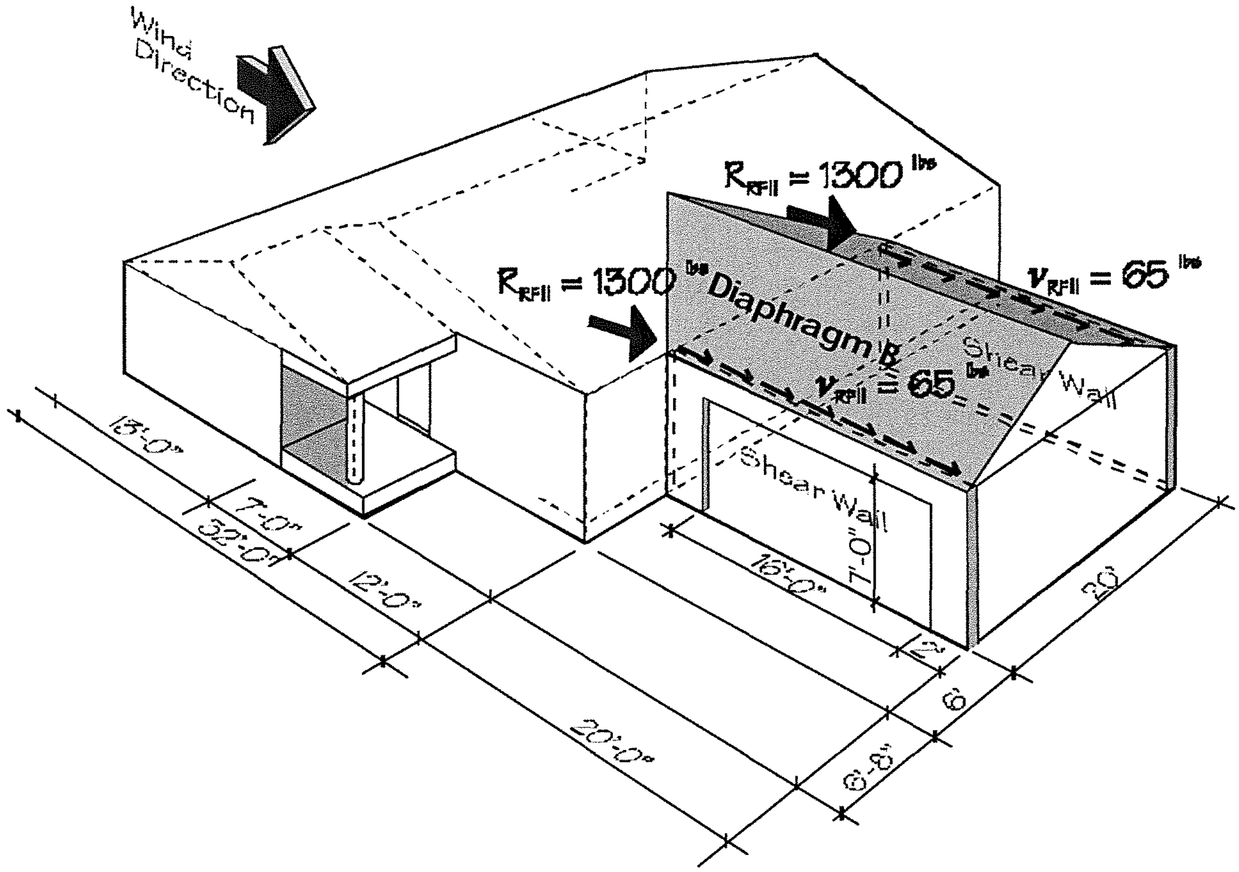


Figure 5.20 - Perspective of One Story Residence
Diaphragm B Reaction and Shear

Roof Diaphragm shear: $\mathbf{v_{RFII}} = \mathbf{R_{RFII}} \div \mathbf{L}$ plf

$\mathbf{v_{RFII}} = 2640 \div 37.33 = \mathbf{71}$ **plf**

First Floor Diaphragm Reaction: $\mathbf{R_{F1II}} = \mathbf{F_{F1}} \times \mathbf{W/2}$ lbs.

$\mathbf{R_{F1II}} = 288 \times 32/2 = \mathbf{4608}$ **lbs.**

First Floor Diaphragm shear: $\mathbf{v_{F1II}} = \mathbf{R_{F1II}} \div \mathbf{L}$ plf

$\mathbf{v_{F1II}} = 4608 \div 37.33 = \mathbf{123}$ **plf**

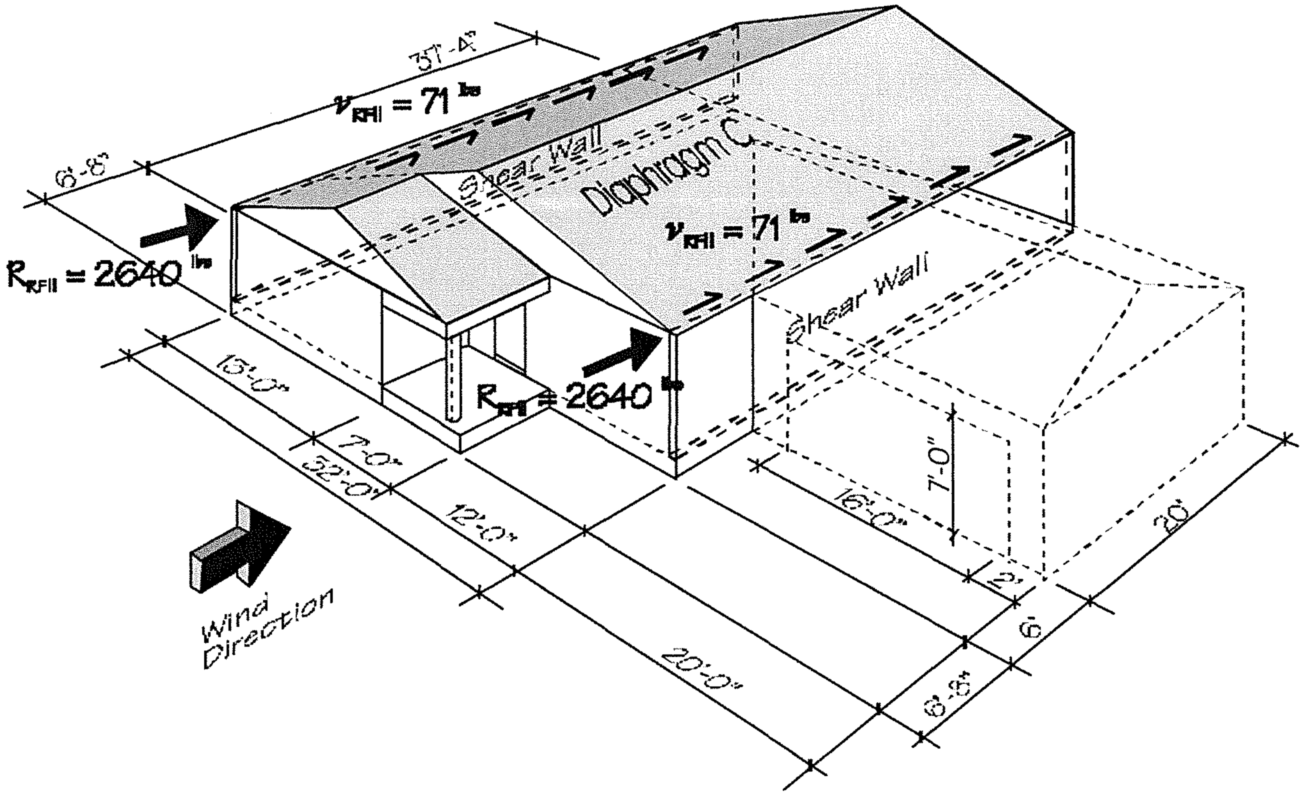


Figure 5.21 - Perspective of One Story Residence
Diaphragm C Reaction and Shear

Diaphragm D: Perpendicular to the ridge of the garage gable

Refer to Figure 5.22 for a perspective illustration of the location of these values.

Roof/Ceiling Diaphragm Reaction: $\mathbf{R_{RF\perp} = F_{RF} \times L/2}$
lbs.

$$R_{RF\perp} = 105 \times 20/2 = \mathbf{1050 \quad lbs.}$$

Roof Diaphragm shear: $\mathbf{v_{RF\perp} = R_{RF\perp} \div W}$ plf

$$v_{RF\perp} = 1050 \div 20 = \mathbf{53 \quad plf}$$

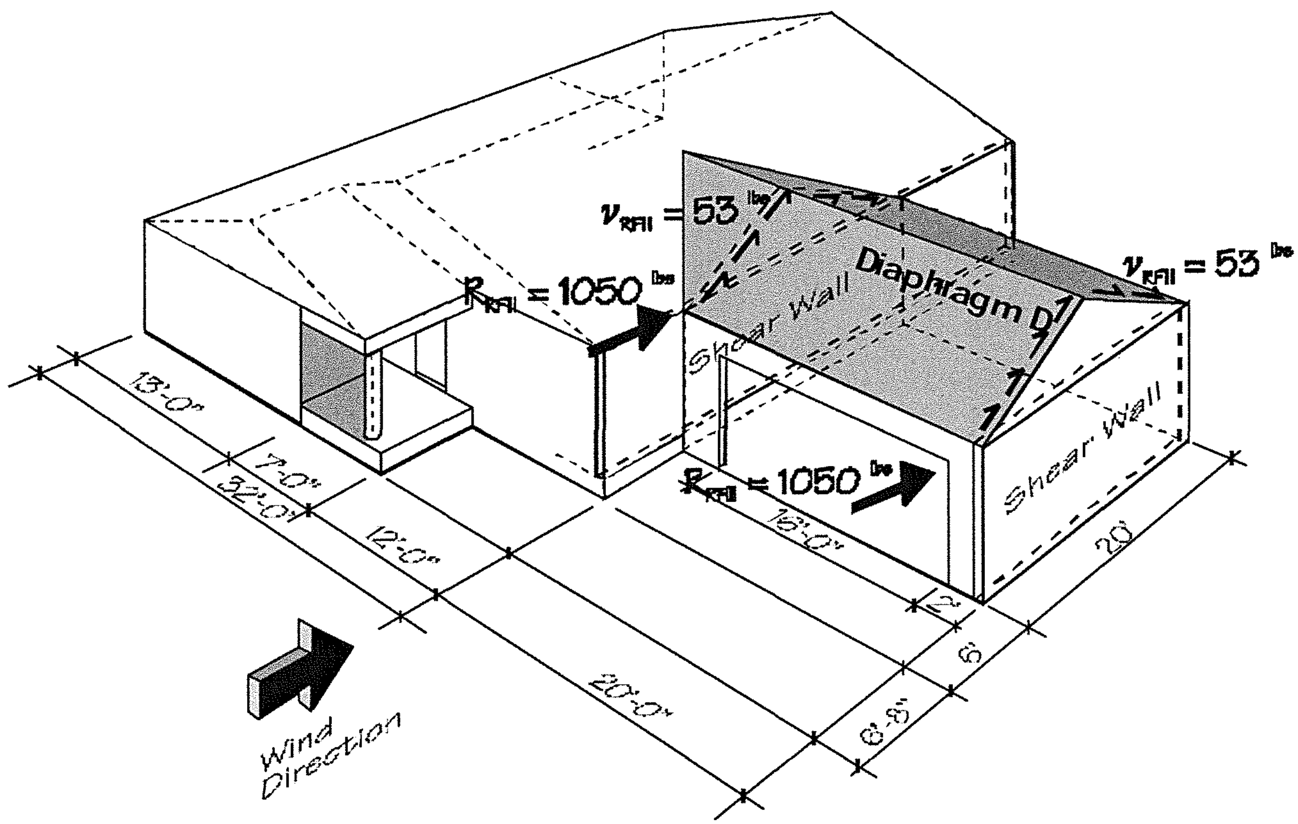


Figure 5.22 - Perspective of One Story Residence Diaphragm D Reaction and Shear

6. Select the appropriate APA (The Engineered Wood Association) rated structural sheathing panel grade and thickness, nail size and spacing, nail pattern across the diaphragm and blocking requirements from Table B.27 and Table B.30 in Appendix B.

A. **Wind Direction**: **East/West Direction**: **Roof**

Diaphragm A: Perpendicular to the ridge of the house gable

Refer to Figure 5.19 for a reference while working on gable roof **Diaphragm A** and **Diaphragm AA.**

Select specific gravity "**G**" from Table B.30. Enter Table B.27 with Specific Gravity "**G**" = 0.5 for the Southern Pine top chord of the roof trusses and a gravity load selection of 7/16" APA Rated Sheathing from step #3, plus nominal 2" wide truss top chords. Read across row to find an allowable shear just greater than the required shear $v_{RF\perp}$ = **97 plf**. The smallest, but larger, value is 170 plf for unblocked diaphragms. Read nail size and spacing requirements that accompanies this allowable shear.

Select: **7/16" APA Rated Sheathing**

v_{allow} = 170 plf > 97 plf required

Nails: 8d Com @ 6" at all Panel edges, **12" @ intermediate framing** members.

No Blocking Required

Diaphragm AA: Perpendicular to the ridge of the house gable

The required shear is $v_{RF\perp}$ = **18 plf**. The smallest, but larger, value is again 170 plf for unblocked diaphragms. Read nail size and spacing requirements that accompanies this allowable shear.

Select: **Same as Diaphragm A**

Diaphragm B: Parallel to the ridge of the garage gable

Refer to Figure 5.20 for a reference while working on gable roof **Diaphragm B.**

The required shear is $v_{RF\perp}$ = **65 plf**. The smallest, but larger, value is again 170 plf for unblocked diaphragms. Read nail size and spacing requirements that accompanies this allowable shear.

Select: **Same as Diaphragm A**

B. Wind Direction: **North/South Direction**: **Roof**

Diaphragm C: Parallel to the ridge of the house gable

Refer to Figure 5.21 for a reference while working on gable roof **Diaphragm C.**

The required shear is $v_{RF\perp}$ = **71 plf**. The smallest, but larger, value is again 170 plf for unblocked diaphragms. Read nail size and spacing requirements that accompanies this allowable shear.

Select: **Same as Diaphragm A**

Diaphragm D: Perpendicular to the ridge of the garage gable

Refer to Figure 5.22 for a reference while working on gable roof **Diaphragm D.**

The required shear is $\mathbf{v_{RF\perp}}$ = **53 plf**. The smallest, but larger, value is again 170 plf for unblocked diaphragms. Read nail size and spacing requirements that accompanies this allowable shear.

Select: **Same as Diaphragm A**

A. Wind Direction: **East/West Direction**: **First Floor**

Floor Diaphragm: Perpendicular to the ridge of the house gable

Refer to Figure 5.23 for a perspective that illustrates the reactions and shears on the **First Floor Diaphragm** below roof **Diaphragm A.**

Enter Table with Specific Gravity "**G**" = 0.5 (Table B.30 in Appendix B) for the equivalent Southern Pine top chord of the solid web floor "I" joists and a gravity load selection of 19/32" APA Rated Sheathing from step #3, plus nominal 2" wide joist flange. Read across row to find an allowable shear just greater than the required shear $\mathbf{v_{F1\perp}}$ = **212 plf** . The smallest, but larger, value is 215 plf for unblocked diaphragms. Read nail size and spacing requirements that accompanies this allowable shear.

Select: **19/32" APA Rated Sheathing**

$\mathbf{v_{allow}}$ = **215 plf > 212 plf required**

Nails: 10d Com @ 6" at all Panel edges, **12" @ intermediate framing** members.

No Blocking Required

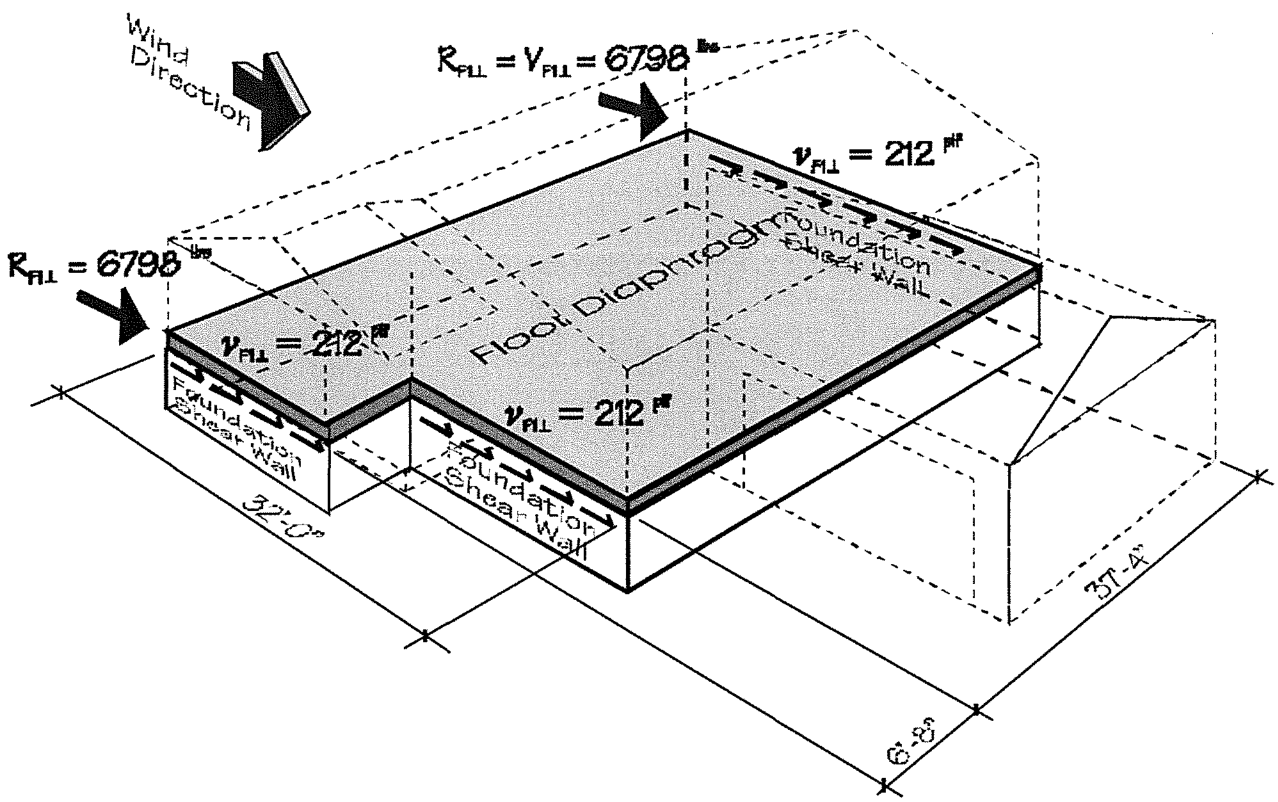


Figure 5.23 - Perspective of One Story Residence Floor Diaphragm Reaction and Shear

B. **Wind Direction**: **North/South Direction**: **First Floor**

First Floor Diaphragm: Parallel to the ridge of the house gable

Refer to Figure 5.24 for a perspective that illustrates the reactions and shears on the **First Floor Diaphragm** below roof **Diaphragm C.**

The required shear is $\mathbf{v_{F1||}}$ = **123 plf**. The smallest, but larger, value is again 215 plf for unblocked diaphragms. Read nail size and spacing requirements that accompanies this allowable shear.

Select: **Same as First Floor Diaphragm** in the East/West Direction.

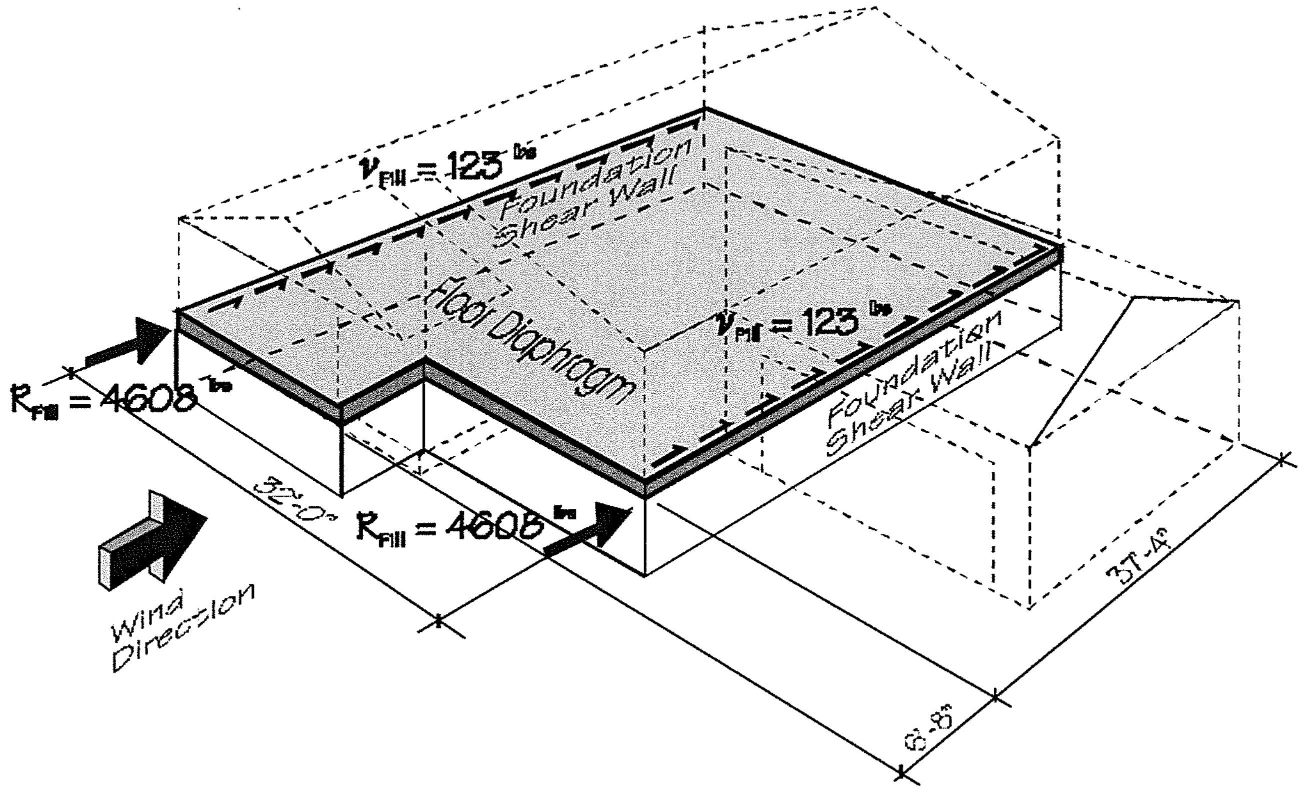


Figure 5.24 - Perspective of One Story Residence
Floor Diaphragm Reaction and Shear

7. Select shear wall locations for each roof diaphragm section. This is done visually, based on the orientation of the wind with respect to the long side or the gable roof ridge.

   Return to Figures 5.17 and 5.18 to review where shear walls were selected.

8. Calculate shear wall reactions from equations in step #5 using diaphragm loads determined in step #4. Refer to Figure 5.25 through 5.32 for each shear wall's reactions, shears, uplift and sliding information.

   A. Wind Direction: East/West Direction Diaphragm A and AA

   Shear wall A: $R_{RF\perp} = 166 \times 37.33/2 =$ **3098 lbs.**

Shear Wall B: $R_{RF\perp+\perp}$ = **350 + 3098 = 3448 lbs.**

Shear Wall C: $R_{RF\perp}$ = **350 lbs.**

Diaphragm B

Shear Wall D: R_{RFII} = 130 × 20/2 = **1300 lbs.**

Shear Wall E: R_{RFII} = 130 × 20/2 = **1300 lbs.**

B. Wind Direction: North/South Direction: Diaphragm C and D

Shear Wall F: R_{RFII} = 165 × 32/2 = **2640 lbs.**

Shear Wall G: $R_{RF11+\perp}$ = 2640 + 1050 = **3690 lbs.**

Shear Wall H: $R_{RF\perp}$ = 105 × 20/2 = **1050 lbs.**

9. **Traditional Shear Wall Design Method:** Select Structural Panels for sheathing shear walls. Select Panel grades, thickness and nail size and spacing from Table B.28 in Appendix B. Refer to Figure 5.25 through 5.32 for each shear wall's reactions, shears, uplift and sliding information.

A. Wind Direction: East/West Direction Diaphragm A and AA

a. Determine the location and height of openings in the shear walls associated with each diaphragm.

b. Calculate the total sum of the lengths of solid wall segments, those sheathed with structural panels, that extend from the bottom plate to the top of the double top plate.

Shear wall A: 11'-4" + 11'-4" = **22'-8"**

Shear Wall B: 6'-0" + 3'-0" = **9'-0"**

Shear Wall C: 5'-0" + 5'-0" = **10'-0"**

Diaphragm B

Shear Wall D: 12'-9" + 4'-3" = **17'-0"**

Shear Wall E: **Steel Rigid Frame**: Garage Door Opening

B. Wind Direction: North/South Direction: Diaphragm C and D

Shear Wall F: 5'-10" + 6'-6" + 6'-4" + 15'-8" = **34'-4"**

Shear Wall G: **20'-0"**

Shear Wall H: **20'-0"**

c. Calculate the shear (in plf) that the shear wall must transfer from the top of the wall down to the bottom of the wall.

$$v_{RF\perp ACT} = R_{RF\perp} \div \Sigma W_{solid} \quad \text{plf}$$

A. Wind Direction: East/West Direction Diaphragm A and AA

Shear wall A: v_{ACT} = 3098 lbs. ÷ 22.667'= **137 plf**

→ **Shear Wall B:** v_{ACT} = 3448 lbs. ÷ 9.0' = **383 plf**

Shear Wall C: v_{ACT} = 350 lbs. ÷ 10.0'= **35 plf**

Diaphragm B

Shear Wall D: v_{ACT} = 1300 lbs. ÷ 17.0' = **77 plf**

Shear Wall E: v_{ACT} = 1300 lbs. ÷ 20.0'= **65 plf**
(Shear transfer to steel rigid frame)

B. Wind Direction: North/South Direction: Diaphragm C and D

Shear Wall F: v_{ACT} = 2640 lbs. ÷ 34.333'= **77 plf**

→ **Shear Wall G:** v_{ACT} = 3690 lbs. ÷ 20.0'= **185 plf**

Shear Wall H: v_{ACT} = 1050 lbs. ÷ 20.0' = **53 plf**

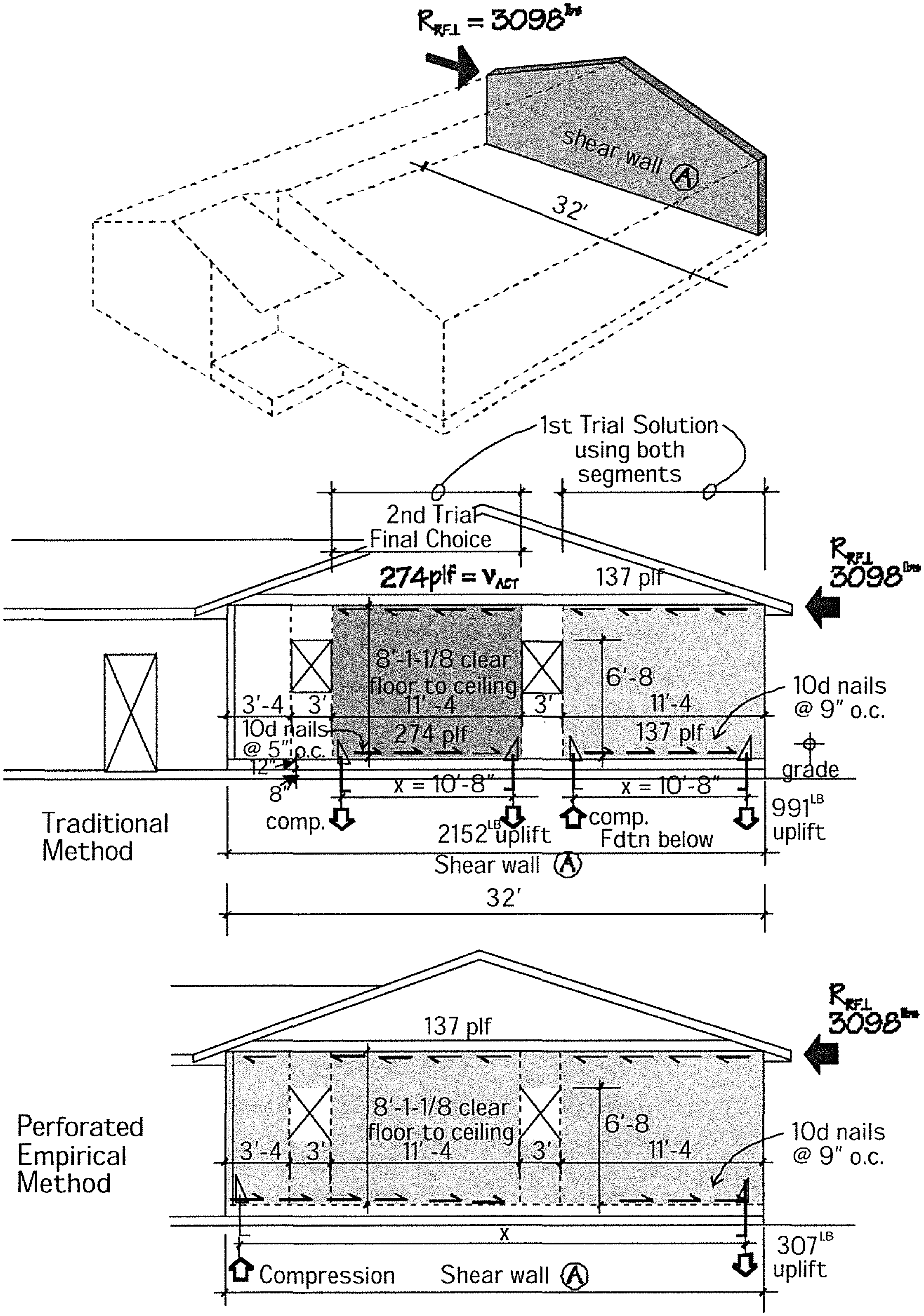


Figure 5.25 - Shear Wall A - Method Comparison

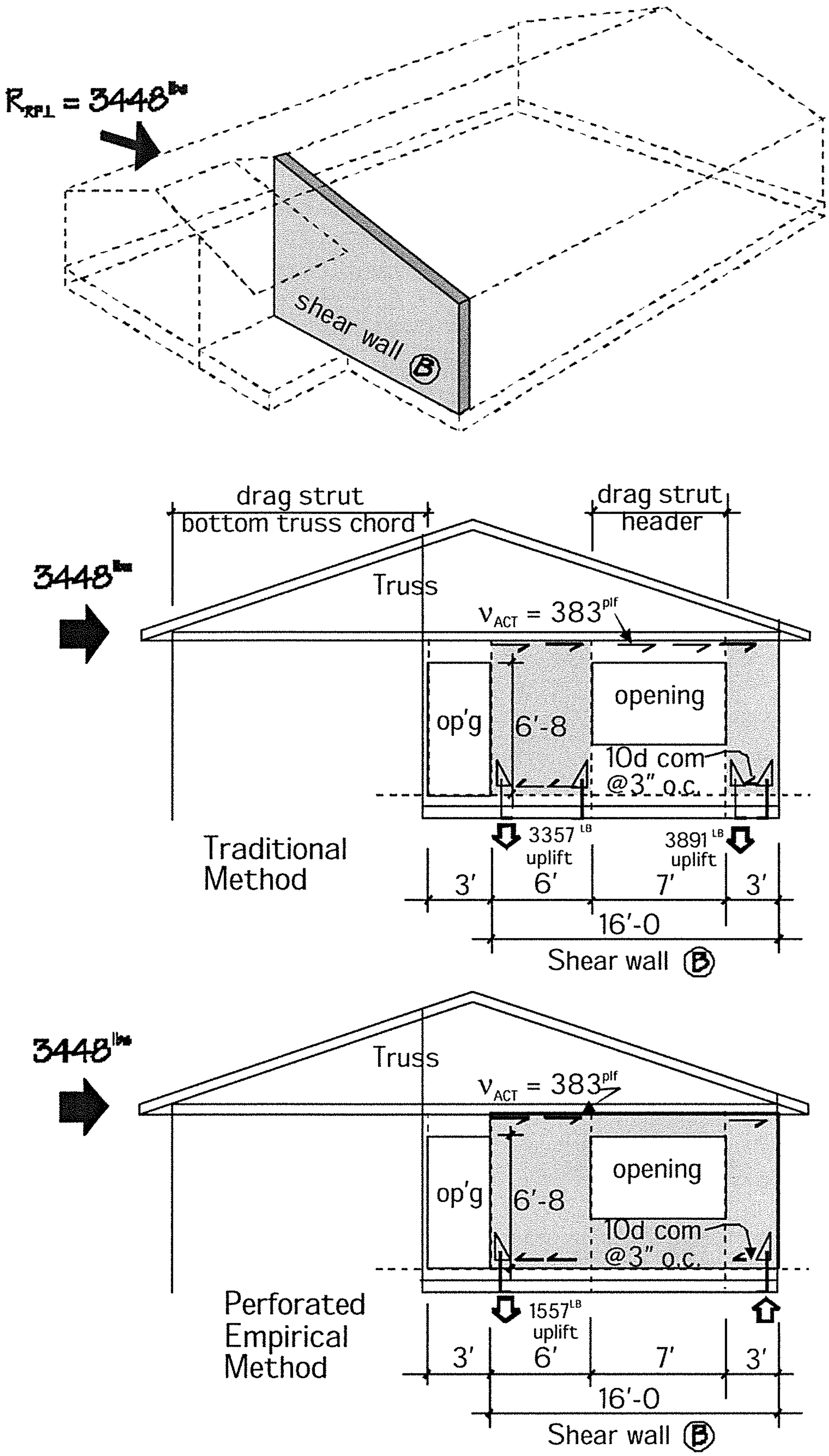


Figure 5.26 - Shear Wall B - Method Comparison

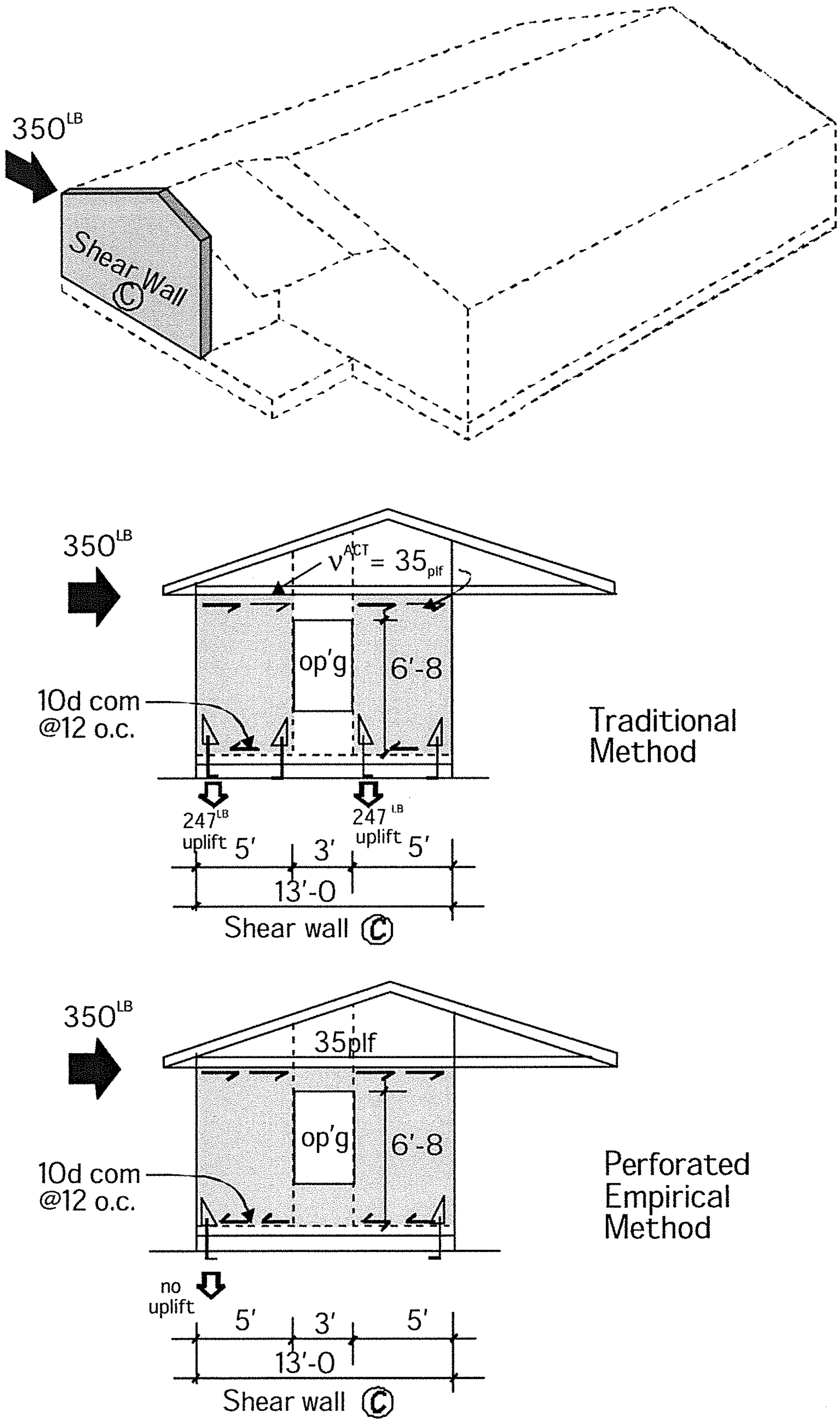
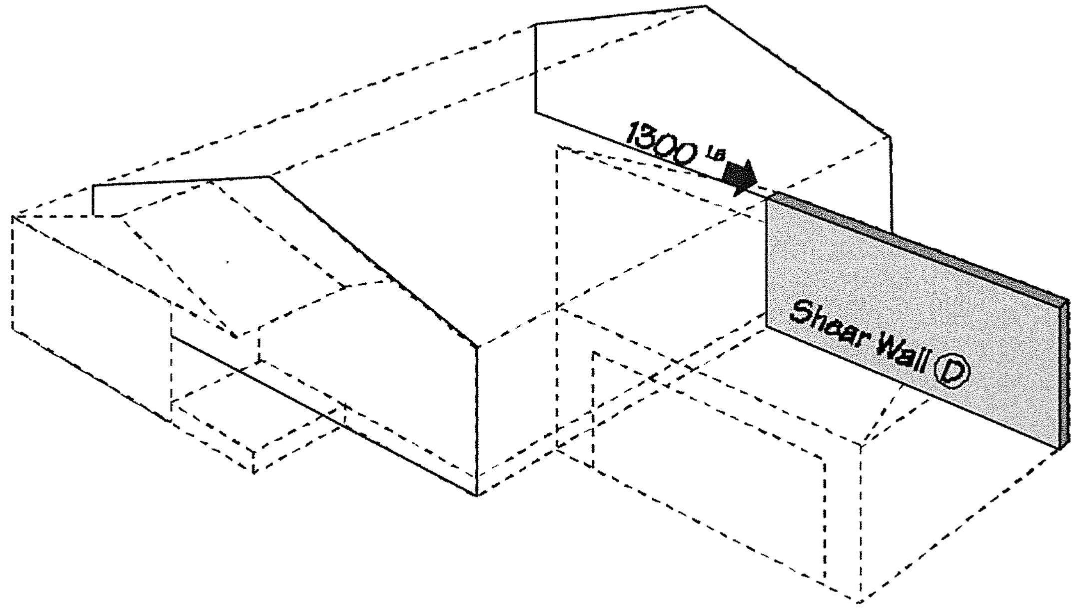
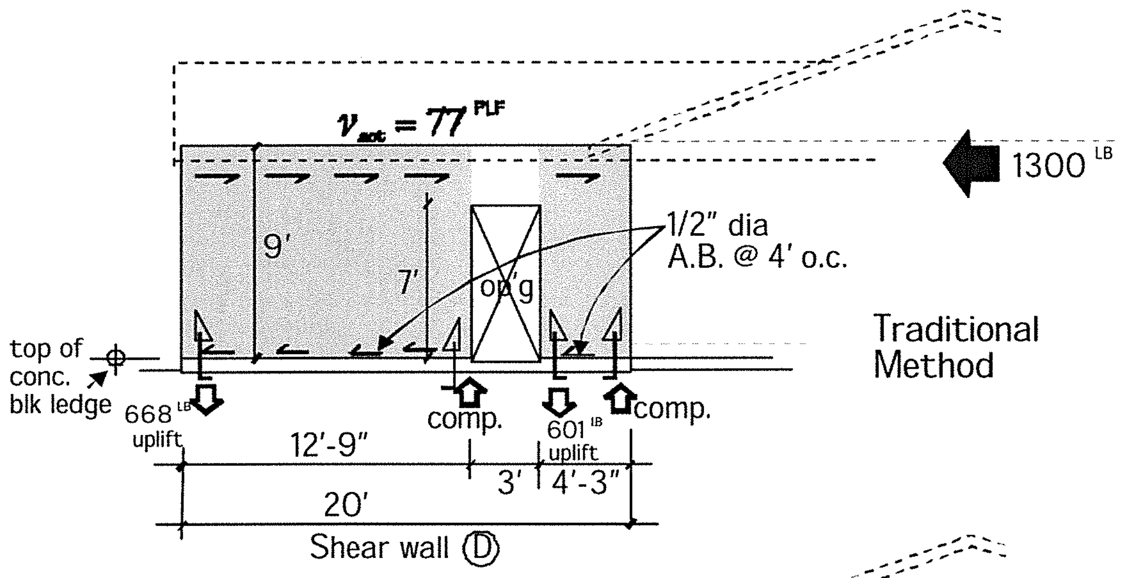


Figure 5.27 - Shear Wall C - Method Comparison





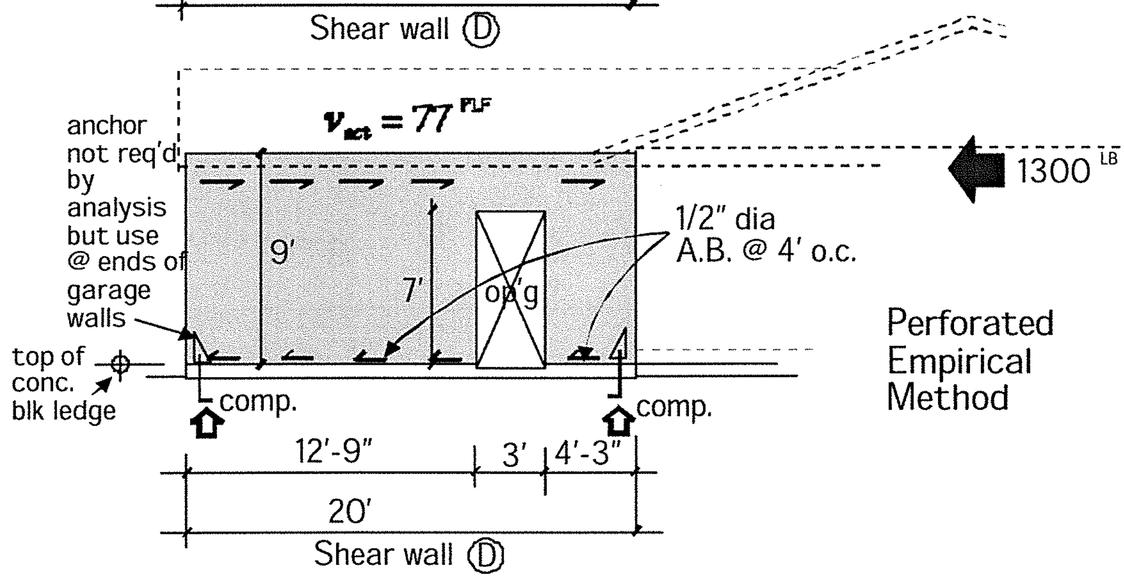
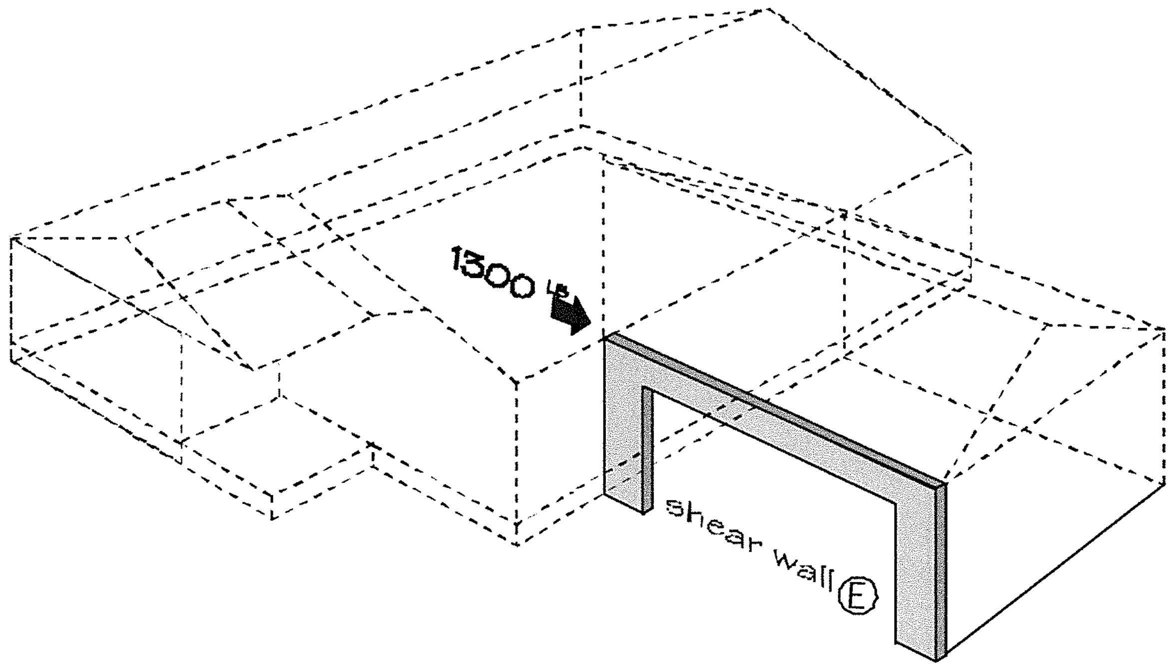
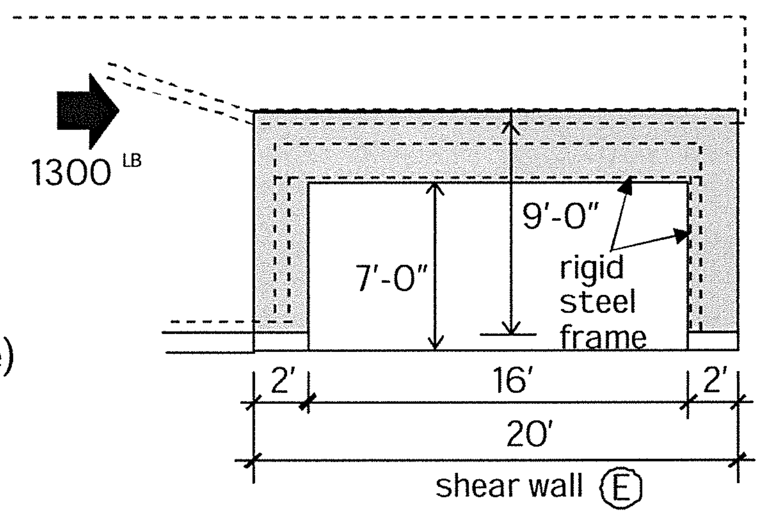


Figure 5.28 - Shear Wall D - Method Comparison

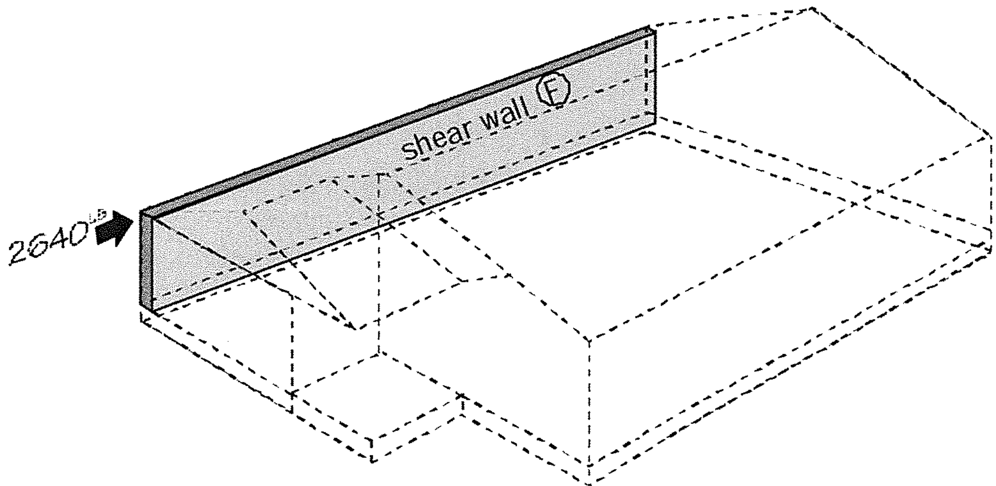


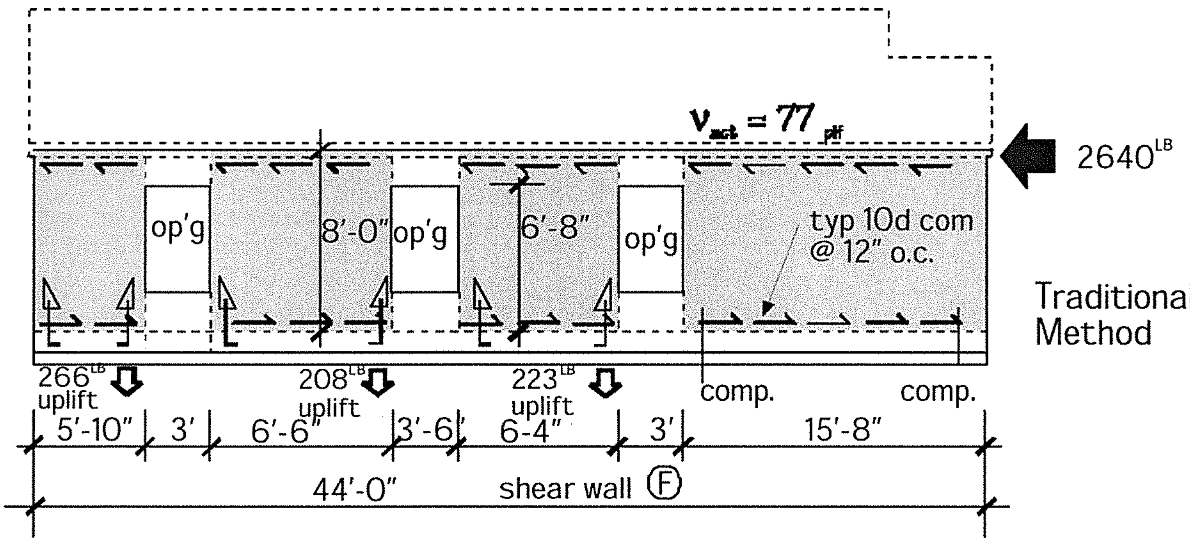


Traditional
Method
(not applicable)

Perforated
Empirical
Method
(not applicable)

Figure 5.29 - Shear Plane E - Method Comparison
Steel Rigid Frame Required





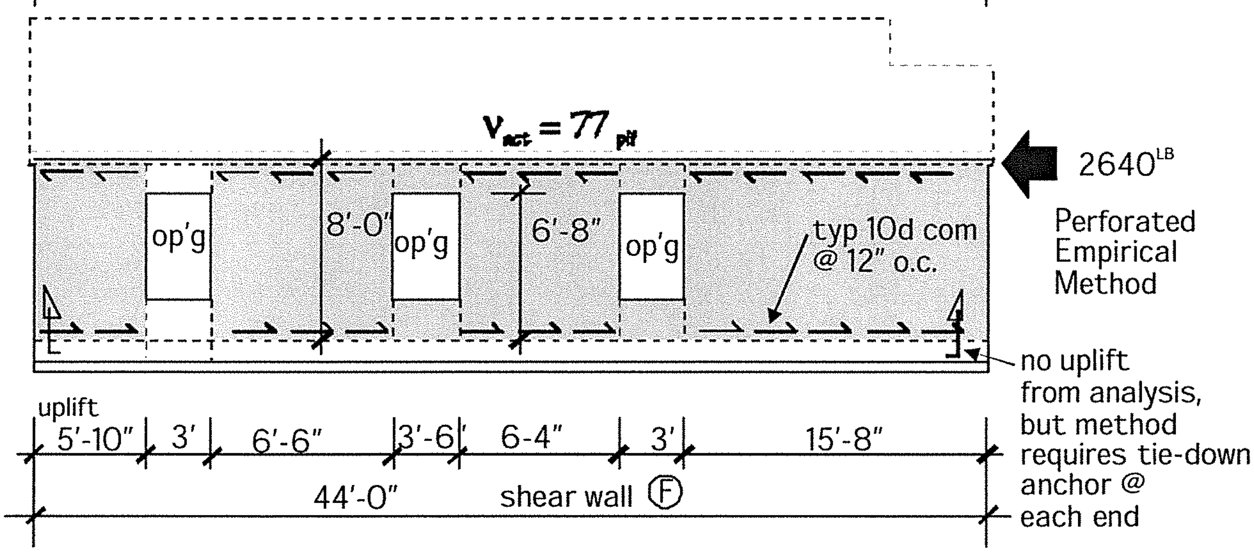


Figure 5.30 - Shear Wall F - Method Comparison

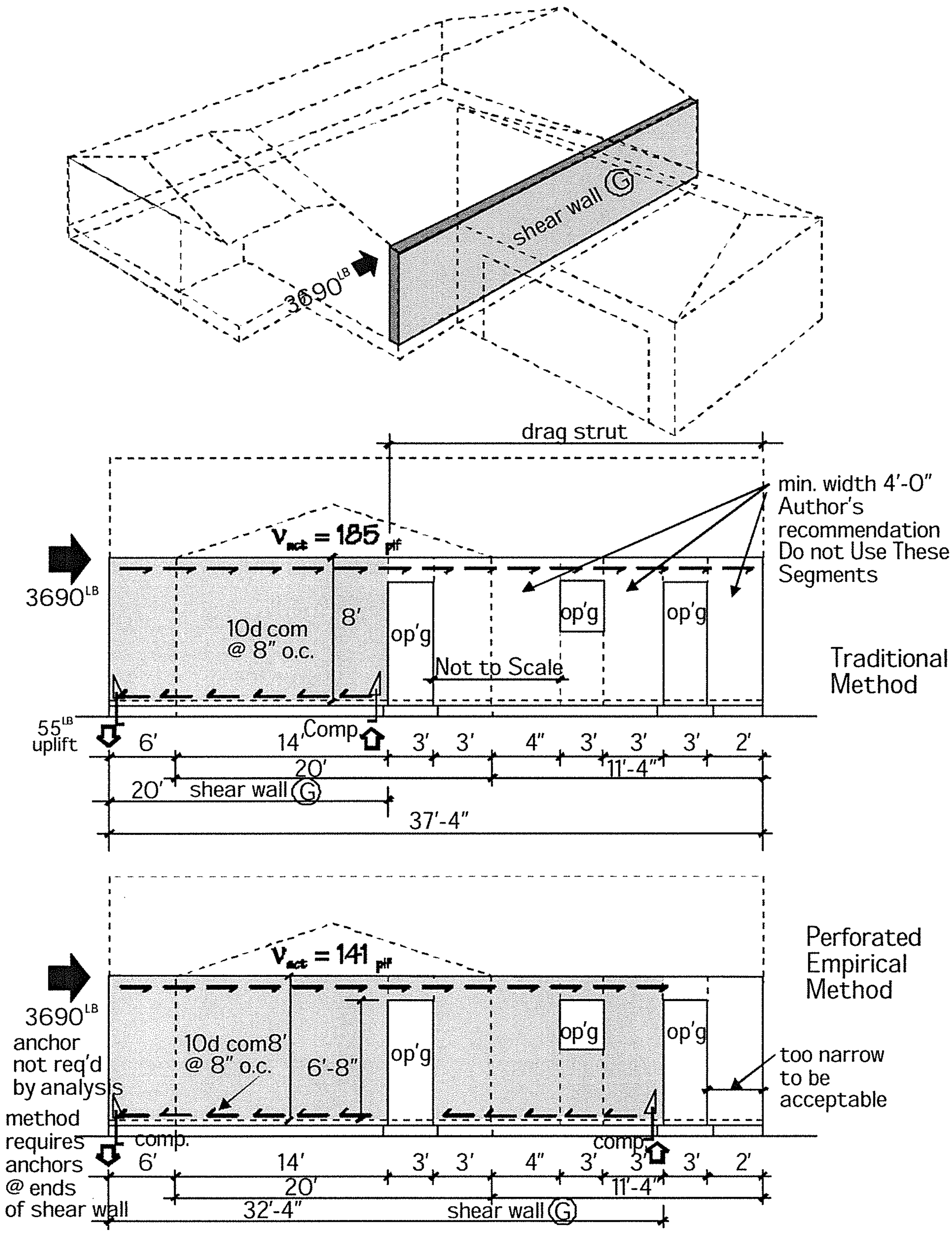
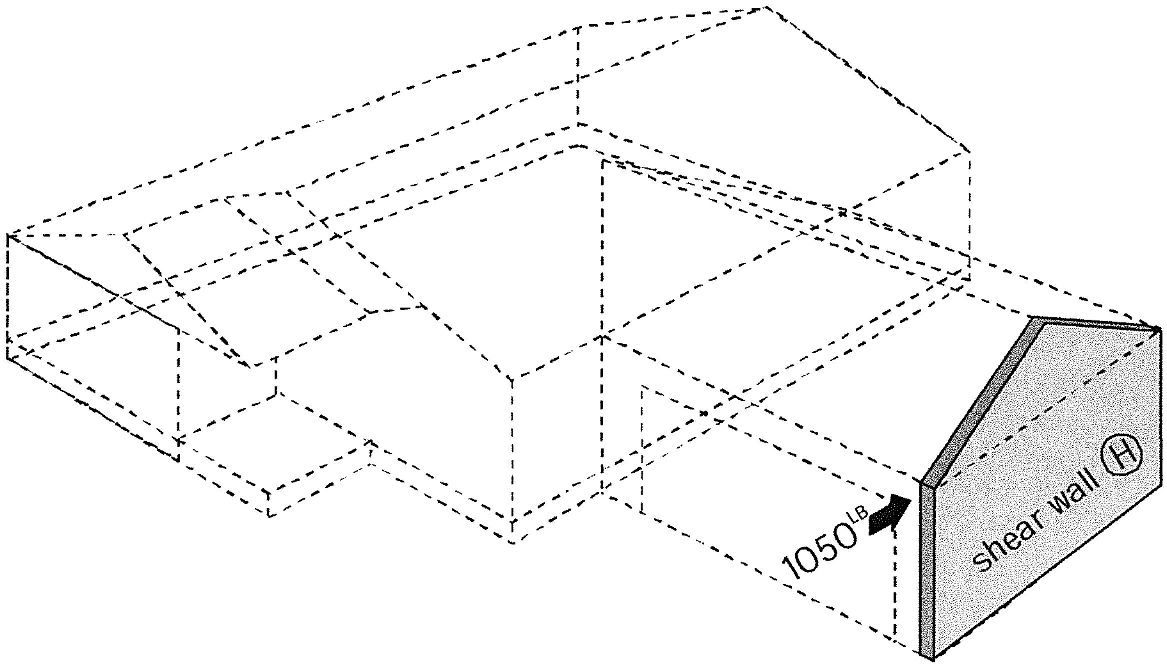
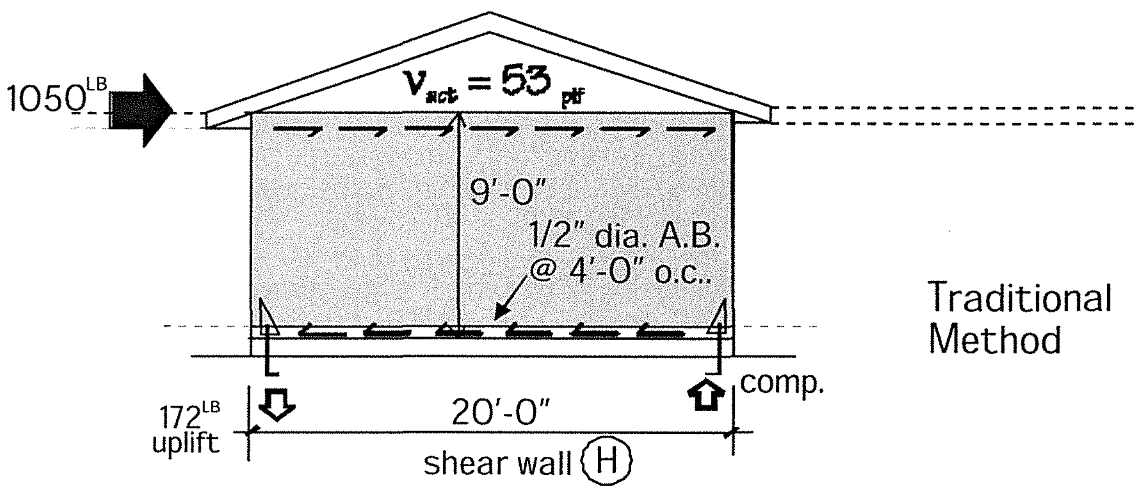


Figure 5.31 - Shear Wall G - Method Comparison





Traditional Method

Perforated Empirical Method results will be the same as the Traditional Method

Figure 5.32 - Shear Wall H - Method Comparison

d. Determine stud wall framing lumber species and grade as described in step #3 and sample dimension lumber grade stamps in Figure 5.3.

Dimension lumber selected for all the 2" x 4" wall studs shows on the grade stamp the following:

	Species = **S-P-F**
Table B.30 in Appendix B:	specific gravity **"G" = 0.42**.
	Construction Grade
	KD-15% Moisture Content
	(KD = kiln dried)

e. Select panel grade, thickness, nail size and nail spacing for the required shear $\mathbf{v_{ACT}}$:

NOTE: It is common today to include the shear wall capacity of the interior gypsum drywall. Refer to Table B.29 in Appendix B for shear values of various interior finishes. Add 100 plf of shear capacity to the structural wood panel shear capacity found in Table B.28 of Appendix B to account for gypsum drywall.

f. Select an Allowable shear and appropriate nailing to provide a shear allowable greater than the shear actual. Calculate the total length of solid full-height shear wall required based on the allowable shear capacity selected, using the following equation:

$$\mathbf{W_{solidreq} = R_{RF\perp \text{ or } RFII} \div v_{allow}} \qquad \text{feet}$$

Compare with the available: $\mathbf{\Sigma W_{solid} \geq W_{solidreq}}$

NOTE: An arrow ($\longrightarrow$) has been placed in front of the largest shear wall shear required, and another arrow has been placed in front of the largest typical group of required shears. See step (c) above. This is to simplify the number of different nailing requirements that the contractor and his crew will need to remember.

Review the use of Table B.28 in Appendix **B** from step#11 in the Procedure: Expanded Version. Enter Table B.28 at the second column for S-P-F species and horizontally at the middle row for APA Rated Sheathing. From this same Example in Chapter 4, 1/2" or 15/32" APA Rated Sheathing was selected for the uplift chain resistance. Also, 8d Com nails at 3" on center were selected for butt splices in step #6. See Step #5 and #6 in the solution to verify these facts.

Thus, 8d Com nails will be selected for the typical shear walls. Read 213 plf as an allowable shear capacity for 6" panel edge nail spacing. The 213 plf + 100 plf provides a combined allowable shear that includes the gypsum drywall. The 313 plf allow > 185 plf required shear of **Shear Wall G**. This will work for all shear walls except **Shear Wall B**, which requires a shear of 383 plf. Move to the next tabled value and read 312 plf for 4" nail spacing at all panel edges. Again adding the gypsum drywall produces an allowable shear of 312 + 100 = 412 plf. The 412 plf allow > 383 plf required shear for **Shear Wall B**. To summarize the required shear wall lengths using the two allowable shear values produces the following required lengths:

A. Wind Direction: East/West Direction Diaphragm A and AA

Shear Wall A: $W_{solidreq} = R_{RF\perp \text{ or } RF\parallel} \div v_{allow}$ feet

$W_{solidreq} = 3098 \div 313 = \mathbf{9.9}$ **feet**

→ **Shear Wall B:** $W_{solidreq} = 3448 \div 412 = \mathbf{8.4}$ **feet**

Shear Wall C: $W_{solidreq} = 350 \div 313 = \mathbf{1.1}$ **feet**

Diaphragm B

Shear Wall D: $W_{solidreq} = 1300 \div 313 = \mathbf{4.2}$ **feet**

Shear Wall E: $v_{ACT} = 1300$ lbs. $\div$ 20.0'= **65 plf**
(Shear transfer to steel rigid frame)

B. Wind Direction: North/South Direction: Diaphragm C and D

Shear Wall F: $W_{solidreq} = 2640 \div 313 = \mathbf{8.4}$ **feet**

→ **Shear Wall G:** $W_{solidreq} = 3690 \div 313 = \mathbf{11.8}$ **feet**

Shear Wall H: $W_{solidreq} = 1050 \div 313 = \mathbf{3.4}$ **feet**

g. Calculate the uplift potential of the shear wall segments by the following equations:

First, determine available superimposed dead load on the shear wall and its selfweight: Refer to Table A.11 in Appendix A for any floor framing dead load, and Table A.12 in Appendix A for the wall selfweight.

$$\mathbf{W_{self} = wall_{DL}} \times \text{length of shear wall segment} \qquad \text{(lbs.)}$$

$$\mathbf{W_{floor} = DL_f} \times \text{length of shear wall segment} \qquad \text{(lbs.)}$$

$$\mathbf{W_{roof} = DL_{rf}} \times \text{length of shear wall segment} \qquad \text{(lbs.)}$$

$$\mathbf{W_{totalDL} = W_{self} + W_{floor} + W_{roof}} \qquad \text{(lbs.)}$$

$$\mathbf{R_{anchup\perp} = (R_{RF\perp}} \times \mathbf{y - W_{totalDL}} \times \mathbf{x/2)/ x} \qquad \text{(lbs.)}$$

A. Wind Direction: East/West Direction Diaphragm A and AA

Shear Wall A: has two11'-4" segments, which will share the shear wall reaction. Roof and first floor framing run parallel to the shear wall and contribute no dead load to the shear wall. Only the wall self-weight and gable end triangular wall self-weight resist the overturning moment shown in Figure 5.15. Table A.12 and A.16 in Appendix A will provide the self-weight information:

For 2x4 studs at 16" o.c. with 1/2 inch structural sheathing panels and 1/2" gypsum drywall:

$\mathbf{Wall_{DL}}$ = 25.6 plf +2.7 x 1.7psf = **30.2 plf** avg.

$$\mathbf{W_{self} = wall_{DL}} \times \text{length of shear wall segment} \qquad \text{(lbs.)}$$

$\mathbf{W_{self}}$ = 30.2 plf × 11.333' = **342 lbs.**

Calculate the **overturning and uplift tension** resistance force for each segment:

$$\mathbf{R_{anchup\perp} = (R_{RF\perp SEGMENT}} \times \mathbf{y - W_{totalDL}} \times \mathbf{x/2)/ x} \qquad \text{(lbs.)}$$

$\mathbf{R}_{\mathbf{anchup}\perp}$ = (1549 lbs. × 8' - 342 lbs. × (11'-4" - 8")/2)/10.667 = **991 lbs. of uplift** for each segment, which is very small.

NOTE: If the result is a negative number, then no uplift occurs and no anchor is required. Also, x = length of shear wall - 8" to account for the centerline to centerline distance between the anchor bolts.

The **sliding of shear walls** is a function of a connection of the base of the wall to the supporting medium below the wall, in this case wood subfloor and TJI bandboard or Dimension Lumber bandboard, assuming the worst to be S-P-F lumber and the following fastener spacing equation:

$\mathbf{S}_{\mathbf{SLIDEWALL}}$ = Shear capacity of one fastener × 12 ÷ $\mathbf{v}_{\mathbf{ACT}}$

For **10d-Box nails** in lumber of **"G" = 0.42:**

$\mathbf{R}_{\mathbf{SLIDEWALL}}$ = 104 lbs. × 12 ÷ 137 plf = 9.1 inches, **use 9" o.c.** between bottom plate of wall and floor framing below.

It is easily observed that the full allowable shear of 313 plf is not being efficiently utilized, since two shear wall segments are contemplated where only one 9.9 ft. length of shear wall is required as calculated above. Thus, the most efficient design would use only one 11'-4" shear wall segment. The only equations that change are the uplift force due to overturning and the sliding. A re-calculation of the uplift equation for one shear wall, using 3098 lbs. instead of the value shared by two shear wall segments of 1549 lbs. follows:

$\mathbf{R}_{\mathbf{anchup}\perp}$ = (3098 lbs. × 8' - 342 lbs. × (11'-4" - 8")/2)/10.667 = **2152 lbs. of uplift** for each segment, which is more substantial, but still reasonably well accommodated by the weight of the foundation.

$\mathbf{R}_{\mathbf{SLIDEWALL}}$ = 104 lbs. × 12 ÷ 274 plf = 4.56 inches, **use 5" o.c.** between bottom plate of wall and bandboard framing below each shear wall segment.

For **10d-Box nails,** and conservatively using S-P-F lumber with **"G" = 0.42**

$\mathbf{v}_{\mathbf{ACT}}$ = 3098 ÷ 11.333 = 274 plf

Shear Wall B: has two segments: 6'-0" and 3'-0", which will share the shear wall reaction. Roof and first floor framing run parallel to the shear wall and contribute no dead load to the shear wall. Only the wall self-weight and gable end triangular wall self-weight resist the overturning moment similar to **Shear Wall A**.

$\mathbf{Wall_{DL}}$ = 25.6 plf +2.7 x 1.7psf = **30.2 plf** avg.

$\mathbf{W_{self} = wall_{DL}} \times$ length of shear wall segment (lbs.)

$\mathbf{W_{self1}} =$ 30.2 plf $\times$ 3' = **91 lbs.**

$\mathbf{W_{self2}} =$ 30.2 plf $\times$ 6' = **182 lbs.**

Calculate the **overturning and uplift tension** resistance force for each segment:

$\mathbf{R_{anchup\perp}} = \mathbf{(R_{RF\perp SEGMENT}} \times \mathbf{y - W_{totalDL}} \times \mathbf{x/2)/ x}$ (lbs.)

$\mathbf{R_{anchup\perp 1}}$ =(1149 lbs. $\times$ 8' - 91 lbs. $\times$ (3'-0" - 8")/2)/2.333' = **3891 lbs. of uplift** for this segment, which is very large.

$\mathbf{R_{anchup\perp 2}}$ =(2299 lbs. $\times$ 8' - 182 lbs. $\times$ (6'-0" - 8")/2)/5.333' = **3357 lbs. of uplift** for each segment, which is reasonable.

The **sliding of each shear wall:**

$\mathbf{S_{SLIDEWALL}}$ = Shear capacity of one fastener $\times$ 12 $\div$ $\mathbf{v_{ACT}}$

For **10d-Box nails** in lumber of **"G" = 0.42:**

$\mathbf{R_{SLIDEWALL}}$ = 104 lbs. $\times$ 12 $\div$ 383 plf = 3.25 inches, **use 3" o.c.** between bottom plate of wall and floor bandboard below at each shear wall segment.

Shear Wall C: has two segments 5'-0", which will equally share the shear wall reaction. Roof and first floor framing run parallel to the shear wall and contribute no dead load to the shear wall. Only the wall self-weight and gable end triangular wall self-weight resist the overturning moment similar to **Shear Wall A**.

$\mathbf{Wall_{DL}}$ = 25.6 plf +2.7 x 1.7psf = **30.2 plf** avg.

$\mathbf{W_{self} = wall_{DL}}$ $\times$ length of shear wall segment (lbs.)

$\mathbf{W_{self}}$ = 30.2 plf $\times$ 5' = **151 lbs.** at each wall segment

Calculate the **overturning and uplift tension** resistance force for each segment:

$\mathbf{R_{anchup\perp} = (R_{RF\perp SEGMENT} \times y - W_{totalDL} \times x/2)/ x}$ (lbs.)

$\mathbf{R_{anchup\perp}}$ = (175 lbs. $\times$ 8' - 151 lbs. $\times$ (5'-0" - 8")/2)/4.333' = **247 lbs. of uplift** for each segment, which is very small.

The **sliding of each shear wall:**

$\mathbf{S_{SLIDEWALL}}$ = Shear capacity of one fastener $\times$ 12 $\div$ $\mathbf{v_{ACT}}$

For **10d-Box nails** in lumber of **"G" = 0.42:**

$\mathbf{R_{SLIDEWALL}}$ = 104 lbs. $\times$ 12 $\div$ 35 plf = 35.6 inches, **use 12" o.c. max.** between bottom plate of wall and floor bandboard below at each shear wall segment.

Diaphragm B

Shear Wall D: has two segments: 12'-9" and 4'-3", which will share the shear wall reaction. Roof framing bears on the shear wall and contributes dead load to the shear wall. The wall self-weight and roof load resist the overturning moment.

$\mathbf{Wall_{DL}}$ = 25.6 plf * 9/8 = **28.8 plf**

$\mathbf{W_{self1}}$ = 28.8 plf $\times$ 4.25' = **122.4 lbs.** segment 1

$\mathbf{W_{self2}}$ = 28.8 plf $\times$ 12.75' = **367.2 lbs.** segment 2

From Table B.31 in Appendix B:

$$\mathbf{W_{roof} = DL_{rf}} \times \text{length of shear wall segment} \qquad (\text{ lbs. })$$

$$\mathbf{W_{roof1}} = 76 \text{ plf} \times 4.25' = \mathbf{323 \ \ lbs.}$$

$$\mathbf{W_{roof2}} = 76 \text{ plf} \times 12.75' = \mathbf{969 \ \ lbs.}$$

$$\mathbf{W_{totalDL} = W_{self} + W_{roof}} \qquad (\text{ lbs. })$$

$$\mathbf{W_{totalDL1}} = 122.4 + 323 = \mathbf{445.4 \quad lbs.}$$

$$\mathbf{W_{totalDL2}} = 367.2 + 969 = \mathbf{1336.2 \quad lbs.}$$

Calculate the **overturning and uplift tension** resistance force for each segment:

$$\mathbf{R_{anchupll}} = \mathbf{(R_{RF\perp SEGMENT}} \times \mathbf{y - W_{totalDL}} \times \mathbf{x/2)/ x} \qquad (\text{ lbs. })$$

$\mathbf{R_{anchupll1}}$ =(327 lbs. × 9' - 445.4 lbs. × (4'-3" - 8")/2)/3.57' = **601 lbs. of uplift** for segment 1, which is quite minimal.

$\mathbf{R_{anchupll2}}$ =(982 lbs. × 9' - 1336.2 lbs. × (12.75' - 8")/2)/12.07' = **668 lbs. of uplift** for segment 2, which is quite minimal. Since this is a garage wall, this author recommends that every corner of the garage have a tie-down anchor whether required or not.

The **sliding of each shear wall** segment:

$$\mathbf{S_{SLIDEWALL}} = \text{Shear capacity of one fastener} \times 12 \div \mathbf{v_{ACT}}$$

For **10d-Box nails** in lumber of **"G" = 0.42:**

$\mathbf{R_{SLIDEWALL}}$ = 1130 lbs. × 12 ÷ 77 plf = 176 inches, **use 4" o.c. max.** 1/2" ϕ Anchor bolt, between bottom plate of wall and top of grouted concrete block below each shear wall segment. Tension Chain uplift controls spacing.

Shear Wall E: has two segments: each 2'-0", which are a width less than the minimum recommended by this author. Roof framing bears on the garage door header within the wall plane and will be counted. The wall weight and header weights are minimal and will be ignored.

W_{roof} = **76 plf** and will be applied to the gravity load design of the steel rigid frame, but must be multiplied by 3/2 to re-create the actual dead load.

R_{RFII} = **1300 lbs.** and will be applied to the lateral load design of the rigid frame in combination with the 2/3rds reduced roof dead load of 76 plf.

The steel rigid frame must be designed by a structural engineer and will not be addressed in this Companion Manual.

B. Wind Direction: North/South Direction: Diaphragm C and D

Shear Wall F: has four segments: 5'-10", 6'-6", 6'-4" and 15'-8", which will share the shear wall reaction according to their respective length. Roof framing bears on the shear wall and contributes dead load to the shear wall. The wall self-weight and roof load resist the overturning moment.

$Wall_{DL}$ = **25.6 plf**

W_{self1} = 25.6 plf × 5.83' = **149 lbs.** segment 1

W_{self2} = 25.6 plf × 6.5'= **166 lbs.** segment 2

W_{self3} = 25.6 plf × 6.33'= **162 lbs.** segment 3

W_{self4} = 25.6 plf × 15.667'= **401 lbs.** segment 4

From Table B.31 in Appendix B and a span of **32'**:

W_{roof} = DL_{rf} × length of shear wall segment (lbs.)

W_{roof1} = 121.6 plf × 5.83' = **709 lbs.**

W_{roof2} = 121.6 plf × 6.5'= **790 lbs**.

W_{roof3} = 121.6 plf × 6.33'= **770 lbs**.

W_{roof4} = 121.6 plf × 15.667'= **1905 lbs**.

$W_{totalDL} = W_{self} + W_{roof}$ (lbs.)

$W_{totalDL1}$ = 149+ 709 = **858 lbs.**

$W_{totalDL2}$ = 166+ 790 = **956 lbs.**

$W_{totalDL3}$ = 162+ 770 = **932 lbs.**

$W_{totalDL4}$ = 401+ 1905 = **2306 lbs.**

Calculate the **overturning and uplift tension** resistance force for each segment:

$R_{anchupll} = (R_{RF\perp SEGMENT} \times y - W_{totalDL} \times x/2)/ x$ (lbs.)

$R_{anchupll1}$ =(449 lbs. × 8' - 858 lbs. × (5'-10" - 8")/2)/5.17 = **266 lbs. of uplift** for segment 1, which is minimal.

$R_{anchupll2}$ =(500 lbs. × 8' - 956 lbs. × (6'-6" - 8")/2)/5.83' = **208 lbs. of uplift** for segment 2, which is minimal.

$R_{anchupll3}$ =(488 lbs. × 8' - 932 lbs. × (6'-4" - 8")/2)/5.67' = **223 lbs. of uplift** for segment 3, which is minimal.

$R_{anchupll4}$ =(1155 lbs. × 8' - 2306 lbs. × (15'-8" - 8")/2)/15.0' = **-537 lbs. of compression** for segment 4, which means no uplift and no tie-down anchor is required.

The **sliding of each shear wall** segment:

$S_{SLIDEWALL}$ = Shear capacity of one fastener $\times$ 12 $\div$ v_{ACT}

For **10d-Box nails** in lumber of **"G" = 0.42:**

$R_{SLIDEWALL}$ = 104 lbs. $\times$ 12 $\div$ 77plf = 16.2 inches, **use 12" o.c. max.** between bottom plate of wall and floor bandboard below at each shear wall segment.

Shear Wall G: has four segments, but 3 of them are less than 4'0" and are not recommended to be used. Therefore, use one segment 20'-0", which is a completely solid wall segment. Roof framing bears on the shear wall and contributes dead load to the shear wall. The wall self-weight and roof load resist the overturning moment.

$\mathbf{Wall_{DL} = 25.6\ plf}$

W_{self1} = 25.6 plf $\times$ 20' = **512 lbs.**

From Table B.31 in Appendix **B**:

$W_{roof} = DL_{rf} \times$ length of shear wall segment (lbs.)

W_{roof1} = 121.6 plf $\times$ 20.0' = **2432 lbs.**

$W_{totalDL} = W_{self} + W_{roof}$ (lbs.)

$W_{totalDL}$ = 512 + 2432 = **2944 lbs.**

Calculate the **overturning and uplift tension** resistance force for each segment:

$R_{anchupll} = (R_{RF\perp SEGMENT} \times y - W_{totalDL} \times x/2)/ x$ (lbs.)

$R_{anchupll}$ = (3690 lbs. $\times$ 8' - 2944 lbs. $\times$ (20'-0" - 8")/2)/19.33' = **55 lbs. of uplift** for segment 1, which is minimal. Since this is a garage wall, this author recommends that every corner of the garage have a tie-down anchor whether required or not. Place an anchor at the South End of the wall.

The **sliding of each shear wall** segment:

$\mathbf{S_{SLIDEWALL}}$ = Shear capacity of one fastener × 12 ÷ $\mathbf{v_{ACT}}$

For **10d-Box nails** in lumber of **"G" = 0.42:**

$\mathbf{R_{SLIDEWALL}}$ = 104 lbs. × 12 ÷ 185 plf = 6.7 inches, **use 6" o.c.** between bottom plate of wall and floor bandboard below the shear wall segment.

Shear Wall H: has one 20'-0" solid wall segment. Roof framing runs parallel to the shear wall and contributes no dead load to the shear wall. Only the wall self-weight and gable end triangular wall self-weight resist the overturning moment shown in Figure 5.15. Table A.12 and A.16 in Appendix A will provide the self-weight information:

For 2x4 studs at 16" o.c. with 1/2 inch structural sheathing panels and 1/2" gypsum drywall the wall reduced DL = 25.6 plf, and enter Table A.16 with 20' span and 4/12 slope and read $\mathbf{H_{avg}}$ = 1.7':

$\mathbf{Wall_{DL}}$ = 25.6 plf x 9'/8' +1.7 x 1.7psf = **31.7 plf** avg.

$\mathbf{W_{self}}$ **=** $\mathbf{wall_{DL}}$ × length of shear wall segment (lbs.)

$\mathbf{W_{self}}$ **=** 31.7 plf × 20.0' = **634 lbs.**

Calculate the **overturning and uplift tension** resistance force for each segment:

$\mathbf{R_{anchup\perp}}$ = **(** $\mathbf{R_{RF\perp SEGMENT}}$ × **y - $\mathbf{W_{totalDL}}$** × **x/2)/ x** (lbs.)

$\mathbf{R_{anchup\perp}}$ = (1050 lbs. × 9' - 634 lbs. × (20'-0" - 8")/2)/19.333 = **172 lbs. of uplift** for the segment, which is very small.

$\mathbf{R_{SLIDEWALL}}$ = 1130 lbs. × 12 ÷ 53 plf = 256 inches, **use 4' o.c. max.** 1/2" ϕ Anchor bolt, between bottom plate of wall and top of grouted concrete block below each shear wall segment. Tension Chain uplift controls spacing.

10. **Perforated Shear Wall Empirical Design Method**: Refer to Figures 5.25 through 5.32 for illustrations comparing this method to the Traditional Method for each shear wall.

a. Determine the location and height of openings in the shear walls associated with each diaphragm. Calculate the height of the tallest opening $\mathbf{H_{OPG}}$ and ratio it to the height of the wall $\mathbf{H}$.

$$\text{Ratio} = \mathbf{H_{OPG}} / \mathbf{H}$$

All opening heights are 6'-8" from the bottom of the bottom plate of the walls of the house. Garage openings are 7'-0" from the bottom of the wall stud bottom plate.

House: Ratio = 6.667"/8.0" = **0.833**

Garage: Ratio = 7.0"/9.0" = **0.778**

b. Calculate the total sum of the lengths of solid wall segments, those sheathed with structural panels, that extend from the bottom plate to the top of the double top plate across the total length of the elevation $\mathbf{W_{wall}}$.

$$\beta_1 = \Sigma \mathbf{W_{solid}} / \mathbf{W_{wall}}$$

A. Wind Direction: East/West Direction Diaphragm A and AA

Shear wall A: 3'-4" + 11'-4" + 11'-4" = **26'-0"**

$$\beta_1 = \Sigma \mathbf{W_{solid}} / \mathbf{W_{wall}} = 26.0'/32' = \mathbf{0.8125}$$

Shear Wall B: 6'-0" + 3'-0" = **9'-0"**

$$\beta_1 = \Sigma \mathbf{W_{solid}} / \mathbf{W_{wall}} = 9.0'/16' = \mathbf{0.5625}$$

Shear Wall C: 5'-0" + 5'-0" = **10'-0"**

$$\beta_1 = \Sigma \mathbf{W_{solid}} / \mathbf{W_{wall}} = 10.0'/13' = \mathbf{0.769}$$

Diaphragm B

Shear Wall D: 12'-9" + 4'-3" = **17'-0"**

$$\beta_1 = \Sigma W_{solid} / W_{wall} = 17.0'/20' = \mathbf{0.850}$$

Shear Wall E: **Steel Rigid Frame**: Garage Door Opening

B. Wind Direction: **North/South Direction**: **Diaphragm C and D**

Shear Wall F: 5'-10" + 6'-6" + 6'-4" + 15'-8" = **34'-4"**

$$\beta_1 = \Sigma W_{solid} / W_{wall} = 34.333'/44' = \mathbf{0.780}$$

Shear Wall G: 20'-0" + 3'-4" + 3'-0" = **26'-4"**

$$\beta_1 = \Sigma W_{solid} / W_{wall} = 26.333'/37.333' = \mathbf{0.7054}$$

Shear Wall H: **20'-0"**

$$\beta_1 = \Sigma W_{solid} / W_{wall} = 20.0'/20' = \mathbf{1.0}$$

c. Calculate the length increase factor, which has been determined empirically to be:

$$R_\beta = \frac{1}{\dfrac{(1 - \beta_1)}{3 \times \left[\dfrac{H_{OPG}}{H_{wall}}\right]} + \beta_1}$$

d. Calculate the minimum required length of perforated shear wall from the following equation:

$$W_{perf} = W_{solidreq} \times R_?$$

Compare: $W_{perf} \leq W_{wall}$

A. Wind Direction: East/West Direction Diaphragm A and AA

Shear Wall A: $R_? = 1.127$

$W_{perf} = W_{solidreq} \times R_?$? ? ? '? ? ? ? ? '

Compared with *the traditional shear wall design method*, using one shear wall segment of 11'-4", nothing is gained by use of the *perforated empirical design method*. However, to place anchor bolts at each end of the 32' total length of the elevation would improve the overall behavior under wind. A check should be made of overturning and sliding as follows:

$Wall_{DL}$ = 25.6 plf +2.7 x 1.7psf = **30.2 plf** avg.

W_{self} = 30.2 plf × 32' = **966 lbs.**

$R_{anchup\perp}$ = (3098 lbs. × 8' - 966 lbs. × (32'-0" - 8")/2)/31.333 = **307 lbs. of uplift** for the entire length of wall, which is very small.

NOTE: A re-calculation of the sliding equation is not required. The perforated Shear wall design method still requires that sliding be handled only by the full height sheathed segments of the shear wall. Thus, v_{ACT} is unchanged from the traditional method.

$R_{SLIDEWALL}$ = 104 lbs. × 12 ÷ 137 plf = 9.1 inches, **use 9" o.c. 10d-COM** nails between bottom plate of wall and bandboard framing below.

The perforated shear wall solution uses less anchors, reducing cost.

→ **Shear Wall B:** $R_? = 1.356$

W_{perf} ? ? ? '? ? ? ? ? ? ' **< 16' ok**

Compared with the *traditional shear wall design method*, using two shear wall segments of 6'-0" and 3'-0", much can be gained by the use of the *perforated empirical design method*. Use the full 16 feet for the shear wall length and place anchor bolts at each end of the 16' total length of the elevation. A check should be made of overturning and sliding as follows:

$\textbf{Wall}_{\textbf{DL}}$ = 25.6 plf +2.7 x 1.7psf = **30.2 plf** avg.

$\textbf{W}_{\textbf{self}}$ = 30.2 plf × 16' = **483 lbs.**

$\textbf{R}_{\textbf{anchup}\perp}$ = (3448 lbs. × 8' - 483 lbs. × (16'-0" - 8")/2)/15.333 = 1557 **lbs. of uplift** for the entire length of wall, which is a reasonable amount to be taken by the foundation.

$\textbf{R}_{\textbf{SLIDEWALL}}$ = 104 lbs. × 12 ÷ 383 plf = 3.25 inches, **use 3" o.c.** between bottom plate of wall and bandboard framing below.

For **10d-Box nails,** and conservatively using S-P-F lumber with **"G" = 0.42**

The perforated shear wall solution uses less anchors, reducing cost.

Shear Wall C: $\textbf{R}_{?}$ = **1.161**

$\textbf{W}_{\textbf{perf}}$? ?'??? ? ? ?'

Compared with the *traditional shear wall design method*, using two shear wall segments of 5'-0" each; much can be gained by the use of the *perforated empirical design method*. Use the full 13 feet for the shear wall length and place anchor bolts at each end of the elevation. A check should be made of overturning and sliding as follows:

$\textbf{Wall}_{\textbf{DL}}$ = 25.6 plf +2.7 x 1.7psf = **30.2 plf** avg.

$\textbf{W}_{\textbf{self}}$ = 30.2 plf × 13' = **393 lbs.**

$R_{anchup\perp}$ = (175 lbs. × 8' - 483 lbs. × (13'-0" - 8")/2)/12.333' = **-128 lbs. of compression.** There is no uplift to be taken by the foundation. The perforated empirical method requires a tie-down anchor at each end of the elevation.

$R_{SLIDEWALL}$ = 104 lbs. × 12 ÷ 35 plf = 35.6 inches, **use 12" o.c. max.** between bottom plate of wall and bandboard framing below.
For **10d-Box nails,** and conservatively using S-P-F lumber with **"G" = 0.42**

The perforated shear wall solution uses the same nailing but an anchor at each end of the elevation, reducing cost.

Diaphragm B

Shear Wall D: $R_{?}$ = **1.094**

W_{perf} ? ? ? "?? ? ? ? ? ?'

Compared with the *traditional shear wall design method*, using two shear wall segments of 4'-3" and 12'-9"; the *perforated empirical design method may offer some savings*. Use the full 20 feet for the shear wall length and place tie-down anchors at each end of the elevation. A check should be made of overturning and sliding as follows:

$Wall_{DL}$ = 25.6 plf * 9'/8' = **28.8 plf**

W_{self} = 28.8 plf × 20.0' = **576 lbs.** Entire wall length

From Table 31 in Appendix **B**:

W_{roof} = DL_{rf} × length of shear wall segment (lbs.)

W_{roof} = 76 plf × 20.0' = **1520 lbs.**

$W_{totalDL}$ = W_{self} + W_{roof} (lbs.)

$W_{totalDL}$ = 576 + 1520 = **2096 lbs.**

$R_{anchupll}$ = (1300 lbs. × 8' - 2096 lbs. × (20'-0" - 8")/2)/19.333' = **-510 lbs. of negative uplift.** There is no required uplift to be resisted by the foundation. Again, as for the traditional method, this author recommends that tie-down anchors be provided at each end of a garage wall.

$R_{SLIDEWALL}$ = 1130 lbs. × 12 ÷ 77 plf = 176 inches, **use 4' o.c. max. 1/2" ϕ Anchor bolt**, between bottom plate of wall and top of grouted concrete block below each shear wall segment. Tension Chain uplift controls spacing.

The perforated shear wall solution uses one less tie down anchor, reducing cost.

Shear Wall E: **Steel Rigid Frame**: Garage Door Opening. See traditional Method discussion of this wall plane.

B. Wind Direction: **North/South Direction**: **Diaphragm C and D**

Shear Wall F: **$R_{?}$ = 1.152**

W_{perf} ? ? ? "[illegible] ? ? ? ?' < 34.333'

Compared with *the traditional shear wall design method*, using four shear wall segments of 5'-10", 6'-6", 6'-4" and 15'-8"; the *perforated empirical design method will offer some savings*. Use the full 44 feet for the shear wall length and place anchor bolts at each end of the elevation. Roof dead load bears on this wall. A check should be made of overturning and sliding as follows:

$Wall_{DL}$ = 25.6 plf

W_{self} = 25.6 plf × 44.0' = **1126 lbs.** Entire wall length

From Table B.31 in Appendix B: using a span of 32' and roof slope of 4 in 12:

$$\mathbf{W_{roof} = DL_{rf}} \times \text{length of shear wall segment} \qquad (\text{ lbs. })$$

$$\mathbf{W_{roof}} = 121.6 \text{ plf} \times 44.0' = \mathbf{5350 \ \ lbs.}$$

$$\mathbf{W_{totalDL} = W_{self} + W_{roof}} \qquad (\text{ lbs. })$$

$$\mathbf{W_{totalDL}} = 1126 + 5350 = \mathbf{6476 \text{ lbs.}}$$

$\mathbf{R_{anchupll}}$ = (2640 lbs. × 8' - 6476 lbs. × (44'-0" - 8")/2)/43.333' = **-2750 lbs. of compression.** There is no required uplift to be resisted by the foundation. The perforated empirical method requires a tie-down anchor at each end of the elevation.

$\mathbf{R_{SLIDEWALL}}$ = 104 lbs. × 12 ÷ 77 plf = 16.2 inches, **use 12" o.c. max.** between bottom plate of shear wall full height segments and bandboard framing below.

For **10d-Box nails,** and conservatively using S-P-F lumber with **"G" = 0.42**

The perforated shear wall solution uses the same nailing but reduced tie-down anchors, reducing cost.

Shear Wall G: $\mathbf{R_{?}}$ = **1.215**

$\mathbf{W_{perf}}$? ? "?? ?? ? ?? ? **'<26'-4"**

Compared with the *traditional shear wall design method*, using one shear wall segments of 20'-0"; the *perforated empirical design method may offer some savings*. Use the full 26'-4" feet for the shear wall length, ignoring the 2'-0" segment as not acceptable. Place anchor bolts at each end of the 26'-4" length of the elevation. A check should be made of overturning and sliding as follows:

$$\mathbf{Wall_{DL} = 25.6 \text{ plf}}$$

W_{self} = 25.6 plf × 26.333' = **674 lbs.** Entire wall length

From Table B.31 in Appendix B:
for span = 32' and slope = 4/12

$W_{roof} = DL_{rf}$ × length of shear wall segment (lbs.)

W_{roof} = 121.6 plf × 26.333' = **3202 lbs.**

$W_{totalDL} = W_{self} + W_{roof}$ (lbs.)

$W_{totalDL}$ = 674 + 3202 = **3876 lbs.**

v_{ACT} = 3690 ÷ 26.333' = **141 plf**

$R_{anchupll}$ = (3690 lbs. × 8' - 3876 lbs. × (26'-4" - 8")/2)/25.667' = **-788 lbs. of compression.** There is no required uplift to be resisted by the foundation. Again, as for the traditional method, this author recommends that tie-down anchors be provided at each end of a wall associated with the garage.

$R_{SLIDEWALL}$ = 104 lbs. × 12 ÷ 141 plf = 8.9 inches, **use 8" o.c.** between bottom plate of shear wall full height segments and bandboard framing below.

For **10d-Box nails,** and conservatively using S-P-F lumber with **"G" = 0.42**

The perforated shear wall solution uses larger nail spacing but the same number of tie down anchors, slightly reducing cost.

Shear Wall H: **$R_?$ = 1.0**

W_{perf} ? ? ? "?**? ??'**

Compared with the *traditional shear wall design method*, using one **solid shear wall** segment of 20'-0"; the *perforated empirical design method will not offer any savings,* since there are no openings and the length increase factor is 1.0. A check of overturning and sliding will produce the same numbers as for the traditional method.

Floor Diaphragm

Wind Direction: **East/West Direction**

The floor diaphragm has a shear value that must transfer from the floor framing to the mudsill and to the foundation. The shear is $\mathbf{V_{F1\perp} = 212}$ **plf**.

$R_{SLIDEWALL}$ = 1130 lbs. × 12 ÷ 212 plf = **5'-4"** , **use 4' o.c. max. 1/2" ϕ Anchor bolt**, between mudsill and top of grouted concrete block below each shear wall segment. Tension Chain uplift controls spacing.

Wind Direction: **North/South Direction**

The floor diaphragm has a shear value that must transfer from the floor framing to the mudsill and to the foundation. The shear is $\mathbf{V_{F1\perp} = 123}$ **plf**.

$R_{SLIDEWALL}$ = 1130 lbs. × 12 ÷ 123 plf = **9'-3", use 4' o.c. max. 1/2" ϕ Anchor bolt**, between mudsill and top of grouted concrete block below each shear wall segment. Tension Chain uplift controls spacing.

11. Determine gable end wall stability. Either the ceiling plane is capable of providing the stability or diagonal wood struts must provide the stability. Refer to Tables B.17 through B.20 in Appendix B for windward, leeward and total wind loads in plf. Review Figure 5.33 for an illustration of use of the ceiling plane diaphragm to provide stability for the gable end wall.

Stability Medium: Ceiling Diaphragm: **Below Diaphragm C**

a. Select Values from Table B.20 in Appendix B for building width =32', one story residence, roof slope h = 4, and wind speed = 90 MPH:

W_{CWIN} = 93 plf, W_{CLEE} = -39 plf, W_{CTOT} = 132 plf

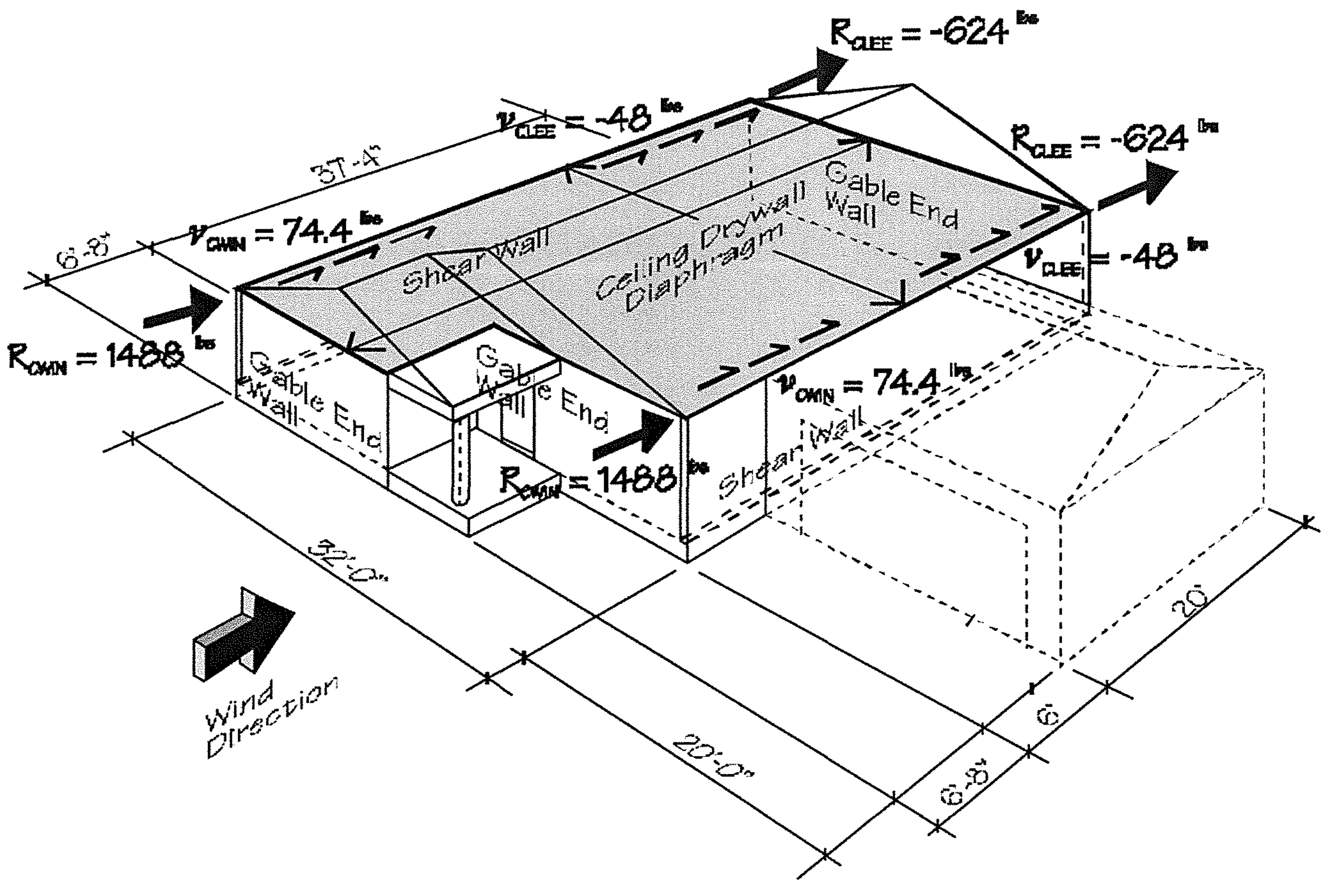


Figure 5.33 - Perspective of One Story Residence
Ceiling Diaphragm
Stability for Gable End Walls

b. Select from Table B.29 in Appendix B for 5/8" gypsum wallboard used on all ceilings:

$$v_{ALLOW} = 115 \text{ plf}$$

c. Calculate gable end wall reaction and diaphragm shear.

Windward Ceiling Diaphragm Reaction: $R_{CWIN} = W_{CWIN} \times W/2$

Reference Figure 5.33 for an illustration of all the involved forces.

$$R_{CWIN} = 93 \text{ plf} \times 32'/2 = \mathbf{1488\ \ lbs.}$$

Ceiling Diaphragm shear: $v_{CWIN} = R_{CWIN} \div L_{ACT}$ plf

ν_{CWIN} = 1488 ÷ 20 = **74.4 plf** < 115 allow. ok

Leeward Ceiling Diaphragm Reaction: $R_{CLEE} = W_{CLEE} \times W/2$

R_{CLEE} = -39 × 32'/2 = **-624 lbs.**

Ceiling Diaphragm shear: $\nu_{CLEE} = R_{CLEE} \div L_{AVAIL}$ (plf)

ν_{CLEE} = -624 ÷ 13' = **-48 plf** < 115 plf allow ok

It is clear that the ceiling plane can provide the diaphragm shear strength for bracing the gable end wall. It will be necessary to connect the diaphragm plane to the wall with metal connectors for the condition when either wall will have suction pulling on it. Use the -39 plf suction value and select a connector capacity, such as Simpson's A34 Framing anchor to connect ceiling joists to the double top plate. The connector shear capacity = 365 lbs.

S = 365 lbs. × 12 ÷ 39 plf = **112** inches, **use 2' o.c. max.** (author's Recommendation as a max. spacing for end wall suction.)

Stability Medium: Ceiling Diaphragm: **Below Diaphragm B**

a. Select Values from Table B.17 in Appendix B for building width =20', one story residence, roof slope h = 4, and wind speed = 90 MPH. Refer to Figure 21 in Chapter 7 in the WMM for a detail of diagonal bracing. Also, refer to Figure 5.34 for an illustration of the diagonal bracing used for the garage.

W_{CWIN} = 85 plf, W_{CLEE} = -27 plf, W_{CTOT} = 112 plf

b. No ceiling exists in the garage, wood diagonal braces will be required.

c. Calculate gable end wall reaction and diaphragm shear.

Windward Ceiling Diaphragm Reaction: $R_{CWIN} = W_{CWIN} \times W/2$

R_{CWIN} = 85 plf × 20'/2 = **850 lbs.**

Leeward Ceiling Diaphragm Reaction: $R_{CLEE} = W_{CLEE} \times W/2$

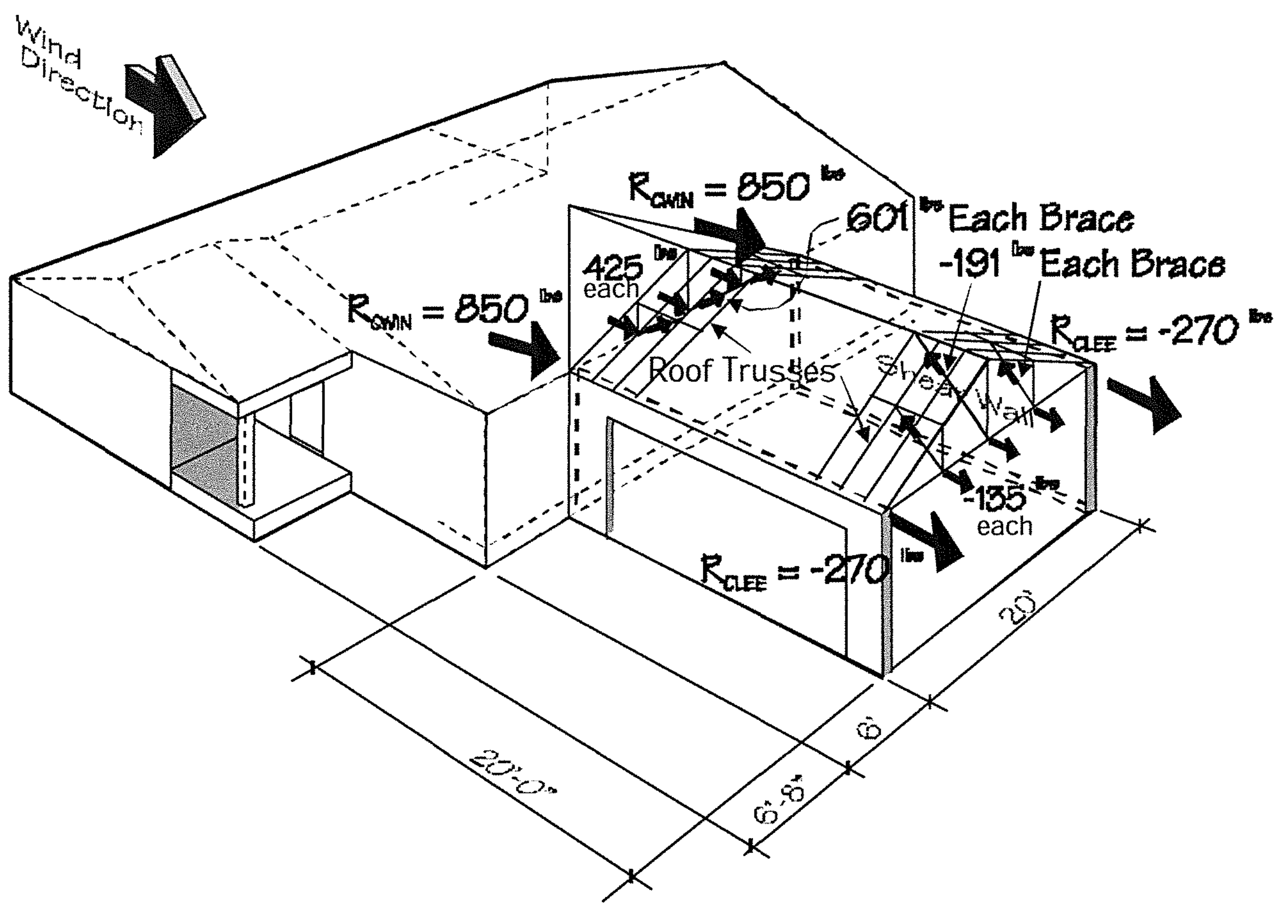


Figure 5.34 - Perspective of one story residence
No Ceiling Diaphragm - Use Diagonal Braces
For Stability of Gable End Wall

$$R_{CLEE} = -27 \times 20'/2 = \mathbf{-270\ lbs.}$$

Windward Horizontal Force per brace:

$$R_{CWINBR} = W_{CWIN} \times W/\text{no. of braces+1}$$

Select 3 braces :

$$R_{CWINBR} = 85 \text{ plf} \times 20'/(3+1) = \mathbf{425\ lbs.}$$

Brace Force at 45°: $F_{45} = R_{CWINBR} \times 1.414$

$$F_{45} = 425 \times 1.414 = \mathbf{601\ lbs.}$$

Leeward Horizontal Force per brace:

$$\mathbf{R_{CLEEBR}} = \mathbf{W_{CLEE}} \times \mathbf{W/no.\ of\ braces{+}1} \qquad (\text{lbs.})$$

$$\mathbf{R_{CLEEBR}} = -27 \text{ plf } \times 20'/4 = \mathbf{135\ lbs.}\ \text{Compression}$$

Brace Force at 45°: $\mathbf{F_{45}} = \mathbf{R_{CLEEBR}} \times 1.414 \qquad (\text{lbs.})$

$$\mathbf{F_{45}} = -135 \times 1.414 = \mathbf{-191\ lbs.}\ \text{tension}$$

Selection of the dimension lumber diagonal brace is based on lumber species and grade, and should follow the *National Design Specification for Wood Construction* for design of wood members.

12. Design critical connections of diaphragm:
    - A. Tension chord splices for diaphragm tension force **T**.
    - B. Diaphragm to shear wall connection to transfer $\mathbf{v}$ in plf.
    - C. Garage Door Framing

    These items are beyond the scope of this Manual

Two Story Residences

The stability system for a two story residence follows the same procedure as used for the one story residence, except for the added diaphragm at the second floor level. Tables in Appendix B for two story residences are similar to those for the one story residence. Figure 5.35 illustrates the wind loads at the roof, second floor and first floor for a wind direction perpendicular to the ridge of a gable roof. Compare this Figure 5.35 with Figure 5.5 at the beginning of the Chapter for the one story residence. Note the similarity.

Notes

The stability system procedure detailed for one story residences is similar in most respects to that for the two story residence; however, there are a few comments to aid the user:

1. Tables B.5 through B.8 in Appendix B are the appropriate Tables for selecting floor and roof diaphragm forces for a wind direction perpendicular to the ridge of a gable roof. Note the second floor force ($\mathbf{F_{F2}}$) and that the Tables have a reminder of "two story" in bold.

2. The maximum shear (v) in (plf) for the second floor diaphragm uses the following equations:

Wind Direction: Perpendicular to ridge of gable roof:

Second Floor Diaphragm Reaction: $\mathbf{R_{F2\perp} = F_{F2} \times L/2}$ (lbs.)

Second Floor Diaphragm shear: $\mathbf{v_{F2\perp} = R_{F2\perp} \div W}$ (plf)

3. Tables B.13 through B.16 in Appendix B are the appropriate Tables for selecting floor and roof diaphragm forces for a wind direction parallel to the ridge of a gable roof. Note the second floor force ($\mathbf{W_{F2}}$) and that the Tables have a reminder of "two story" in bold.

4. The maximum shear (v) in (plf) for the second floor diaphragm uses the following equations:

Wind Direction: Parallel to the ridge of a gable roof:

Second Fl. Diaphragm Reaction: $\mathbf{R_{F2II} = W_{F2} \times W/2}$ (lbs.)

Second Floor Diaphragm shear: $\mathbf{v_{F2II} = R_{F2II} \div L}$ (plf)

5. Tables B.21 through B.24 in Appendix B are the appropriate Tables for selecting the wind force for the gable end wall stability, except that the gable end wall is the second floor wall. Otherwise the equations are same as found in step 14 in the expanded version of the General Stability System Procedure.

Nomenclature

The following acronyms (variables) will be used to relate to the Figures in this Chapter and to the Tables found in Appendix B:

$\mathbf{F}_{\perp}$ = The diaphragm Force at a particular level in pounds per foot of unit length when the *wind direction is perpendicular to the roof's ridge*. $\mathbf{F}_{\perp}$ is the label for a set of Tables in Appendix B. Typical subscripts used in these Tables for the various levels of a residence include:

$\mathbf{F}_{\mathbf{RF}}$ = diaphragm force in plf at the roof/ceiling plane.

$\mathbf{F}_{\mathbf{F2}}$ = diaphragm force in plf at the second floor plane.

$\mathbf{F}_{\mathbf{F1}}$ = diaphragm force in plf at the first floor plane.

$\mathbf{F}_{\mathbf{II}}$ = The diaphragm Force at a particular level in pounds per foot of unit length when the wind direction is parallel to the roof's ridge. $\mathbf{F}_{\mathbf{II}}$ is the label for a set of Tables in Appendix B. Typical subscripts used in these Tables for the various levels of a residence include:

$\mathbf{W}_{\mathbf{RF}}$ = diaphragm force in plf across the building width at the roof/ceiling plane.

$\mathbf{W}_{\mathbf{F2}}$ = diaphragm force in plf across the building width at the second floor plane.

$\mathbf{W}_{\mathbf{F1}}$ = diaphragm force in plf across the building width at the first floor plane.

$\mathbf{C}_{\mathbf{II}}$ = The diaphragm Force at the ceiling level adjacent to the roof in pounds per foot of unit length when the *wind direction is parallel to the roof's ridge*. $\mathbf{C}_{\mathbf{II}}$ is the label for a set of Tables in Appendix B. Typical subscripts used in these Tables for the various ceiling pressures and suctions on the end wall of a residence include:

$\mathbf{w}_{\mathbf{CWIN}}$ = diaphragm pressure across the windward side end wall at the upper ceiling level when the wind direction is parallel to the ridge.

$\mathbf{w}_{\mathbf{CLEE}}$ = diaphragm pressure across the leeward side end wall at the upper ceiling level when the wind direction is parallel to the ridge.

$\mathbf{w}_{\mathbf{CTOT}}$ = diaphragm pressure summed across the windward and leeward side end walls at the ceiling level when the wind direction is parallel to the ridge.

L = Building length for a selected diaphragm for a selected wind direction.

W = Building width for a selected diaphragm for a selected wind direction.

$\mathbf{w_{wind}}$ and $\mathbf{w_{lee}}$ = generic symbols for windward and leeward positive pressure and negative pressure (suction) loads in psf on the walls perpendicular to the wind direction. These wind pressures multiplied over the tributary width of influence constitute a wind load in plf at the top and bottom of a windward or leeward wall. Various subscripts are used to describe particular locations, sums and surfaces receiving the wind pressure (+) or suction (-). Several of these acronyms are:

Wind Direction: *perpendicular* to long dimension of diaphragm

$\mathbf{w_{wind\perp}}$ = linear wind pressure in plane of roof or floor diaphragms from windward wall uniform pressure × tributary width of wall height (plf). Answer will always have a (+) sign.

Roof: $\mathbf{w_{wind\perp rf}}$

2nd Floor: $\mathbf{w_{wind\perp f2}}$

1st Floor: $\mathbf{w_{wind\perp f1}}$

$\mathbf{w_{lee\perp}}$ = linear wind pressure in plane of roof or floor diaphragms from leeward wall uniform suction × tributary width of wall height (plf). Table values will always have a (-) sign.

Roof: $\mathbf{w_{lee\perp rf}}$

2nd Floor: $\mathbf{w_{lee\perp f2}}$

1st Floor: $\mathbf{w_{lee\perp f1}}$

$\mathbf{W_{RH}}$ = horizontal component of wind pressure or suction at mean height of windward slope of a gable roof form. This is a lineal wind load that is assumed to contribute to the linear in-plane roof diaphragm (plf) load. Answer will have a pressure (+) sign for high slope roofs and a suction (-) sign for low slope roofs.

L_{RH} = horizontal component of wind suction at mean height of leeward slope of a gable roof form. This is a lineal wind load that is assumed to contribute to the linear in-plane roof diaphragm (plf) load. Answer will have a suction (-) sign for all roof slopes.

Wind Direction: *parallel* to long dimension of diaphragm:

$W_{windwallrf}$ = linear wind pressure in plane of roof diaphragm from windward wall uniform pressure x tributary width of wall height (plf). Answer will always have a (+) sign.

$W_{leewallrf}$ = linear wind pressure in plane of roof diaphragm from leeward wall uniform suction x tributary width of wall height (plf). Answer will always have a (-) sign.

$W_{windgable}$ = triangular wind pressure in plane of roof diaphragm from windward gable-end wall: uniform pressure × average height of gable triangle (plf). Answer will always have a (+) sign. It is possible that if gable triangle has a portion of it that exist above 15 feet, that a higher wind pressure will exist in an upper (peak) triangle.

$W_{leegable}$ = triangular wind suction in plane of roof diaphragm from leeward gable-end wall: uniform suction x average height of gable triangle (plf). Answer will always have a (-) sign.

$W_{windtotrf} = W_{windwallrf} + W_{windgable}$

$W_{leetotrf} = W_{leewallrf} + W_{leegable}$

$W_{windwallf1}$ = linear wind pressure in plane of floor diaphragm from windward wall uniform pressure × tributary width of wall height (plf). Answer will always have a (+) sign.

$W_{leewallf1}$ = linear wind pressure in plane of floor diaphragm from leeward wall uniform suction × tributary width of wall height (plf). Answer will always have a (-) sign.

v = Maximum unit shear across the width (**W**) of a diaphragm in (plf)

Wind Direction: *Perpendicular* to ridge or long dimension

$v_{RF\perp}$ = maximum diaphragm shear in plf at the roof/ceiling plane.

$v_{F2\perp}$ = maximum diaphragm shear in plf at the second floor plane.

$v_{F1\perp}$ = maximum diaphragm shear in plf at the first floor plane.

Wind Direction: *Parallel* to ridge or long dimension

$v_{RF\parallel}$ = maximum diaphragm shear in plf at the roof/ceiling plane.

$v_{F2\parallel}$ = maximum diaphragm shear in plf at the second floor plane.

$v_{F1\parallel}$ = maximum diaphragm shear in plf at the first floor plane.

R = The reaction at the ends of a simply supported diaphragm model in lbs. This is also the force that is to be resisted by the shear walls at the ends of the diaphragm, since the shear walls are the resistance and therefore the supports.

Wind Direction: *Perpendicular* to ridge or long dimension

$R_{RF\perp}$ = Diaphragm reaction at roof

$R_{F1\perp}$ = Diaphragm reaction at first floor

$R_{F2\perp}$ = Diaphragm reaction at second floor

Wind Direction: *Parallel* to ridge or long dimension

$R_{RF\parallel}$ = Diaphragm reaction at roof

$R_{F1\parallel}$ = Diaphragm reaction at first floor

$R_{F2\parallel}$ = Diaphragm reaction at second floor

R_{anchup} = Net uplift (tension) force at the end of a shear wall segment, requiring anchorage into the structure below the base of the shear wall. Only 2/3ths of the dead load assigned to this shear wall is assumed to counter the wind overturning effect, based on all three model Codes.

Wind Direction: *Perpendicular* to ridge or long dimension

$R_{anchup\perp}$ = Net uplift force.

Wind Direction: *Parallel* to ridge or long dimension

$R_{anchup\parallel}$ = Net uplift force.

Selected References

5.1 "1994 Standard Building Code", *Southern Building Code Congress International*, Article 2310.1.4, Table 2310.1, p. 591.

5.2 "Structural Engineering Design Provisions- Volume 2", *Uniform Building Code by the International Conference of Building Officials*, 1994 Edition, Article 2314, Table 23-I-I, p. 2-897.

5.3 " WFCM Wood Frame Construction Manual for One- and Two-Family Dwellings", *American Forest and Paper Association and American Wood Council*, 1995 SBC, High Wind Edition, p.13.

5.4 "Guide to Understanding WWPA Grade Stamps and Quality Control Identification", *Western Wood Products Association*, Guide#0320/TG/Rev.9-91/17M,pp.1-2.

5.5 "Machine Stress Rated Lumber - Technical Guide", *Western Wood Products Association*, Guide#TG-4/Rev.11/83, pp.1-2.

5.6 "Canadian Dimension Lumber Data Book", *Canadian Wood Council*, Fifth Revised Edition, 1984, pp. 2-9.

5.7 "The BOCA National Building Code/1996", *Building Officials and Code Administrators International, Inc.*, Table 2307.3.1 (1) and (2), 1996, p. 266.

5.8 "1996 Revisions to the 1994 Standard Building Code", *Southern Building Code Congress International*, Article 2310.1.4, Table 2309.3A, p. 589.

5.9 "Residential and Commercial - Design Construction Guide", *APA-The Engineered Wood Association*, 1994 Edition, pp. 23 and 42. Formerly the American Plywood Association.

5.10 Ibid., pp. 5-9.

5.11 "Plywood Design Specification", *American Plywood Association*, Revised April, 1978, pp.14-15.

5.12 "APA Design/Construction Guide - Diaphragms, *American Plywood Association*, 1991, Table 1, p. 6. Modified by Author.

5.13 " WFCM Wood Frame Construction Manual for One- and Two-Family Dwellings", *American Forest and Paper Association and American Wood Council*, 1995 SBC, High Wind Edition, Supplement Table 8, p.232.

5.14 Breyer, Donald E., Design of Wood Structures, Third Edition, McGraw-Hill, Inc., New York, 1993, pp.449-450.

5.15 " WFCM Wood Frame Construction Manual for One- and Two-Family Dwellings", *American Forest and Paper Association and American Wood Council*, 1995 SBC, High Wind Edition, Table 3B, p.191. Modified by Author.

5.16 "Uniform Building Code", *International Conference of Building Officials*, 1991 Edition, pp.774-775.

5.17 "1996 Revisions to the 1994 Standard Building Code", *Southern Building Code Congress International*, Article 2313 Wind Provisions, Article 2313, p. 604.

5.18 " Commentary to the WFCM Wood Frame Construction Manual for One- and Two-Family Dwellings", *American Forest and Paper Association and American Wood Council*, 1995 SBC, High Wind Edition, p.71.

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Appendix A – Uplift Chain Design Tables

Introduction

The author has developed the following tables and charts unless noted otherwise.

Tension Chain – Uplift Tables

The following tables and charts are derived based on the concepts cited in Chapter 2, dealing with direct tension uplift.

Nomenclature - Symbols

See page 182-183 of Chapter 4 for the definitions of nomenclature and symbols used in this Appendix A.

Examples

Examples of the use of these Tables are found in Chapter 4, pages 145-181.

Table A.1

Structural Panel Axial Tension Allowable Capacity [1,2]

t	APA Span rating	T_{all} (lbs./ft)[3]	
		Parallel[4]	Perpendicular[4]
15/32, 1/2, 3/8	24/0	3072	1317
15/32, 1/2, 19/32, 5/8	32/16	3439	1649
19/32, 5/8, 23/32, 3/4	40/20	3830	2115
23/32, 3/4, 7/8	48/24	5346	2567

[1] OSB board or plywood are acceptable

[2] Panel tension capacities derived from APA Technote N375-*Design Capacities of APA Performance Rated Structural-use Panels*, July 1988, and *Plywood Design Specification*, August, 1986. Smaller allowable tension values of both documents used in above table.

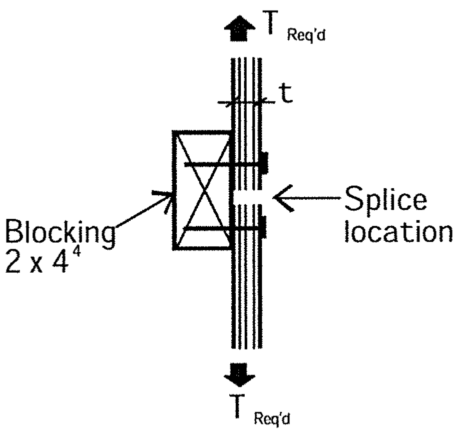
[3] Increased by 1.33 for wind (author's choice),NDS 1997 permits increase of 1.6.

[4] Parallel or perpendicular to grain refers to the orientation of the face grain with respect to the tension uplift direction.

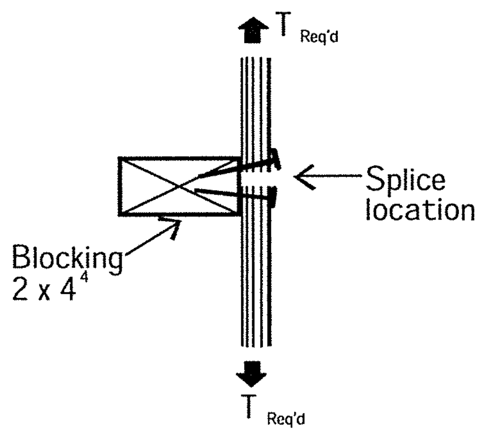
Table A.2

Lateral Allowable Capacity per Nail (lbs.)[1]

Nail Size & Length[3]	Type[2]	Dia.	't' of Panel (inches) 3/8	15/32	1/2	19/32	5/8	23/32	3/4
6d x 2"	C	0.113	80.9	86.1	88.2	95.1	97.6	99.8	100.0
	B	0.099	64.8	70.1	72.1	78.8	81.2	84.7	84.7
8d x 2-1/2"	C	0.131	99.4	104.1	106.1	112.9	115.4	123.6	126.5
	B	0.113	80.9	86.1	88.2	95.1	97.6	105.6	108.5
10d x 3"	C	0.148	129.0	133.5	135.9	142.5	145.2	154.1	157.3
	B	0.128	100.1	105.1	107.2	114.3	116.9	125.4	128.4



For 6d and 8d Nails



For 10d Nails

[1] Allowable capacities based on NDS 1997 criteria, and duration of load factor 1.33 for wind (author's recommendation) and diaphragm factor 1.1. Orientation of panel does not influence nail capacity. A wind load factor Of 1.6 is permitted by the ASCE - -93.

[2] Common nail = C, Box nail = B.

[3] Nails extend beyond blocking for some 8d and t combinations. 10d nails should use blocking orientation horizontal.

[4] Dimension Lumber species shall be those with a specific gravity of 0.5 or higher.

R_{up}

20′

Table A.3

Uplift Requirements for a Single Story Residence

SPAN (L)	ROOF DL	OVERHANG MAX.	LENGTH RANGE
20′	15 PSF	2′	40′ TO 80′

ROOF SLOPE h	DIRECT UPLIFT FORCE (R_{up}) POUNDS / FOOT OF BUILDING LENGTH			
	80 MPH	90 MPH	100 MPH	110 MPH
3:12 TO 6:12	198	280	370	470
7:12 TO 12:12	136	202	275	356
	OVERTURNING			
3:12	106	162	225	295
4:12	84	135	191	255
5:12	45	86	132	182
6:12	1	30	64	100
7:12	*	*	4	28
8:12 TO 12:12	*	*	*	*

CONNECTION SPACING (IN)	12	16	19.2	24	48
MULTIPLIER	1.0	~~13.3~~	1.6	2.0	4

* NO UPLIFT EXISTS

1.3

R_{up} 24′

Table A.4
Uplift Requirements for a Single Story Residence

SPAN (L)	ROOF DL	OVERHANG MAX.	LENGTH RANGE
24′	15 PSF	2′	40′ TO 80′

ROOF SLOPE h	DIRECT UPLIFT FORCE (R_{up}) POUNDS / FOOT OF BUILDING LENGTH			
	80 MPH	90 MPH	100 MPH	110 MPH
3:12 TO 6:12	233	324	425	536
7:12 TO 12:12	172	249	333	431

	OVERTURNING			
3:12	134	198	270	349
4:12	109	168	232	304
5:12	66	113	165	223
6:12	16	51	90	132
7:12	*	*	22	51
8:12 TO 12:12	*	*	*	*

CONNECTION SPACING (IN)	12	16	19.2	24	48
MULTIPLIER	1.0	13.3	1.6	2.0	4

* NO UPLIFT EXISTS

R_{up}
28′

Table A.5

Uplift Requirements for a Single Story Residence

SPAN (L)	ROOF DL	OVERHANG MAX.	LENGTH RANGE
28′	15 PSF	2′	40′ TO 80′

ROOF SLOPE h	DIRECT UPLIFT FORCE (R_{up}) POUNDS / FOOT OF BUILDING LENGTH			
	80 MPH	90 MPH	100 MPH	110 MPH
3:12 TO 6:12	252	353	463	587
7:12 TO 12:12	193	279	375	481

	OVERTURNING			
3:12	147	218	299	388
4:12	*	*	4.5	31
5:12	*	*	21	52
6:12	*	*	24	54
7:12	*	*	*	57
8:12 TO 12:12	*	*	*	*

CONNECTION SPACING (IN)	12	16	19.2	24	48
MULTIPLIER	1.0	13.3	1.6	2.0	4

* NO UPLIFT EXISTS

R_{up}
32′

Table A.6

Uplift Requirements for a Single Story Residence

SPAN (L)	ROOF DL	OVERHANG MAX.	LENGTH RANGE
32′	15 PSF	2′	40′ TO 80′

ROOF SLOPE h	DIRECT UPLIFT FORCE (R_{up}) POUNDS / FOOT OF BUILDING LENGTH			
	80 MPH	90 MPH	100 MPH	110 MPH
3:12 TO 6:12	275	386	509	645
7:12 TO 12:12	214	311	418	537

	OVERTURNING			
3:12	159	238	328	426
4:12	42	90	145	205
5:12	*	*	22	57
6:12	*	*	*	*
7:12	*	*	*	*
8:12 TO 12:12	*	*	*	*

CONNECTION SPACING (IN)	12	16	19.2	24	48
MULTIPLIER	1.0	13.3	1.6	2.0	4

* NO UPLIFT EXISTS

R_{up2}
20′

Table A.7

Uplift Requirements for Top Story of a Two Story Residence

SPAN (L)	ROOF DL	OVERHANG MAX.	LENGTH RANGE
20′	15 PSF	2′	40-80′ LONG

ROOF SLOPE h	DIRECT UPLIFT FORCE (R_{up2}) POUNDS / FOOT OF BUILDING LENGTH			
	80 MPH	90 MPH	100 MPH	110 MPH
3:12 TO 6:12	224	309	409	515
7:12 TO 12:12	162	235	316	406

	OVERTURNING			
3:12	118	177	243	317
4:12	97	150	211	277
5:12	56	98	146	204
6:12	9	41	76	116
7:12	*	*	12	39
8:12	*	*	3	27
9:12 to 12:12	*	*	*	*

CONNECTION SPACING (IN)	12	16	19.2	24	48
MULTIPLIER	1.0	13.3	1.6	2.0	4

* NO UPLIFT EXISTS

R_{up2}
24′

Table A.8

Uplift Requirements for Top Story of a Two Story Residence

SPAN (L)	ROOF DL	OVERHANG MAX.	LENGTH RANGE
24′	15 PSF	2′	40-80′ LONG

ROOF SLOPE h	DIRECT UPLIFT FORCE (R_{up2}) POUNDS / FOOT OF BUILDING LENGTH 80 MPH	90 MPH	100 MPH	110 MPH
3:12 TO 6:12	242	338	449	572
7:12 TO 12:12	186	270	365	471

ROOF SLOPE	OVERTURNING 80 MPH	90 MPH	100 MPH	110 MPH
3:12	129	196	271	355
4:12	106	166	235	310
5:12	59	109	166	229
6:12	7	44	84	130
7:12	*	*	15	47
8:12 TO 12:12	*	*	*	*

CONNECTION SPACING (IN)	12	16	19.2	24	48
MULTIPLIER	1.0	13.3	1.6	2.0	4

* NO UPLIFT EXISTS

R_{up2} 28′

Table A.9

Uplift Requirements for Top Story of a Two Story Residence

SPAN (L)	ROOF DL	OVERHANG MAX.	LENGTH RANGE
28′	15 PSF	2′	40-80′ LONG

ROOF SLOPE h	DIRECT UPLIFT FORCE (R_{up2}) POUNDS / FOOT OF BUILDING LENGTH			
	80 MPH	90 MPH	100 MPH	110 MPH
3:12 TO 6:12	263	372	493	629
7:12 TO 12:12	209	307	417	539

	OVERTURNING			
3:12	142	216	303	395
4:12	114	181	259	344
5:12	63	119	182	254
6:12	7	49	96	148
7:12	*	*	14	50
8:12 TO 12:12	*	*	*	*

CONNECTION SPACING (IN)	12	16	19.2	24	48
MULTIPLIER	1.0	13.3	1.6	2.0	4

* NO UPLIFT EXISTS

R_{up2}

32′

Table A.10

Uplift Requirements for Top Story of a Two Story Residence

SPAN (L)	ROOF DL	OVERHANG MAX.	LENGTH RANGE
32′	15 PSF	2′	40-80′ LONG

ROOF SLOPE h	DIRECT UPLIFT FORCE (R_{up}) POUNDS / FOOT OF BUILDING LENGTH			
	80 MPH	90 MPH	100 MPH	110 MPH
3:12 TO 6:12	315	438	573	724
7:12 TO 12:12	260	373	498	636

	OVERTURNING			
3:12	178	262	357	459
4:12	158	238	327	411
5:12	96	160	229	308
6:12	30	76	128	185
7:12	*	7	43	52
8:12 TO 12:12	*	*	*	*

CONNECTION SPACING (IN)	12	16	19.2	24	48
MULTIPLIER	1.0	13.3	1.6	2.0	4

* NO UPLIFT EXISTS

DL_{fl}

Table A.11
Floor Framing Dead Load

1st or 2nd Floor Dead* Load (lb/ft)						
Spacing \ span	6′	8′	10′	12′	14′	16′
12″	45.3	60.3	75.4	90.4	105.6	120.6
16″	41.7	55.6	69.5	83.4	97.4	111.3
19.2″	40.0	53.3	66.6	80.0	93.3	106.6
24″	38.2	51.0	63.7	76.5	89.2	101.9
48″	34.7	46.3	57.9	69.5	81.0	92.6

*Assumes2/3rds of DL available to resist uplift

Assumes:

2 x 10 joists:	3.5 plf * 12/spacing
3/4″ sub-Floor:	2.3 psf
floor finish carpet:	1.0 psf
5/8″ drywall ceiling:	2.5 psf
mech/elect:	2.0 psf

Wall$_{DL}$

Table A.12

Wood Stud Wall Weights [1, 2, 3]

	Studs		Interior Partition		Exterior Wall		Exterior Wall		Gable End	
			½ Drywall Each Face		½ Drywall Each Face		1" Gyp Plast+1/2" Stru.Panel		Structural Panel	
Spacing	2 x 4 Bare	2 x 6 Bare	2 x 4	2 x 6	2 x 4	2 x 6	2 x 4	2 x 6	2 x 4	2 x 6
12"	1.2 / 9.6	1.9 / 15.2	3.9 / 31.2	4.6 / 36.8	3.4 / 27.2	4.1 / 32.8	7.4 / 59.2	8.1 / 64.8	2.1	2.8
16"	1.0 / 8.0	1.6 / 12.8	3.7 / 29.6	4.3 / 34.4	3.2 / 25.6	3.8 / 30.4	7.2 / 57.6	7.8 / 62.4	1.7	2.3
24"	0.8 / 6.4	1.2 / 9.6	3.5 / 28.0	3.9 / 31.2	3.0 / 24.0	3.4 / 27.2	7.0 / 56.0	7.4 / 59.2	1.2	1.63

1. A total of 3 plates (2 @top, 1 @ bottom) have been included and spread over an 8' wall height.

2. Lower values are psf of vertical surface.
   Upper values are plf of 8' high wall.

3. All tabled values are 2/3rds of actual dead loads, for uplift consideration.

4. Actual material weights:

   structural panel: 1/2" = 1.3^{PSF} per side

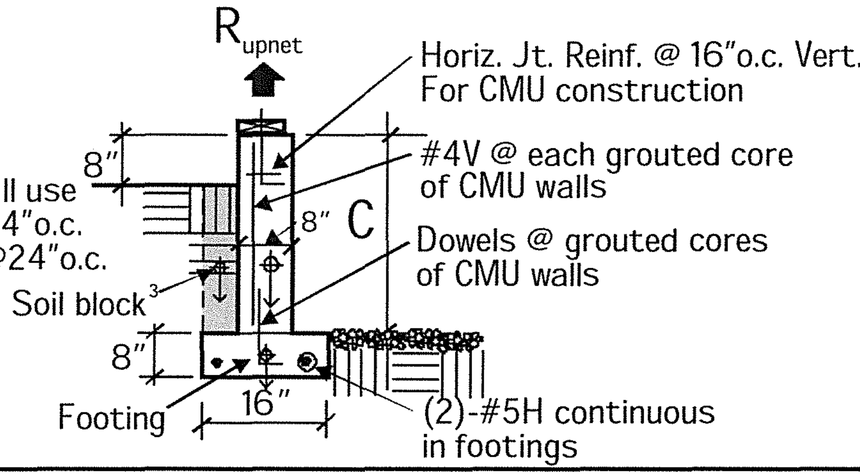
   1" gypsum plaster on wood lath = 8^{PSF} per side

   1/2" gypsum board (drywall) = 2.0^{PSF}

Table A.13

Crawl Space Foundation Wall Uplift Resistance[1]

Note:
C.I.P. Conc. Walls shall use dowels as shown @24"o.c. With wall reinf. #4V@24"o.c. & #4H@16"o.c.



Lbs./Ft. Of Building Length[2]						
C	8" CMU (NLWT) Grouted @				8" CMU NLWT grouted solid & Vert.Reinf.	8" C.I.P. Conc.
	48"o.c.	40"o.c.	32"o.c.	24"o.c.		
2'-0"	198	199	202	206	237	258
2'-8"	240	242	246	251	293	320
3'-4"	282	285	289	295	347	382
4'-0"	325	327	333	341	402	445
4'-8"	367	370	376	386	457	507

[1]Potential resistance to uplift (withdrawal) is the maximum uplift resistance which can be provided by the foundations shown. It is computed by adding the weights of the crawlspace wall material and soil over the top of the footing, plus the footing weight . To fully provide this potential resistance, adequate connections to the footing and the superstructure must be provided.
Material weights used: concrete(NLWT) = 150 pcf; 8" concrete block (NLWT) and grouted solid = 84 psf; 8" concrete block (NLWT) grouted at 48" o.c. = 55 psf; grouted at 40" o.c. = 56 psf; grouted at 32" o.c. = 58 psf: grouted at 24" o.c. = 61 psf: soil = 120 pcf; grout wt. Assumed = 140 pcf.

[2]Tabled dead load resistance to uplift represents 2/3rds of the actual weights of the materials shown in item 1.

[3]Soil rectangle above footing lip assumes sandly soil. A wedge at approximately 30 degrees might be considered if clay were the soil type.

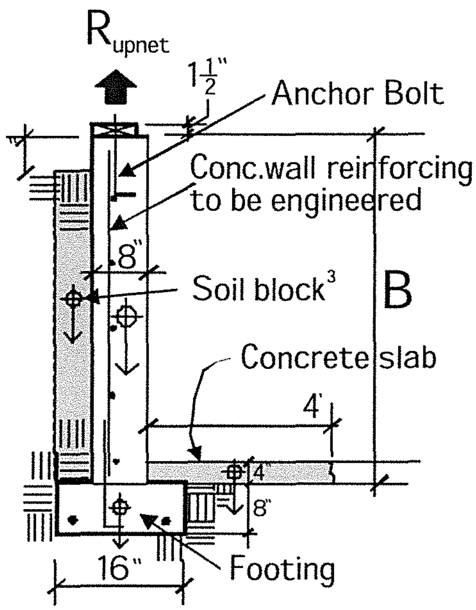


Table A.14

Basement Foundation Wall Uplift Resistance[1]

Lbs./Ft. of Building Length[2]						
B		8" CMU-Grouted@		8" CMU	8" C.I.P.
		4'o.c.	2'o.c.	Solid Grtd	Conc.
8'	12 courses	469	501	623	709
8'-8"	13 courses	498	533	666	759
9'-4"	14 courses	527	565	708	808
10'-0"	15 courses	556	596	749	856

<u>Note:</u>

8" CMU wall shall have #4V at each grouted core.
Solid grouted wall shall have #4V at 32" o.c.
All CMU walls shall have horiz. jt. Reinf. At 16" o.c. Vertically

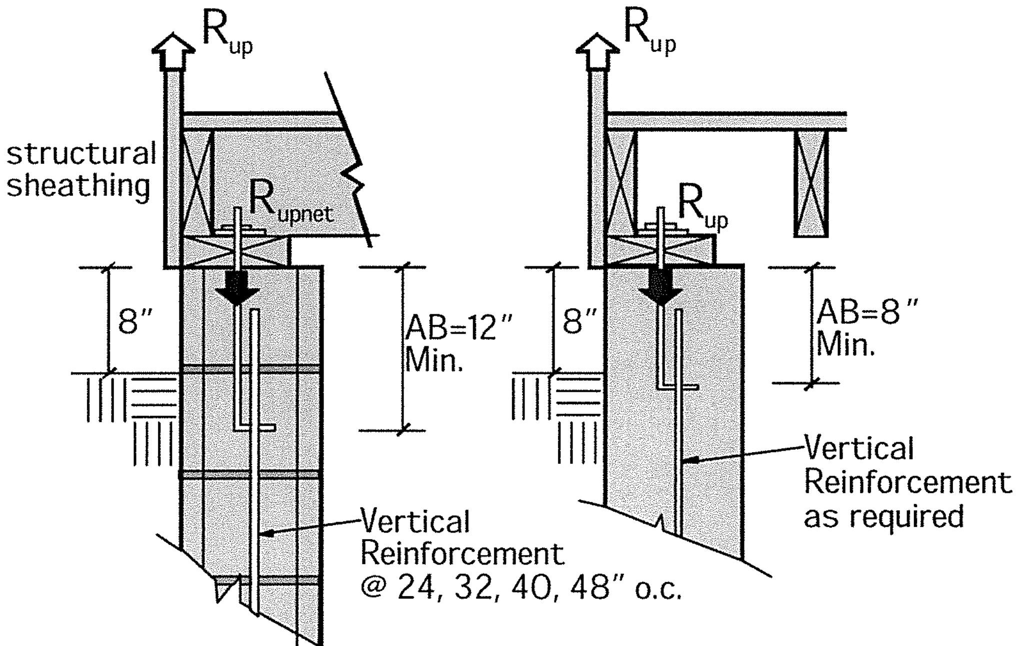
[1]Potential resistance to uplift (withdrawal) is the maximum uplift resistance which can be provided by the foundations shown. It is computed by adding the weights of the basement wall material and soil over the top of the footing, plus the footing weight and an assumed 4 foot length of concrete slab. To fully provide this potential resistance, adequate connections to the footing and the superstructure must be provided.

Material weights used: concrete(NLWT) = 150 pcf; 8" concrete block (NLWT) and grouted solid = 84 psf; 8" concrete block (NLWT) and grouted at 4' o.c. = 55 psf; 8" concrete block (NLWT) and grouted at 2' o.c. = 61 psf; soil = 120 pcf; grout wt. Assumed = 140 pcf.

[2]Tabled dead load resistance to uplift represents 2/3rds of the actual weights of the materials shown in item 1.

[3]Soil rectangle above footing lip assumes sandly soil. A wedge at approximately 30 degrees might be considered if clay were the soil type.

Table A.15
Vertical Anchor Capacity for Foundation Walls[1]



Floor Framing Perpendicular to Wall

Floor Framing Parallel to Wall

Vertical Capacity[4] lbs. per Spacing			Required Anchorage[2]		
Standard Washer	Over-sized Washer	Square Washer	Anchor Bolt	Rebar[3]	Spacing[4]
146	239	824	1/2"	#4	6'-0" max.
164	270	927	↓	↓	5'-4"
187	307	1060	↓	↓	4'-8"
218	359	1236	↓	↓	4'-0"
262	431	1483	↓	↓	3'-4"
327	538	1852	↓	↓	2'-8"
437	718	2472	↓	↓	2'-0"

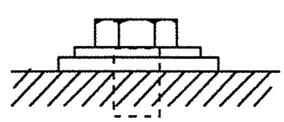
[1] Values are based on vertical capacity per foot of wall.

[2] Assuming 1-1/2" thick sill plate, 3/4" edge distance for wood or composite nailer plates or 20 diameter end distance for plywood sheathing; APA rated, properly seasoned wood; Group III woods, not permanently loaded, and a 25% length of bearing factor increase for washers, and 1.0 factor for square washers.

[3] It is assumed that a reinforcing bar of the same diameter and spacing as the anchor is adequately embedded in the footing and lapped with the anchor.

[4] Spacing and capacity is based on allowable compression of wood perpendicular to grain for $F_c = 565$ psi and washer as defined below:

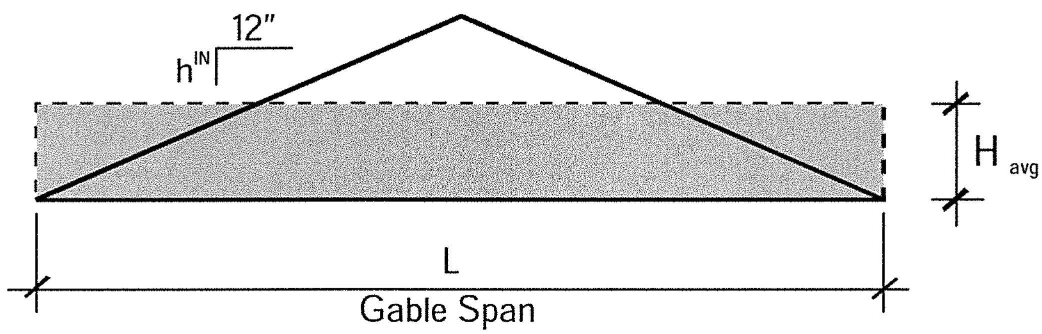
Standard washer: 1-3/8" O.D. and 0.5625" I.D. washer (for 1/2" ϕ bolt)
Over-sized washer: 1-3/4" O.D. and 0.6875" I.D. washer (for 5/8" ϕ bolt) placed under the standard washer



H_{avg}

Table A.16

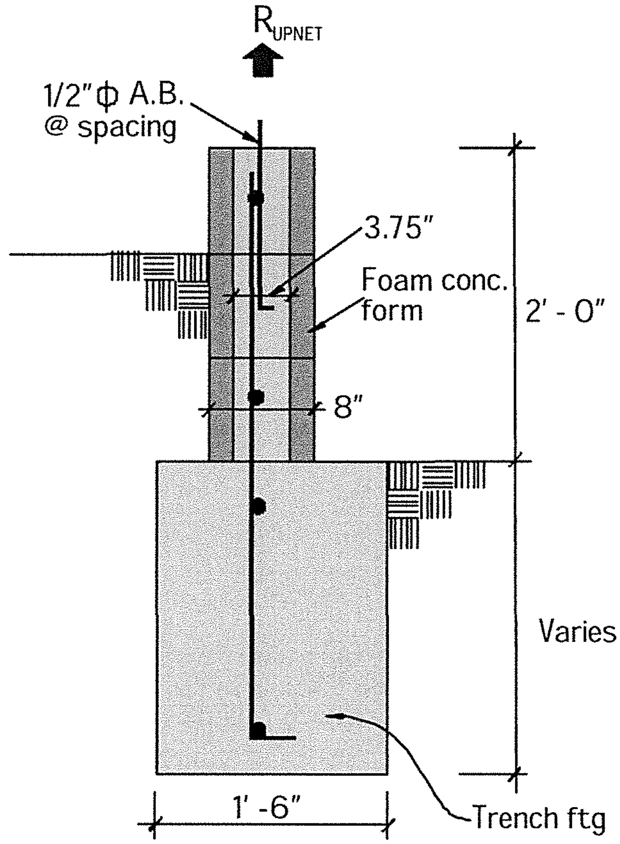
Gable Ends - Average Height



	H_{avg} in Feet			
h \ l	20'	24'	28'	32'
3	1.3	1.5	1.8	2.0
4	1.7	2.0	2.3	2.7
5	2.1	2.5	2.9	3.3
6	2.5	3.0	3.5	4.0
7	2.9	3.5	4.1	4.7
8	3.3	4.0	4.7	5.3
9	3.8	4.5	5.3	6.0
10	4.2	5.0	5.8	6.7
12	5.0	6.0	7.0	8.0

Table A.17

Uplift Resistance of Trench Footing/Crawl Space



Trench-ftg Depth	Uplift Resistance in LBS						
	Anchor Bolt Spacing						
	1'-0"	2'-0"	2'-8"	3'-4"	4'-0"	4'-8"	6'-0"
1'-0"	212.5 LB	425	567	708	850	992	1275
1'-6"	287.5	575	767	958	1150	1342	1725
2'-0"	362.5	725	968	1207	1450	1693	2175
2'-6"	437.5	875	1167	1458	1750	2042	2625
3'-0"	512.5	1025	1368	1707	2050	2393	3075
3'-6"	587.5	1175	1567	1958	2350	2742	3525

Appendix B – Stability System Design Tables

Introduction

The author has developed the following tables and charts unless noted otherwise.

Stability System – Diaphragm and Shear Wall Tables

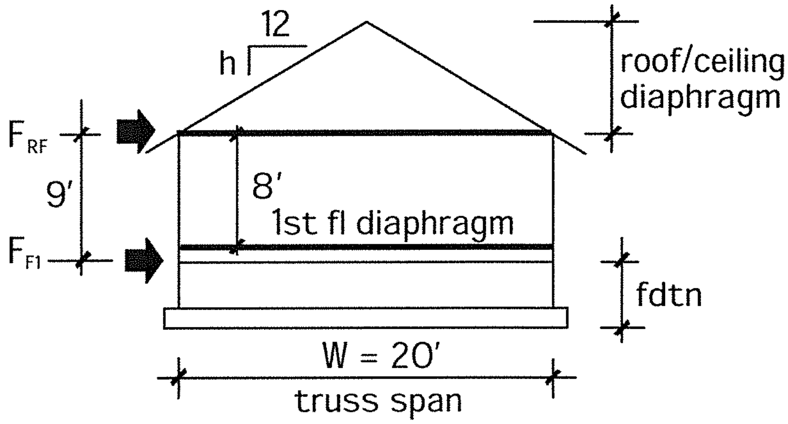
The following tables and charts are derived based on the concepts cited in Chapter 3 and Chapter 5, dealing with the lateral load resisting system's stability.

Nomenclature - Symbols

See page 283-287 of Chapter 5 for the definitions of nomenclature and symbols used in this Appendix B.

Examples

An example of the use of these Tables is found in Chapter 5, pages 233-281.



$F_{\perp}$ 20’

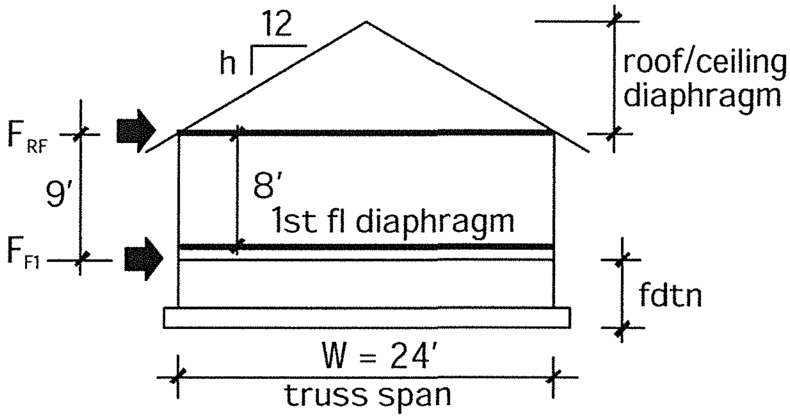
1 STORY

Table B.1

Wind Perpendicular to Roof Ridge

Roof Slope	Lateral Wind Load to Diaphragms LB/FT of Length							
	Fastest Mile Wind Speed (MPH)							
	80		90		100		110	
h	F_{RF}	F_{F1}	F_{RF}	F_{F1}	F_{RF}	F_{F1}	F_{RF}	F_{F1}
3	79	191	109	261	124	299	151	364
4	83	195	105	248	130	305	157	370
5	98	210	124	266	152	328	185	398
6	122	234	155	297	190	366	231	444
7	151	263	192	334	237	413	287	500
8	184	296	233	376	288	463	348	561
9	222	334	281	424	347	523	421	634
10	261	373	331	474	407	583	494	707
12	318	430	403	546	497	672	602	815

1. Wind loads are based on ASCE:7-93 for “Main Wind Force Resistance System (MWFRS).”
2. For 10” floor to ceiling height, multiply above values by 1.25.
3. See Figure 1 in Chapter 5 for method of calculation.



$F_{\perp}$ 24’

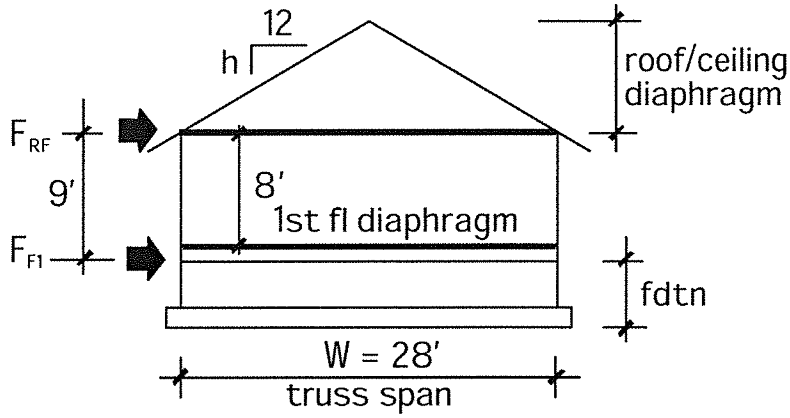
1 STORY

Table B.2

Wind Perpendicular to Roof Ridge

Roof Slope	Lateral Wind Load to Diaphragms LB/FT of Length							
	Fastest Mile Wind Speed (MPH)							
	80		90		100		110	
h	F_{RF}	F_{F1}	F_{RF}	F_{F1}	F_{RF}	F_{F1}	F_{RF}	F_{F1}
3	111	223	141	283	173	349	210	423
4	82	194	104	246	128	303	155	368
5	99	211	125	268	154	330	187	400
6	127	239	162	304	199	347	241	454
7	162	274	205	347	253	429	307	520
8	200	312	253	396	312	488	378	591
9	244	356	309	452	382	557	462	675
10	289	401	367	510	452	627	548	761
12	356	468	451	594	556	732	674	887

1. Wind loads are based on ASCE:7-93 for “Main Wind Force Resistance System (MWFRS).”
2. For 10” floor to ceiling height, multiply above values by 1.25.
3. See Figure 1 in Chapter 5 for method of calculation.



F⊥ 28'

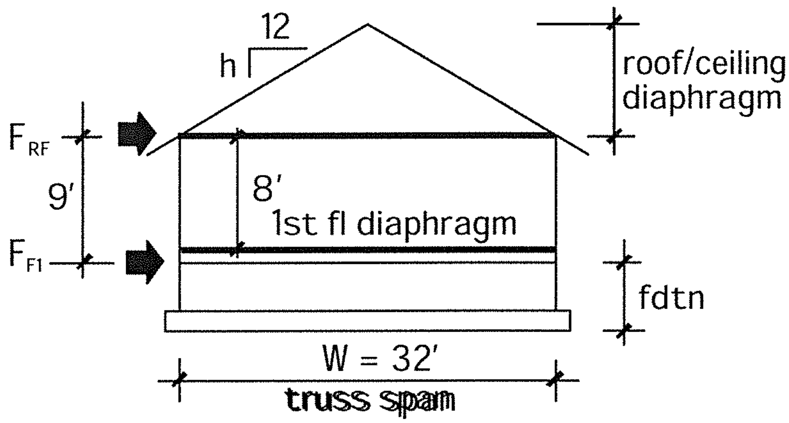
1 STORY

Table B.3

Wind Perpendicular to Roof Ridge

Roof Slope	Lateral Wind Load to Diaphragms LB/FT of Length — Fastest Mile Wind Speed (MPH)							
	80		90		100		110	
h	F_{RF}	F_{F1}	F_{RF}	F_{F1}	F_{RF}	F_{F1}	F_{RF}	F_{F1}
3	114	226	144	287	178	354	216	429
4	140	252	178	321	219	395	266	479
5	148	260	188	331	232	408	282	495
6	178	290	226	369	279	454	304	517
7	171	284	218	360	269	445	327	539
8	215	327	273	415	337	512	349	562
9	266	378	337	480	416	592	504	717
10	318	430	403	546	498	673	604	817
12	394	506	500	642	626	803	753	966

1. Wind loads are based on ASCE:7-93 for "Main Wind Force Resistance System (MWFRS)."
2. For 10" floor to ceiling height, multiply above values by 1.25.
3. See Figure 1 in Chapter 5 for method of calculation.



F⊥ 32'

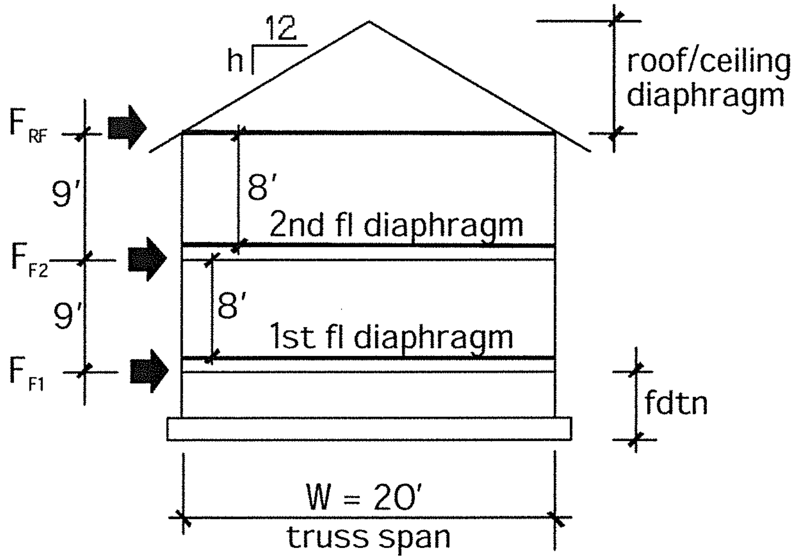
1 STORY

Table B.4

Wind Perpendicular to Roof Ridge

Roof Slope	Lateral Wind Load to Diaphragms LB/FT of Length							
	Fastest Mile Wind Speed (MPH)							
	80		90		100		110	
h	F_{RF}	F_{F1}	F_{RF}	F_{F1}	F_{RF}	F_{F1}	F_{RF}	F_{F1}
3	160	272	203	345	250	425	303	516
4	131	213	166	309	205	381	249	462
5	156	268	197	340	244	419	296	509
6	190	302	240	383	296	472	359	572
7	227	339	288	431	356	531	432	645
8	268	380	340	482	419	594	460	673
9	288	400	365	508	451	626	540	753
10	351	464	443	586	546	723	663	877
12	443	556	561	705	691	866	839	1053

1. Wind loads are based on ASCE:7-93 for "Main Wind Force Resistance System (MWFRS)."
2. For 10" floor to ceiling height, multiply above values by 1.25.
3. See Figure 1 in Chapter 5 for method of calculation.



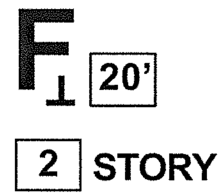


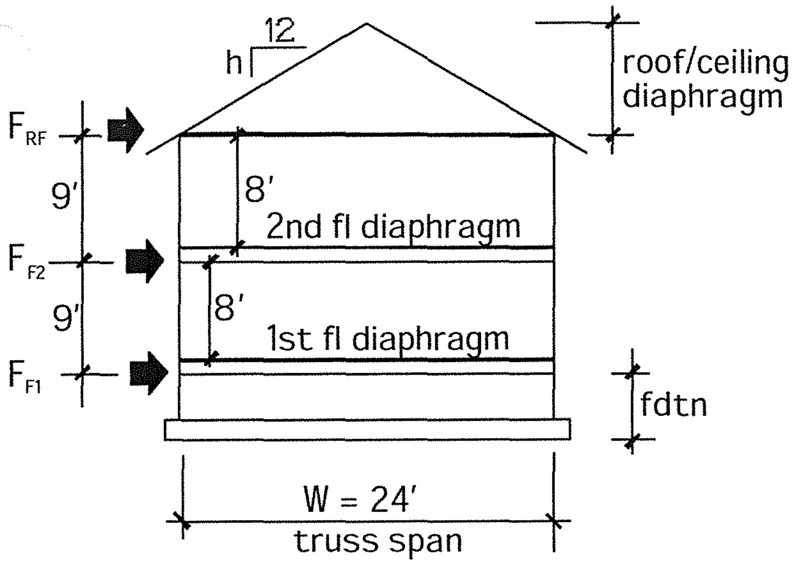
Table B.5

Wind Perpendicular to Roof Ridge

	Lateral Wind Load to Diaphragms LB/FT of Length											
Roof Slope	Fastest Mile Wind Speed (MPH)											
	80			90			100			110		
h	F_{RF}	F_{F2}	F_{F1}	F_{RF}	F_{F2}	F_{F1}	F_{RF}	F_{F2}	F_{F1}	F_{RF}	F_{F2}	F_{F1}
3	95	304	418	120	384	528	148	474	652	179	573	788
4	98	306	419	124	387	530	153	479	657	185	578	793
5	113	321	435	144	407	550	178	503	681	214	607	822
6	141	349	463	177	441	585	219	545	723	266	661	877
7	172	381	495	217	481	625	268	594	773	326	721	937
8	189	399	514	239	505	650	296	624	803	358	754	971
9	227	436	551	287	552	697	354	682	862	428	825	1042
10	268	476	590	340	605	750	418	744	923	507	903	1119
12	357	565	679	396	661	806	487	813	992	590	986	1202

1. Wind loads are based on ASCE:7-93 for "Main Wind Force Resistance System (MWFRS)."

2. For 10" floor to ceiling height, multiply above values by 1.25.



$F_{\perp}$ 24'

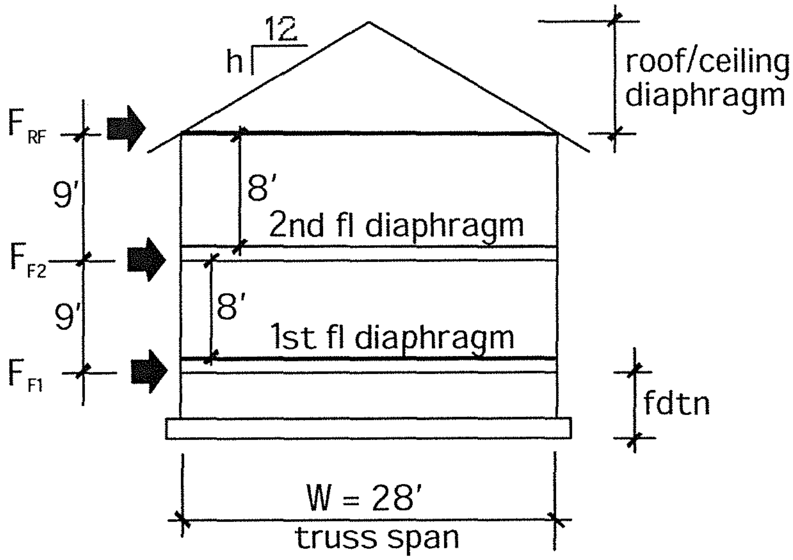
2 STORY

Table B.6

Wind Perpendicular to Roof Ridge

Roof Slope	Lateral Wind Load to Diaphragms LB/FT of Length											
	Fastest Mile Wind Speed (MPH)											
	80			90			100			110		
h	F_{RF}	F_{F2}	F_{F1}	F_{RF}	F_{F2}	F_{F1}	F_{RF}	F_{F2}	F_{F1}	F_{RF}	F_{F2}	F_{F1}
3	93	302	416	118	382	526	145	471	649	175	569	785
4	96	304	417	123	385	529	151	477	655	183	576	791
5	116	324	438	146	410	554	180	507	685	218	613	829
6	146	355	469	184	447	592	228	554	732	277	672	888
7	185	394	509	234	500	645	289	617	796	349	745	962
8	225	434	548	283	547	691	350	675	853	423	817	1033
9	274	482	596	348	613	758	428	754	932	519	915	1132
10	324	533	647	411	676	821	505	831	1009	613	1009	1225
12	404	615	731	512	778	924	632	962	1142	763	1161	1379

1. Wind loads are based on ASCE:7-93 for “Main Wind Force Resistance System (MWFRS).”
2. For 10” floor to ceiling height, multiply above values by 1.25.



F⊥ 28'

2 STORY

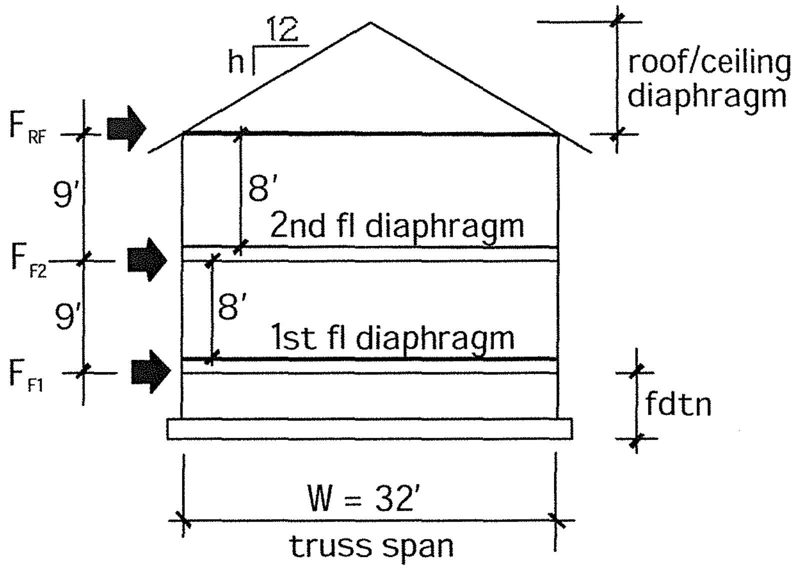
Table B.7

Wind Perpendicular to Roof Ridge

Roof Slope	Lateral Wind Load to Diaphragms LB/FT of Length											
	Fastest Mile Wind Speed (MPH)											
	80			90			100			110		
h	F_{RF}	F_{F2}	F_{F1}	F_{RF}	F_{F2}	F_{F1}	F_{RF}	F_{F2}	F_{F1}	F_{RF}	F_{F2}	F_{F1}
3	91	298	412	115	378	521	142	467	645	171	564	779
4	95	303	416	121	384	527	149	475	653	180	573	788
5	117	326	440	148	411	555	182	509	687	221	616	832
6	153	363	478	194	459	604	239	567	746	289	685	902
7	194	402	515	246	509	653	304	629	807	367	761	977
8	242	451	565	308	573	718	378	705	883	459	855	1072
9	302	511	626	381	646	792	471	799	978	569	966	1183
10	358	568	682	453	719	865	559	887	1067	678	1075	1272
12	446	656	771	567	834	980	699	1027	1206	844	1242	1459

1. Wind loads are based on ASCE:7-93 for "Main Wind Force Resistance System (MWFRS)."

2. For 10" floor to ceiling height, multiply above values by 1.25.



$F_{\perp}$ 32'

2 STORY

Table B.8

Wind Perpendicular to Roof Ridge

Roof Slope	Lateral Wind Load to Diaphragms LB/FT of Length											
	Fastest Mile Wind Speed (MPH)											
	80			90			100			110		
h	F_{RF}	F_{F2}	F_{F1}	F_{RF}	F_{F2}	F_{F1}	F_{RF}	F_{F2}	F_{F1}	F_{RF}	F_{F2}	F_{F1}
3	89	296	410	113	375	519	139	464	642	168	561	776
4	91	300	414	115	379	523	142	469	647	179	574	790
5	119	329	443	10	416	561	186	514	694	225	621	838
6	158	365	479	200	463	607	246	571	749	297	691	907
7	207	415	529	263	528	673	323	649	827	391	787	1004
8	262	472	586	331	597	743	410	738	917	496	893	1110
9	329	540	655	416	683	829	513	842	1023	620	1019	1237
10	391	601	715	494	759	904	611	938	1117	737	1133	1349
12	494	706	821	625	893	1039	772	1102	1282	935	1334	1552

1. Wind loads are based on ASCE:7-93 for “Main Wind Force Resistance System (MWFRS).”
2. For 10” floor to ceiling height, multiply above values by 1.25.

F‖ 20'

1 STORY

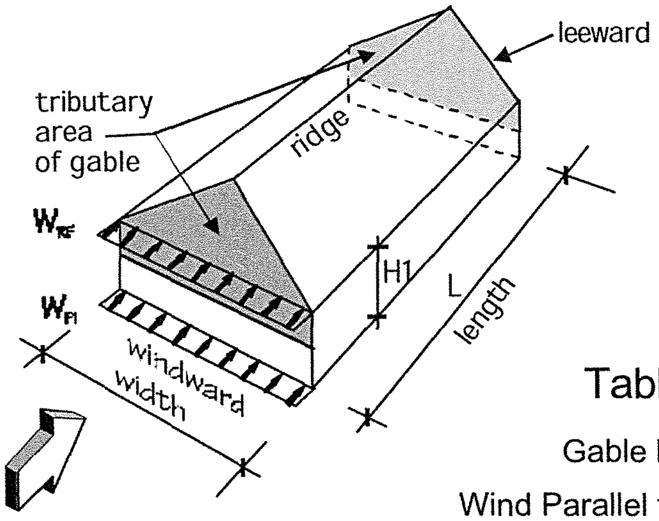


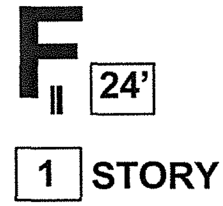
Table B.9

Gable End Wall

Wind Parallel to Ridge of Roof

Roof Slope	Lateral Wind Load to Diaphragms LB/FT of Length							
	Fastest Mile Wind Speed (MPH)							
	80		90		100		110	
h	W_{RF}	W_{F1}	W_{RF}	W_{F1}	W_{RF}	W_{F1}	W_{RF}	W_{F1}
3	95	186	121	236	149	291	181	353
4	103	193	130	245	161	303	195	367
5	110	201	140	255	173	315	209	367
6	118	208	150	265	185	327	224	396
7	125	216	160	274	196	338	238	410
8	133	223	169	284	208	350	252	424
9	140	231	178	293	220	362	267	439
10	148	239	188	303	232	374	281	453
12	164	254	208	323	257	399	311	483

1. Table based on ASCE:7-93 wind requirements for MWFRS.
2. H1 = 9FT based on 8FT floor to ceiling weight. If 10FT floor to ceiling height, multiply Table values by 1.25.
3. L = 60 FT was used for Tabled values.



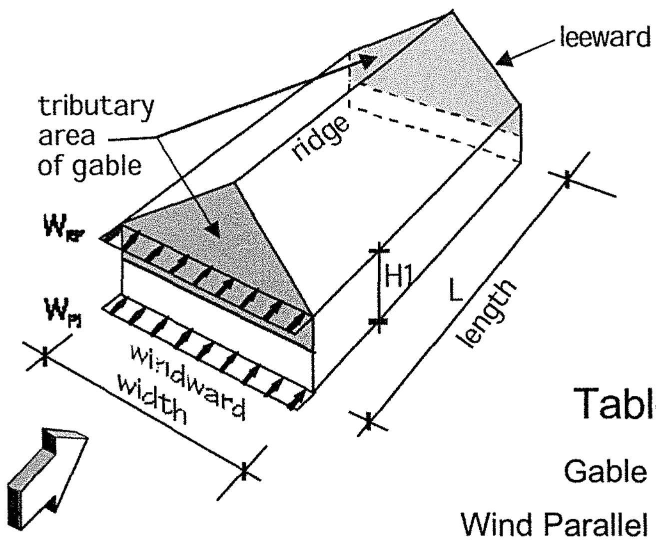


Table B.10

Gable End Wall

Wind Parallel to Ridge of Roof

Roof Slope	Lateral Wind Load to Diaphragms LB/FT of Length							
	Fastest Mile Wind Speed (MPH)							
	80		90		100		110	
h	W_{RF}	W_{F1}	W_{RF}	W_{F1}	W_{RF}	W_{F1}	W_{RF}	W_{F1}
3	102	195	130	248	161	307	195	372
4	112	205	142	260	175	321	212	398
5	121	214	153	271	190	336	230	407
6	130	223	165	283	204	350	248	425
7	140	233	177	295	219	365	266	443
8	149	242	189	307	234	380	283	460
9	159	252	201	319	249	395	302	479
10	168	261	213	331	264	410	320	497
12	188	281	239	357	295	441	358	535

1. Table based on ASCE:7-93 wind requirements for MWFRS.
2. H1 = 9FT based on 8FT floor to ceiling weight. If 10FT floor to ceiling height, multiply Table values by 1.25.
3. L = 60 FT was used for Tabled values.

F ∥ 28'

1 STORY

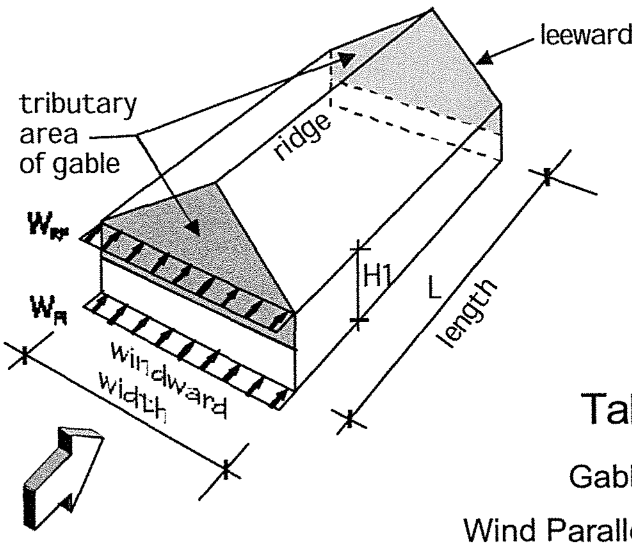


Table B.11

Gable End Wall

Wind Parallel to Ridge of Roof

Roof Slope	Lateral Wind Load to Diaphragms LB/FT of Length							
	Fastest Mile Wind Speed (MPH)							
	80		90		100		110	
h	W_{RF}	W_{F1}	W_{RF}	W_{F1}	W_{RF}	W_{F1}	W_{RF}	W_{F1}
3	108	202	137	257	169	316	205	384
4	119	213	151	271	186	333	226	405
5	130	224	165	285	203	350	247	425
6	141	235	179	299	221	368	268	446
7	152	246	193	313	238	385	289	467
8	163	257	208	327	256	403	310	489
9	175	269	222	342	273	420	332	511
10	187	281	237	357	292	439	355	533
12	213	309	271	393	335	485	404	585

1. Table based on ASCE:7-93 wind requirements for MWFRS.
2. H1 = 9FT based on 8FT floor to ceiling weight. If 10FT floor to ceiling height, multiply Table values by 1.25.
3. L = 60 FT was used for Tabled values.

F II **32'**

1 STORY

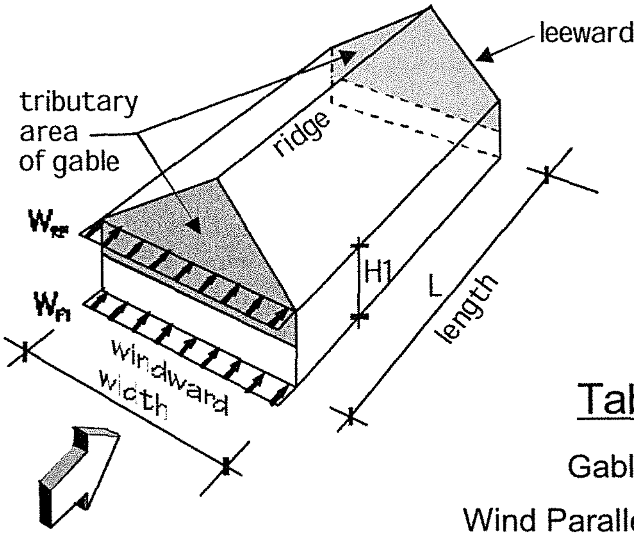


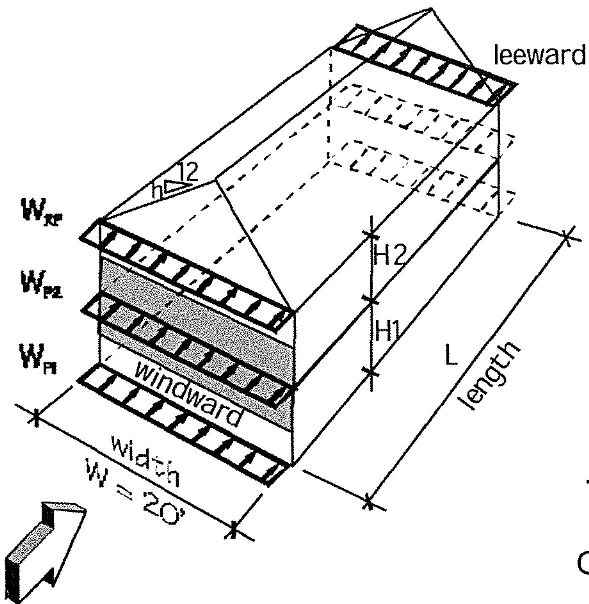
Table B.12

Gable End Wall

Wind Parallel to Ridge of Roof

Roof Slope	Lateral Wind Load to Diaphragms LB/FT of Length							
	Fastest Mile Wind Speed (MPH)							
	80		90		100		110	
h	W_{RF}	W_{F1}	W_{RF}	W_{F1}	W_{RF}	W_{F1}	W_{RF}	W_{F1}
3	117	215	148	272	183	336	222	407
4	130	228	165	288	203	356	247	432
5	143	241	181	305	224	376	271	456
6	156	254	198	321	244	397	296	481
7	169	267	214	338	264	417	321	506
8	182	280	231	354	285	437	345	530
9	195	293	247	371	305	458	370	555
10	211	310	267	392	330	484	399	586
12	242	343	305	432	377	534	456	646

1. Table based on ASCE:7-93 wind requirements for MWFRS.
2. H1 = 9FT based on 8FT floor to ceiling weight. If 10FT floor to ceiling height, multiply Table values by 1.25.
3. L = 60 FT was used for Tabled values.



F II **20**

2 **STORY**

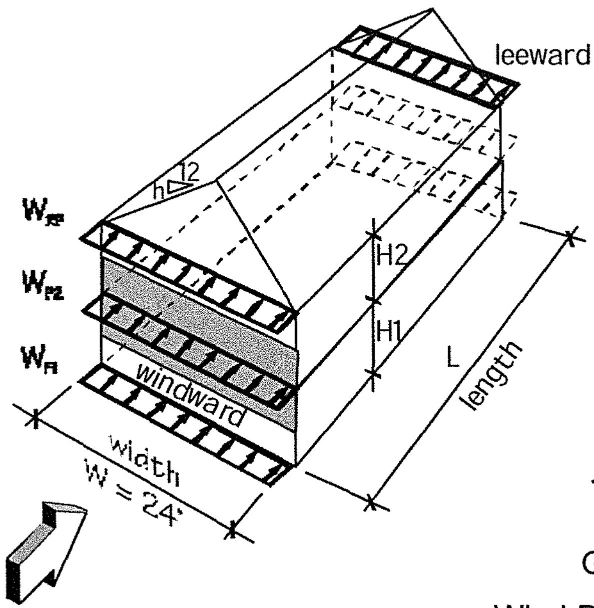
Table B.13

Gable End Wall

Wind Parallel to Roof Ridge

Roof Slope	Lateral Wind Load to Diaphragms LB/FT of Length											
	Fastest Mile Wind Speed (MPH)											
	80			90			100			110		
h	W_{RF}	W_{F2}	W_{FDTN}	W_{RF}	W_{F2}	W_{FDTN}	W_{RF}	W_{F2}	W_{FDTN}	W_{RF}	W_{F2}	W_{FDTN}
3	100	264	355	126	334	450	156	413	555	189	498	670
4	109	271	362	137	343	457	169	426	568	205	513	685
5	117	280	370	1147	353	468	182	439	581	220	529	700
6	125	288	379	159	366	481	196	453	595	238	548	720
7	133	296	387	169	376	491	209	465	608	253	563	735
8	144	307	398	182	390	505	224	482	625	271	582	754
9	152	316	407	193	400	516	237	495	638	287	597	770
10	161	325	416	204	412	527	252	509	652	304	613	785
12	178	342	433	226	433	549	279	536	679	337	645	817

1. Table based on ASCE:7-93 wind requirements for MWFRS.
2. H1 = H2 = 9FT based on 8FT floor to ceiling height. If 10FT floor to ceiling height, multiply Table values by 1.25.
3. L = 60 FT was used for Tabled values.



F || 24

2 STORY

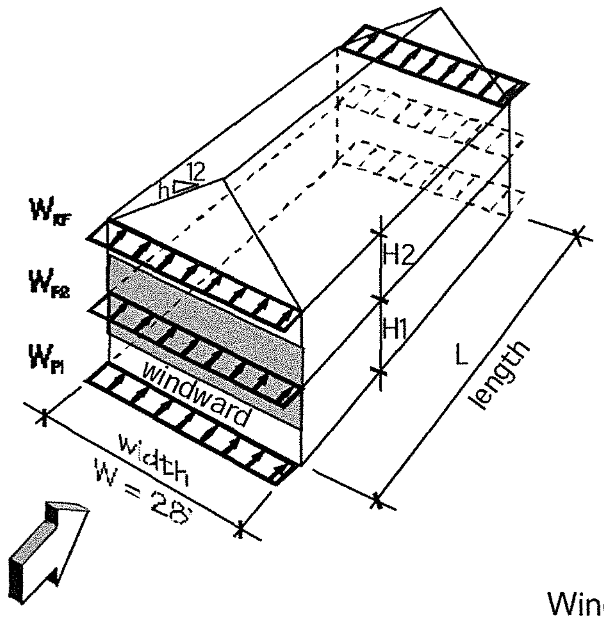
Table B.14

Gable End Wall

Wind Parallel to Roof Ridge

Roof Slope	Lateral Wind Load to Diaphragms LB/FT of Length											
	Fastest Mile Wind Speed (MPH)											
	80			90			100			110		
h	W_{RF}	W_{F2}	W_{FDTN}	W_{RF}	W_{F2}	W_{FDTN}	W_{RF}	W_{F2}	W_{FDTN}	W_{RF}	W_{F2}	W_{FDTN}
3	108	276	370	136	351	470	168	432	579	204	523	701
4	119	287	381	149	362	480	185	449	595	223	541	717
5	129	298	391	164	377	496	202	465	612	245	565	742
6	140	308	402	177	390	509	218	482	628	265	584	762
7	152	321	415	192	406	525	236	501	648	287	607	785
8	161	328	421	203	416	534	251	514	660	304	622	798
9	173	341	435	218	432	550	269	532	678	327	645	822
10	183	351	445	232	445	564	286	548	694	346	665	842
12	208	378	472	263	477	596	326	592	739	393	713	891

1. Table based on ASCE:7-93 wind requirements for MWFRS.
2. H1 = H2 = 9FT based on 8FT floor to ceiling height. If 10FT floor to ceiling height, multiply Table values by 1.25.
3. L = 60 FT was used for Tabled values.



F || 28'

2 STORY

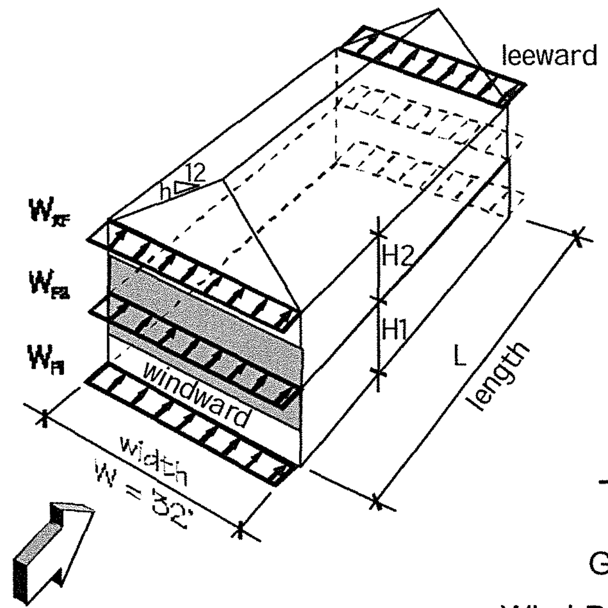
Table B.15

Gable End Wall

Wind Parallel to Roof Ridge

Roof Slope	Lateral Wind Load to Diaphragms LB/FT of Length											
	Fastest Mile Wind Speed (MPH)											
	80			90			100			110		
h	W_{RF}	W_{F2}	W_{FDTN}	W_{RF}	W_{F2}	W_{FDTN}	W_{RF}	W_{F2}	W_{FDTN}	W_{RF}	W_{F2}	W_{FDTN}
3	114	283	378	144	358	477	178	444	591	216	537	716
4	126	295	389	159	373	492	197	462	610	238	560	738
5	139	309	404	176	391	510	217	484	632	264	586	765
6	153	324	419	194	410	530	239	506	654	298	612	792
7	164	333	427	208	423	542	256	522	669	310	630	808
8	178	348	442	225	440	560	277	543	690	337	659	838
9	192	362	456	243	459	579	299	566	714	363	686	866
10	204	374	469	259	475	595	319	585	733	387	710	889
12	231	401	495	293	510	631	362	629	777	438	760	939

1. Table based on ASCE:7-93 wind requirements for MWFRS.
2. H1 = H2 = 9FT based on 8FT floor to ceiling height. If 10FT floor to ceiling height, multiply Table values by 1.25.
3. L = 60 FT was used for Tabled values.



F II 32'

2 STORY

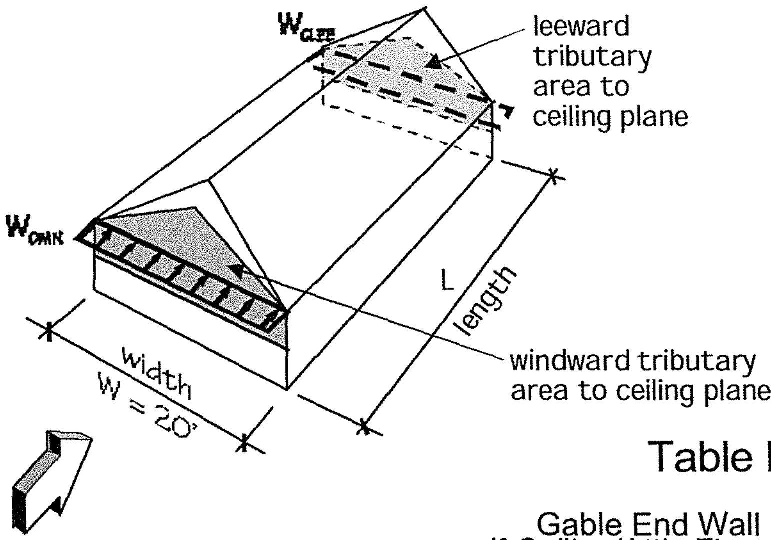
Table B.16

Gable End Wall

Wind Parallel to Roof Ridge

Roof Slope	Lateral Wind Load to Diaphragms LB/FT of Length											
	Fastest Mile Wind Speed (MPH)											
	80			90			100			110		
h	W_{RF}	W_{F2}	W_{FDTN}	W_{RF}	W_{F2}	W_{FDTN}	W_{RF}	W_{F2}	W_{FDTN}	W_{RF}	W_{F2}	W_{FDTN}
3	124	301	399	156	380	504	193	470	623	234	568	753
4	139	315	413	175	398	522	217	494	648	263	598	784
5	155	332	430	196	421	546	241	519	674	292	628	814
6	168	344	442	212	435	559	261	536	689	317	651	837
7	183	359	457	232	456	581	286	562	716	347	682	868
8	200	377	476	253	478	603	312	589	743	377	713	900
9	217	395	494	274	500	626	339	618	773	408	745	932
10	228	405	503	290	514	638	358	635	788	432	766	952
12	263	441	540	333	559	685	412	690	844	497	834	1021

1. Table based on ASCE:7-93 wind requirements for MWFRS.
2. H1 = H2 = 9FT based on 8FT floor to ceiling height. If 10FT floor to ceiling height, multiply Table values by 1.25.
3. L = 60 FT was used for Tabled values.



$C_{||}$ 20'

1 STORY

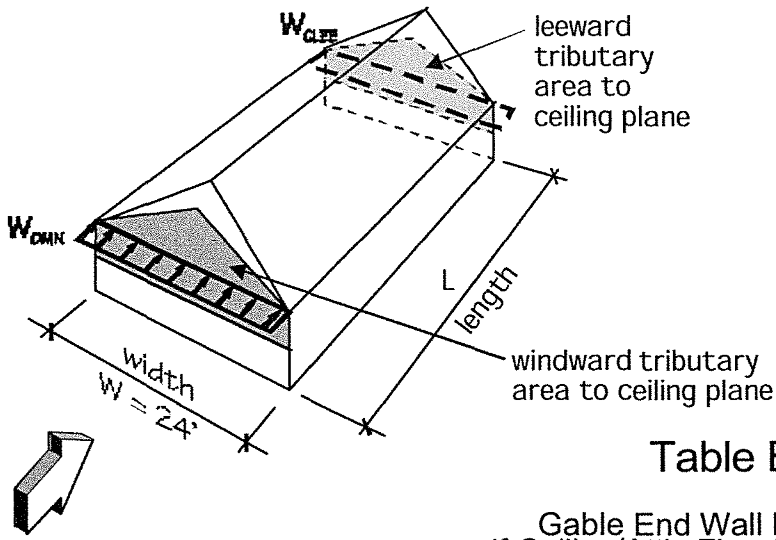
Table B.17

Gable End Wall Bracing Load if Ceiling/Attic Floor/Discrete Braces

Wind Parallel to Roof ridge

Roof Slope	Lateral Wind Load to Diaphragms LB/FT of Length											
	Fastest Mile Wind Speed (MPH)											
	80			90			100			110		
h	W_{CWIN}	W_{CLEE}	W_{CTOT}	W_{CWIN}	W_{CLEE}	W_{CTOT}	W_{CWIN}	W_{CLEE}	W_{CTOT}	W_{CWIN}	W_{CLEE}	W_{CTOT}
3	64	-20	84	81	-25	106	100	-31	131	121	-38	159
4	67	-21	88	85	-27	112	104	-33	137	127	-40	167
5	70	-22	92	88	-28	116	109	-34	143	132	-41	173
6	72	-23	95	92	-29	121	113	-36	149	138	-43	181
7	75	-23	98	96	-30	126	118	-37	155	143	-45	188
8	78	-25	103	99	-31	130	122	-39	161	148	-47	195
9	81	-25	106	103	-32	135	127	-40	167	154	-48	202
10	84	-26	110	106	-34	140	132	-41	173	159	-50	209
12	90	-28	118	114	-36	150	141	-44	185	170	-54	224

1. Table based on ASCE:7-93 wind requirements for MWFRS.
2. H1 = 9FT based on 8FT floor to ceiling height. If 10FT floor to ceiling height, multiply Table values by 1.25.
3. L = 60 FT was used for Tabled values.



C_I 24'

1 STORY

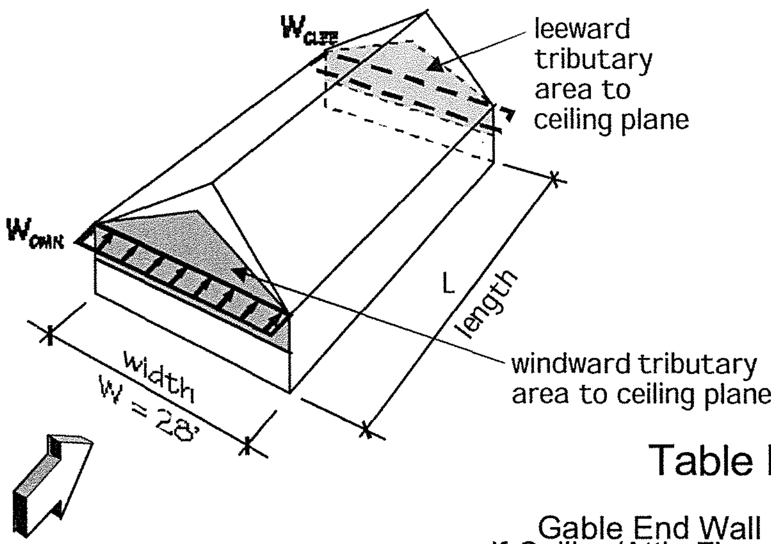
Table B.18

Gable End Wall Bracing Load
if Ceiling/Attic Floor/Discrete Braces

Wind Parallel to Roof ridge

Roof Slope	Lateral Wind Load to Diaphragms LB/FT of Length											
	Fastest Mile Wind Speed (MPH)											
	80			90			100			110		
h	W_{CWIN}	W_{CLEE}	W_{CTOT}	W_{CWIN}	W_{CLEE}	W_{CTOT}	W_{CWIN}	W_{CLEE}	W_{CTOT}	W_{CWIN}	W_{CLEE}	W_{CTOT}
3	66	-23	89	83	-29	112	103	-36	139	124	-44	168
4	69	-24	93	88	-31	119	108	-38	146	131	-46	177
5	73	-25	98	92	-32	124	113	-40	153	138	-48	186
6	76	-26	102	96	-34	130	119	-42	161	144	-51	195
7	79	-28	107	101	-35	136	124	-44	168	151	-53	204
8	83	-29	112	105	-37	142	130	-46	176	157	-55	212
9	86	-30	116	109	-39	148	135	-48	183	164	-58	222
10	90	-31	121	114	-40	154	141	-49	190	170	-60	230
12	97	-34	131	123	-43	166	151	-53	204	183	-65	248

1. Table based on ASCE:7-93 wind requirements for MWFRS.
2. H1 = 9FT based on 8FT floor to ceiling height. If 10FT floor to ceiling height, multiply Table values by 1.25.
3. L = 60 FT was used for Tabled values.



C_{II} 28'

1 STORY

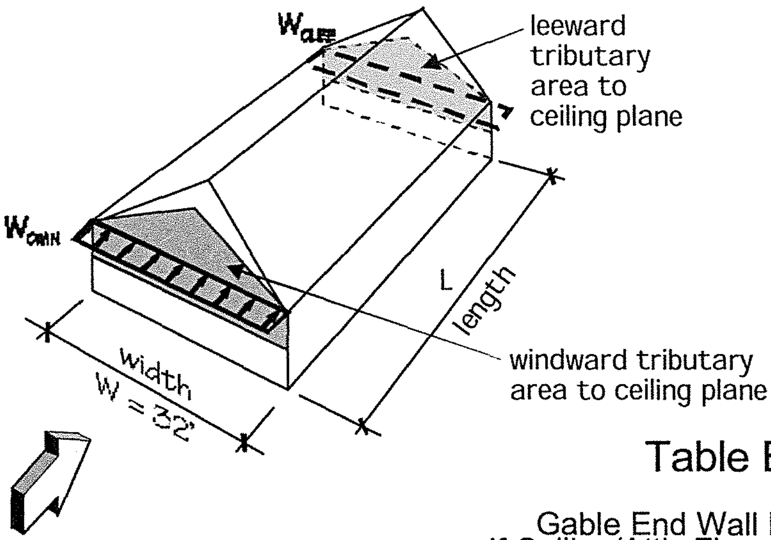
Table B.19

Gable End Wall Bracing Load if Ceiling/Attic Floor/Discrete Braces

Wind Parallel to Roof ridge

Roof Slope	Lateral Wind Load to Diaphragms LB/FT of Length											
	Fastest Mile Wind Speed (MPH)											
	80			90			100			110		
h	W_{CWIN}	W_{CLEE}	W_{CTOT}	W_{CWIN}	W_{CLEE}	W_{CTOT}	W_{CWIN}	W_{CLEE}	W_{CTOT}	W_{CWIN}	W_{CLEE}	W_{CTOT}
3	68	-24	92	85	-32	117	105	-38	143	128	-46	174
4	71	-26	97	90	-33	123	112	-40	152	135	-49	184
5	76	-27	103	96	-35	131	118	-43	161	143	-52	195
6	79	-29	108	101	-37	138	124	-45	169	151	-55	206
7	84	-30	114	106	-39	145	131	-47	178	158	-58	216
8	87	-32	119	111	-41	152	137	-49	186	166	-60	226
9	92	-33	125	116	-42	158	143	-52	195	174	-63	237
10	95	-35	130	121	-44	165	149	-54	203	181	-66	247
12	105	-38	143	134	-49	183	165	-60	225	200	-72	272

1. Table based on ASCE:7-93 wind requirements for MWFRS.
2. H1 = 9FT based on 8FT floor to ceiling height. If 10FT floor to ceiling height, multiply Table values by 1.25.
3. L = 60 FT was used for Tabled values.



C_{II} 32'

1 STORY

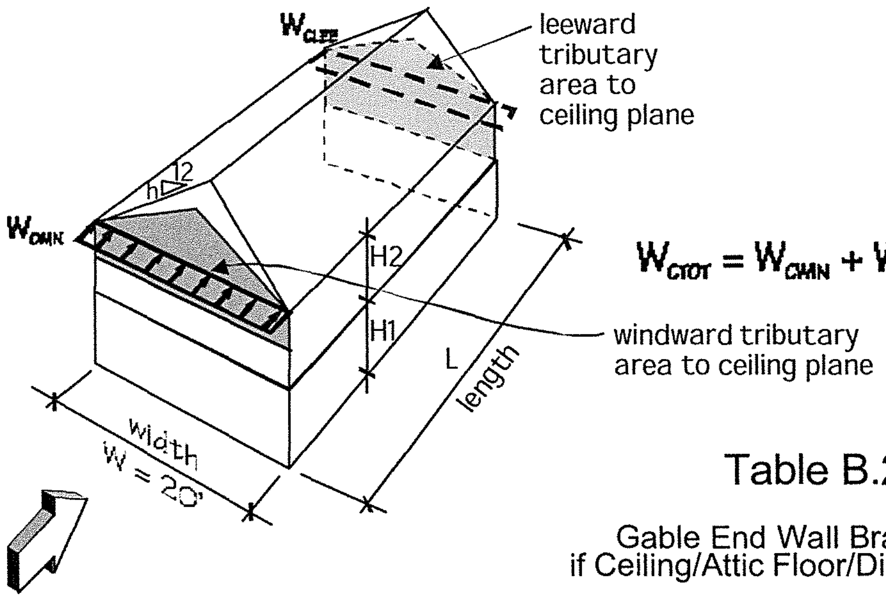
Table B.20

Gable End Wall Bracing Load
if Ceiling/Attic Floor/Discrete Braces

Wind Parallel to Roof ridge

Roof Slope	Lateral Wind Load to Diaphragms LB/FT of Length											
	Fastest Mile Wind Speed (MPH)											
	80			90			100			110		
h	W_{CWIN}	W_{CLEE}	W_{CTOT}	W_{CWIN}	W_{CLEE}	W_{CTOT}	W_{CWIN}	W_{CLEE}	W_{CTOT}	W_{CWIN}	W_{CLEE}	W_{CTOT}
3	69	-29	98	88	-36	124	108	-45	153	131	-54	185
4	74	-30	104	93	-39	132	115	-48	163	140	-58	198
5	78	-32	110	99	-41	140	122	-51	173	148	-62	210
6	83	-34	117	105	-43	148	130	-53	183	157	-65	222
7	87	-36	123	111	-46	157	137	-56	193	166	-68	234
8	92	-38	130	117	-48	165	144	-59	203	175	-72	247
9	97	-40	137	123	-50	173	152	-62	214	183	-76	259
10	103	-43	146	130	-54	184	161	-66	227	194	-80	274
12	114	-48	162	144	-59	203	178	-74	252	215	-89	304

1. Table based on ASCE:7-93 wind requirements for MWFRS.
2. H1 = 9FT based on 8FT floor to ceiling height. If 10FT floor to ceiling height, multiply Table values by 1.25.
3. L = 60 FT was used for Tabled values.



C_{II} 20'

2 STORY

$$W_{CTOT} = W_{CWIN} + W_{CLEE}$$

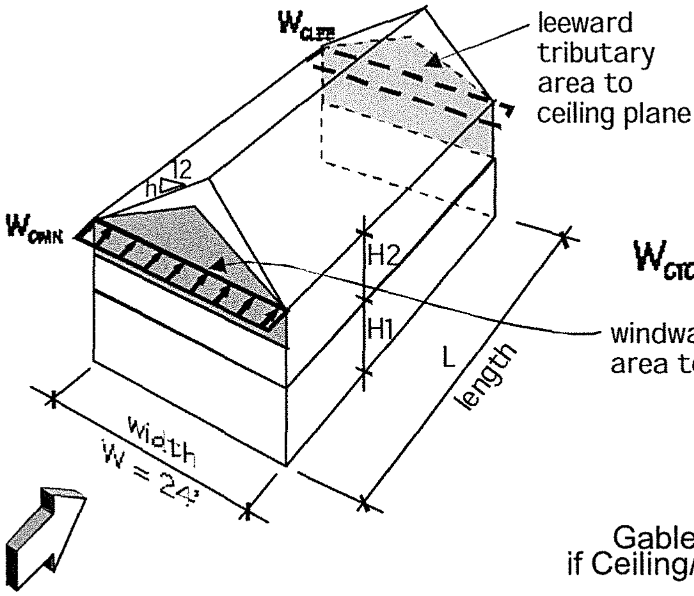
Table B.21

Gable End Wall Bracing Load
if Ceiling/Attic Floor/Discrete Braces

Wind Parallel to Roof Ridge

Roof Slope	Lateral Wind Load to Diaphragms LB/FT of Length											
	Fastest Mile Wind Speed (MPH)											
	80			90			100			110		
h	W_{CWIN}	W_{CLEE}	W_{CTOT}	W_{CWIN}	W_{CLEE}	W_{CTOT}	W_{CWIN}	W_{CLEE}	W_{CTOT}	W_{CWIN}	W_{CLEE}	W_{CTOT}
3	68	-21	89	86	-27	113	106	-33	139	128	-40	168
4	72	-22	94	90	-28	118	111	-35	146	134	-42	176
5	75	-23	98	94	-29	123	116	-36	152	140	-44	184
6	78	-24	102	99	-31	130	122	-38	160	148	-46	194
7	81	-25	106	103	-32	135	127	-40	167	154	-48	202
8	86	-27	113	108	-34	142	133	-42	175	162	-50	212
9	89	-28	117	112	-35	147	138	-43	181	167	-53	220
10	92	-29	121	116	-37	153	144	-45	189	174	-54	228
12	98	-31	129	124	-39	163	153	-48	201	186	-58	244

1. Table based on ASCE:7-93 wind requirements for MWFRS.
2. H1 = H2 = 9FT based on 8FT floor to ceiling height. If 10FT floor to ceiling height, multiply Table values by 1.25.
3. L = 60 FT was used for Tabled values.



C$_{II}$ 24

2 STORY

$$W_{CTOT} = W_{CWIN} + W_{CLEE}$$

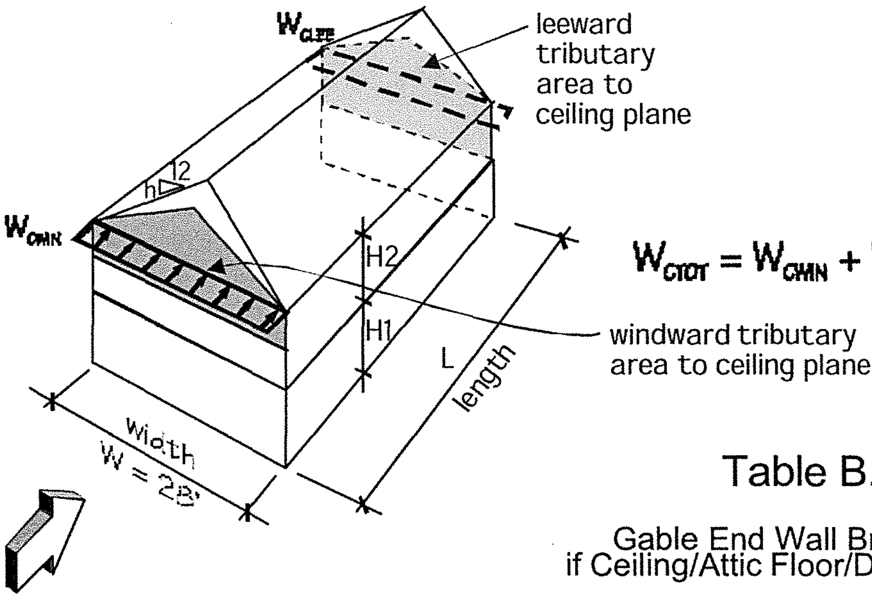
Table B.22

Gable End Wall Bracing Load
if Ceiling/Attic Floor/Discrete Braces

Wind Parallel to Roof Ridge

Roof Slope	Lateral Wind Load to Diaphragms LB/FT of Length											
	Fastest Mile Wind Speed (MPH)											
	80			90			100			110		
h	W_{CWIN}	W_{CLEE}	W_{CTOT}	W_{CWIN}	W_{CLEE}	W_{CTOT}	W_{CWIN}	W_{CLEE}	W_{CTOT}	W_{CWIN}	W_{CLEE}	W_{CTOT}
3	70	-24	94	88	-31	119	109	-38	147	132	-46	178
4	74	-26	100	93	-33	126	115	-41	156	139	-49	188
5	78	-28	106	99	-35	134	122	-43	165	148	-52	200
6	82	-29	111	103	-37	140	128	-45	173	156	-54	210
7	87	-30	117	110	-39	149	135	-47	182	164	-58	222
8	90	-31	121	113	-40	153	140	-49	189	170	-60	230
9	95	-33	128	119	-42	161	148	-51	199	174	-63	242
10	98	-34	132	124	-44	168	153	-54	207	186	-65	251
12	109	-38	147	137	-48	185	169	-60	229	205	-71	276

1. Table based on ASCE:7-93 wind requirements for MWFRS.
2. H1 = H2 = 9FT based on 8FT floor to ceiling height. If 10FT floor to ceiling height, multiply Table values by 1.25.
3. L = 60 FT was used for Tabled values.



C_{II} 28

2 STORY

$$W_{CTOT} = W_{CWIN} + W_{CLEE}$$

Table B.23

Gable End Wall Bracing Load
if Ceiling/Attic Floor/Discrete Braces

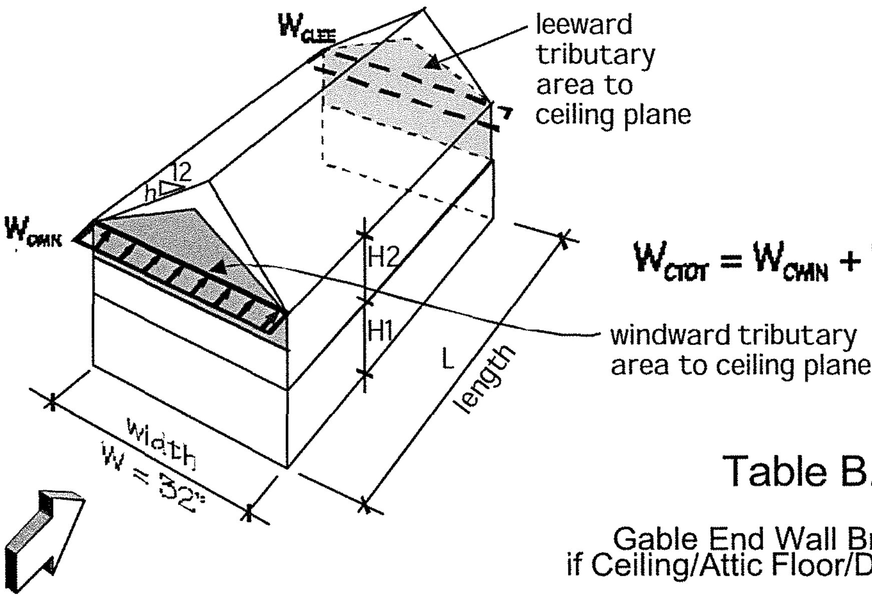
Wind Parallel to Roof Ridge

Roof Slope	Lateral Wind Load to Diaphragms LB/FT of Length											
	Fastest Mile Wind Speed (MPH)											
	80			90			100			110		
h	W_{CWIN}	W_{CLEE}	W_{CTOT}	W_{CWIN}	W_{CLEE}	W_{CTOT}	W_{CWIN}	W_{CLEE}	W_{CTOT}	W_{CWIN}	W_{CLEE}	W_{CTOT}
3	72	-26	98	91	-33	124	112	-41	153	136	-49	185
4	76	-28	104	96	-35	131	119	-43	162	144	-52	196
5	81	-30	111	103	-37	140	122	-46	173	154	-56	210
6	87	-32	119	110	-40	150	135	-49	184	164	-59	223
7	91	-33	124	114	-42	156	141	-51	192	171	-62	233
8	96	-35	131	121	-44	165	150	-54	204	181	-66	247
9	102	-36	138	129	-46	175	158	-57	215	191	-70	261
10	106	-38	144	135	-48	183	165	-60	225	200	-73	273
12	116	-42	158	147	-53	200	182	-66	248	220	-80	300

1. Table based on ASCE:7-93 wind requirements for MWFRS.
2. H1 = H2 = 9FT based on 8FT floor to ceiling height. If 10FT floor to ceiling height, multiply Table values by 1.25.
3. L = 60 FT was used for Tabled values.

C ‖ 32

2 STORY



$$W_{CTOT} = W_{CWIN} + W_{CLEE}$$

Table B.24

Gable End Wall Bracing Load
if Ceiling/Attic Floor/Discrete Braces

Wind Parallel to Roof Ridge

Roof Slope	Lateral Wind Load to Diaphragms LB/FT of Length											
	Fastest Mile Wind Speed (MPH)											
	80			90			100			110		
h	W_{CWIN}	W_{CLEE}	W_{CTOT}	W_{CWIN}	W_{CLEE}	W_{CTOT}	W_{CWIN}	W_{CLEE}	W_{CTOT}	W_{CWIN}	W_{CLEE}	W_{CTOT}
3	74	-31	105	93	-39	132	115	-48	163	139	-58	197
4	79	-33	112	100	-41	141	124	-51	175	151	-61	212
5	86	-35	121	108	-45	153	133	-55	188	162	-66	228
6	90	-37	127	113	-47	160	140	-58	198	170	-70	240
7	96	-39	135	121	-50	171	150	-61	211	181	-75	256
8	102	-42	144	130	-53	183	160	-65	225	193	-79	272
9	109	-45	154	137	-57	194	169	-70	239	204	-84	288
10	112	-46	158	142	-59	201	176	-73	249	213	-87	300
12	125	-52	177	158	-66	224	196	-81	277	237	-98	335

1. Table based on ASCE:7-93 wind requirements for MWFRS.
2. H1 = H2 = 9FT based on 8FT floor to ceiling height. If 10FT floor to ceiling height, multiply Table values by 1.25.
3. L = 60 FT was used for Tabled values.

Table B.25

Recommended Uniform Roof Live Loads for APA Rated Sheathing [c] and APA Rated Sturdi-I-Floor With Long Dimension Perpendicular to Support[e]

Panel Span Rating	Minimum Panel Thickness (in.)	Maximum Span (in.)		Allowable Live Loads (psf)[d]							
				Spacing of Supports Center-to-Center (in.)							
		With Edge Support[a]	Without Edge Support	12	16	20	24	32	40	48	60
APA Rated Sheathing [c]											
12/0	5/16	12	12	30							
16/0	5/16	16	16	70	30						
20/0	5/16	20	20	120	50	30					
24/0	3/8	24	20[b]	190	100	60	30				
24/16	7/16	24	24	190	100	65	40				
32/16	15/32	32	28	325	180	120	70	30			
40/20	19/32	40	32	–	305	205	130	60	30		
48/24	23/32	48	36	–	–	280	175	95	45	35	
60/32	7/8	60	48	–	–	–	305	165	100	70	35
APA Rated Sturd-I-Floor [f]											
16 oc	19/32	24	24	185	100	65	40				
20 oc	19/32	32	32	270	150	100	60	30			
24 oc	23/32	48	36	–	240	160	100	50	30	25	
32 oc	7/8	48	40	–	–	295	185	100	60	40	
48 oc	1-3/32	60	48	–	–	–	290	160	100	100	40

(a) Tongue-and-groove edges, panel edge clips (one midway between each support, except two equally spaced between supports 48 inches on center), lumber blocking, or other

(b) 24 inches for 15/32-inch and 1/2-inch panels.

(c) Includes APA RATED SHEATHING/CEILING DECK.

(d) 10 psf dead load assumed

(e) Applies to panels 24 inches or wider applied over two or more spans.

(f) Also applies to C-C Plugged grade plywood.

Note: Shaded support spacings meet Code Plus recommendations.

Note: Tables 25 and 26 are excerpts from APA - The Engineered Wood Association - *Residential and Commercial, 1994.*

Table B.26

APA Panel Subflooring (APA RATED SHEATHING)[a][f]

Panel Span Rating	Minimum Panel Thickness (in.)	Maximum Span (in.)	Nail Size & Type [e]	Maximum Nail Spacing (in.)	
				Supported Panel Edges	Intermediate Supports
24/16	7/16	16	6d common	6	12
32/16	15/32	16(b)	8d common(c)	6	12
40/20	19/32	20(b)(d)	8d common	6	12
48/24	23/32	24	8d common	6	12
60/32	7/8	32	8d common	6	12

(a) For subfloor recommendations under ceramic tile and for subfloor recommendations under gypsum concrete, contact manufacturer of floor topping.

(b) Span may be 24 inches if 3/4-inch wood strip flooring is installed at right angles to joists

(c) 6d common nail permitted if panel is 1/2 inch or thinner.

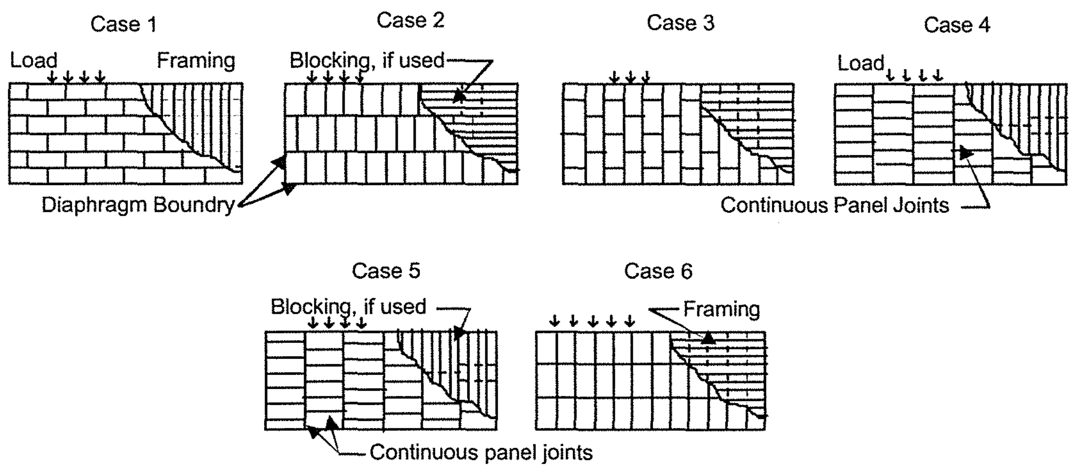
(d) Span may be 24 inches if a minimum 1-1/2 inches of lightweight concrete is applied over panels.

(e) Other code-approved fasteners may be used.

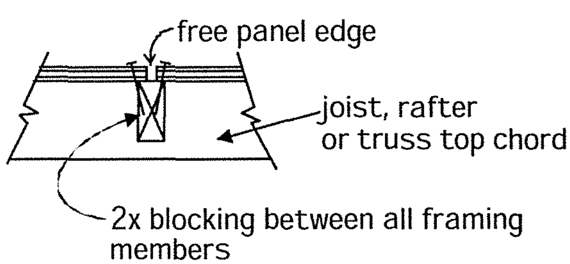
(f) APA RATED STURDI-I-FLOOR may be substituted when the Span Rating is equal to or greater than tabulated maximum span.

Note: Tables 25 and 26 are excerpts from APA - The Engineered Wood Association - *Residential and Commercial, 1994.*

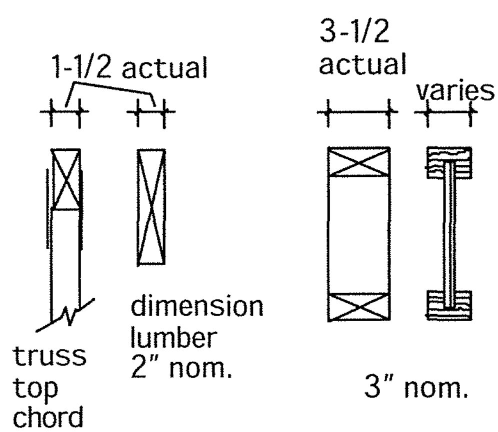
Table B.27 Notes Continued



Structural Panel Layouts
for Diaphragms



Blocked DiaphragmDetail



Note: Table Notes Modified by Author from APA Design/Construction Guide - Diaphragms, 1991, pg. 6

Table B.28

Recommended Shear (PLF) for APA Panel Shear Walls for Wind Loading

Panel Grade	Sheathing Thickness (in.)	Nail Size common or galv. box	S.P.or D.F.LARCH Framing Species (1.0)[1] G ≥ 0.49				S-P-F, HEM-FIR Framing Species (.82)[1] 0.49 > G ≥ 0.42				Western Cedar Framing Species (.65)[1] G < 0.42			
			Panel Edge Nail Spacing (in.)											
			6	4	3	2	6	4	3	2	6	4	3	2
			Recommended Shear Capacity (plf)[1]											
APA Structural I Grades	5/16	6d	200	300	390	510[2]	165	245	320	420[2]	130	195	255	330[2]
	3/8	8d	230[3]	360[3]	460[3]	610[2,3]	188	295	377	500	150	234	300	397
	7/16	8d	255[3]	395[3]	505[3]	670[2,3]	209	324	414	550	166	257	328	436
	15/32	8d	280	430	550	730[2]	230	355	450	600[2]	180	280	360	475[2]
	15/32	10d	340	510	665[2]	870[2]	280	420	545[2]	715[2]	220	330	430[2]	565[2]
APA Rated Sheathing	5/16	6d	180	270	350	450[2]	148	220	287	369	117	176	227	292
	3/8	6d	200	300	390	510[2]	165	245	320	420[2]	130	195	255	330[2]
	3/8	8d	220[3]	320[3]	410[3]	530[2,3]	230	355	320	420[2]	180	280	360	475[2]
	7/16	8d	240[3]	350[3]	450[3]	585[2,3]	197	287	450	600[2]	156	228	293	380
	15/32	8d	260	380	490	640[2]	213	312	402	525	169	247	319	416
	15/32	10d	310	460	6002	770[2]	250	380	490	630	200	300	390	500
	19/32	10d	340	510	6652	870[2]	280	420	545[2]	715[2]	220	330	430[2]	565[2]
APA Rated Siding	5/16	6d	140	210	275	360[2]	115	175	225	295[2]	90	135	180	235[2]
	3/8	8d	160	240	310	410[2]	130	200	255	340[2]	105	155	200	265[2]

1 Multiplier based on species G.

G - Specific gravity of framing species

2 Framing at adjoining panel edges shall be 3 inches nominal or wider and nails shall be staggered.

3 Shear values may be increased to values shown for 15/32-inch sheathing with same nailing provided
(1) studs are spaced a maximum of 16 inches o.c. or
(2) if panels are applied with long dimension across studs.

Note: Table Modified from: Wood Frame Construction Manualfor One- and Two-Family Dwellings, 1995 SBC High Wind Edition, American Forest & Paper Association & the American Wood Council

Typical Layout for Shear Walls

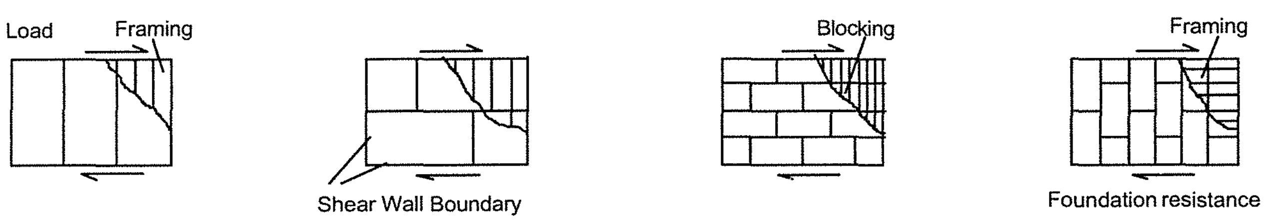


Table B.29
Allowable Shear for Wind or Seismic Forces in Pounds per Foot for Vertical Diaphragms of Lath and Plaster or Gypsum Board Frame Wall Assemblies[1]

Type of Material	Thickness of Material	Wall Construction	Nail Spacing[2] Maximum (in inches)	Shear Value	Maximum Nail Sizes[3,4]
1. Expanded metal, or woven wire lath and portland cement plaster	7/8"	Unblocked	6	180	No. 11 gauge, 1-1/2" long, 7/16" head No. 16 gauge staple, 7/8" legs
2. Gypsum lath	3/8" Lath and 1/2" Plaster	Unblocked	5	100	No. 13 gauge, 1-1/8" long, 19/64" head, plasterboard blue nail
3. Gypsum sheathing board	1/2" x 2' x 8'	Unblocked	4	75	No. 11 gauge, 1-3/4" long, 7/16" head, diamond-point, galvanized
	1/2" x 4'	Blocked	4	175	
	1/2" x 4'	Unblocked	7	100	
4. Gypsum wallboard or veneer base	1/2"	Unblocked	7	100	5d cooler or wallboard
			4	125	
		Blocked	7	125	
			4	150	
	5/8"	Unblocked	7	115	6d cooler or wallboard
			4	145	
		Blocked	7	145	
			4	175	
		Blocked Two Ply	Base ply 9 Face ply 7	250	Base ply-6d cooler or wallboard Face ply-8d cooler or wallboard

1 These vertical diaphragms shall not be used to resist loads imposed by masonry or concrete construction. See Section 4714 (b). Values are for short term loading due to wind. Values must be reduced 25 percent for normal loading. The values for gypsum products must be reduced 50 percent for dynamic loading due to earthquake in Seismic Zones Nos. 3 and 4.

2 Applies to nailing at all studs, top and bottom plates and blocking.

3 Alternate nails may be used if their dimensions are not less than the specified dimensions.

4 For properties of cooler or wallboard nails, see U.B.C. Standard No. 25-17, Table No. 25-17-I.

Table B.30

Specific Gravity for Solid Sawn Lumber

Species Combination	Specific Gravity, G	Species Combination	Specific Gravity, G
Aspen	0.39	Mountain Hemlock	0.47
Balsam Fir	0.36	Northern Pine	0.42
Beech-Birch-Hickory	0.71	Northern Red Oak	0.68
Coast Sitka Spruce	0.39	Northern Species	0.35
Cottonwood	0.41	Northern White Cedar	0.31
Douglas Fir-Larch	0.50	Ponderosa Pine	0.43
Douglas Fir-Larch (North)	0.49	Red Maple	0.58
Douglas Fir-South	0.46	Red Oak	0.67
Eastern Hemlock	0.41	Red Pine	0.44
Eastern Hemlock-Tamarack	0.41	Redwood, close grain	0.44
Eastern Hemlock-Tamarack-(North)	0.47	Redwood, open grain	0.37
Eastern Softwoods	0.36	Sitka Spruce	0.43
Eastern Spruce	0.41	Southern Pine	0.55
Eastern White Pine	0.36	Spruce-Pine-Fir	0.42
Engelmann Spruce-Lodgepole Pine[2] (MSR 1650f and higher grades)	0.46	Spruce-Pine-Fir (South)	0.36
		Western Cedars	0.36
Engelmann Spruce-Lodgepole Pine[2] (MSR 1500f and lower grades)	0.38	Western Cedars (North)	0.35
		Western Hemlock	0.47
Hem-Fir	0.43	Western Hemlock (North)	0.46
Hem-Fir (North)	0.46	Western White Pine	0.40
Mixed Maple	0.55	Western Woods	0.36
Mixed Oak	0.68	White Oak	0.73
Mixed Southern Pine	0.51	Yellow Poplar	0.43

1. Specific gravity based on weight and volumne when oven-dry.
2. Applies only to Engelmann Spruce-Lodgepole Pine machine stress rated (MSR) structural lumber.

DL_{rf}

Table B.31

Truss Reactions

Truss Span FT.	Truss Reaction in lbs./ft. of length ROOF SLOPE 3 to 6 in 12	>6 to 12 in 12
20	76	86
24	91.2	103.2
28	106.4	120.4
32	121.6	137.6
36	136.8	154.8

Notes:

1. DL =

	PSF	
Roof sheath	1.5	Vary with
Asp. shingles	3	Roof slope
Truss 20'-36'	3.1	
5/8" drywall Clg.	2.5	
Batt insul 8"	0.8	

2. 2/3rds DL used in above reaction calculations

3. Linear interpolation for reaction based on truss span between listed values