Chapter

Structural Design of a Typical American Wood-Framed Single-Family Home

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Abstract

Light-wood framing construction techniques have been traditionally used in America for the construction of single-family residences. Dimensional wood lumber is readily available and due to its convenient unit dimension can be packaged neatly and transported to work sites by either commercial transport or personal vehicle. The unit pieces of dimensional lumber are light and easily handled once on the work site. Design of light-framed single-family homes is typically conducted by an architect or construction contractor using prescriptive building codes. A structural engineer can assist, if needed, with design items not within the scope of the building code or if alternative design approaches are required. An owner may choose to involve the engineer to improve quality or economy of the home design. Engineers typically become involved with design items such as foundation design, steel framing design, or engineered product specification. In this chapter, the design of a typical light-framed home is discussed. The main structural assemblies are described and subsequently designed using a combination of prescriptive guidance and engineering design.

Keywords: residential, single-family home, wood, light-framing, house

1. Introduction

The prevailing system used for the construction of single-family homes in the USA is platform framing using light wooden dimensional lumber. Structural assemblies such as the roof, floors, and walls are generally constructed with nominal 50.8 mm (2 inch) lumber members ranging in nominal depths from 101.6 to 304.8 mm (4–12 inches) and sheathed with structural wood panels for stability and security, such as oriented strand board (OSB) or plywood.

Wood structural materials are preferred by US homebuilders largely because (1) the US home building industry is mostly familiar with wood framing method, (2) the units of construction (i.e., studs, joists, panels, etc.) are small and easily transportable, and (3) wood-framed structures can be erected without the need for specialized tools or large equipment.

In this chapter, the complete process of designing a typical US residential dwelling using wood-frame systems will be illustrated. The typical US design methodology and basis will be used to accomplish the designs. The International Residential Code (IRC) [1] is the design basis used by most authorities to regulate the design and construction of single-family residences. The following major aspects are discussed in this chapter:

1. Provide introductory material such as the description of the home to be designed, applicable design codes, and external loading assessment for residential structures.

2. Design the home using a wood-framed platform system. The load path will be discussed as well as specific design codes relating to wood-framed structures. The result of specifying and detailing typical structural elements of the home will be specified and details provided.

The scope is limited to the structural design and performance of one single-family residential dwelling. The load-bearing wall systems are the primary components of the building enclosure, and the structural properties of the wall system are only one of many considerations that must be taken into account. While cladding compatibility, thermal performance or the hygrothermal characteristics of a wall system are very important, such aspects are not the focus of this study and will not be discussed.

The home design considered in this study is a two-story regular-shaped home with a basement and attached two-car garage. The floor plan was provided by S&A Homes, which is a midsized homebuilder that builds homes and provides architectural design services to customers in Pennsylvania and West Virginia. The floor plans and drawings for one of their standard home packages are provided in the Appendix. Clients of S&A Homes can select this floor plan from an array of floor plans and make slight variations to it if desired. S&A Homes will then design, detail, and construct the home for the client on the chosen lot, typically one of S&A's own residential developments.

The home plan/style shown in the Appendix is a popular model in S&A's territory and is representative of the size and style of homes desired by the average homebuyer of this decade. The home consists of nearly 214 m^2 (2300 ft^2) of finished floor area with the basement available for finishing if so desired by the prospective homeowner. The floor plan has features typically seen in modern homes. The first floor contains a large kitchen open to the family room with access to both the dining room and the attached two-car garage. The second floor has four bedrooms with the master suite containing its own large bathroom as well as a sitting area and walk-in closet (WIC).

2. Applicable codes and standards

The IRC is the prevailing design code used for the construction of one- or two-family dwellings in the USA. The 2015 IRC [1] is the current adopted code in the State College, PA area, and will be used as the governing design code for this study. In order to construct a single-family dwelling, the homebuilder must first apply to the local code office for a building permit. It is necessary to provide a complete architectural plan set detailing how the builder intends to comply with the requirements of the IRC, along with several other items such as the manual J [2] heat loss-gain calculations for the structure and selection of energy compliance path. The IRC largely provides a prescriptive basis for home design and in many instances is adequate for single-family home design. The envelope and structural components are typically selected by the architect, builder, or homeowner from design tables within the code. If prefabricated engineered components such as I-joists, laminated veneer lumber (LVL) components, or roof trusses are used in design, a structural engineer is required to review their specification and application.

This is typically the extent of a structural engineer's involvement in residential design other than specialized situations not covered by the IRC and occasionally foundation design. If engineered design is necessary in conjunction with the prescriptive standards, then compliance with the 2015 International Building Code (IBC) [3] requirements for those portions of the design is required. Engineers will conduct their analysis based on requirement set forth in the IRC, IBC if necessary, and ASCE 7-10 minimum design loads for buildings and other structures (ASCE 7) [4] [ASCE stands for American Society of Civil Engineers]. The IRC and IBC also permit designers to refer to the 2015 AWC *Wood Frame Construction Manual* (WFCM) [5] for an alternative prescriptive or engineered approach [AWC stands for American Wood Council].

3. External load determination and serviceability requirements

This study will focus on the appropriate residential structural building loads for the State College, PA area, for an example design case. The designs will include only the effects of dead loading, floor live loading, roof live loading, snow loading, and wind loading. Residential structures in ordinary situations are designed to resist both gravity loads and lateral loads. External loading for homes is prescribed in either Chapter 3 of the 2015 IRC or in ASCE 7. ASCE 7 is the standard referenced in the 2015 IRC, and therefore this version will be referenced in this study. Both the IRC and the ASCE 7 will be used to develop the external loads for this study. In addition to the external loads, the serviceability criteria must also be considered. For this design, only live load deflection limits will be considered.

3.1 Gravity loads

The gravity loads are those loads that act in the direction of gravity. The gravity loads of importance for residential structures are dead load (D_L), floor live load (L_L), roof live load (R_L), and snow load (S_L).

3.2 Dead load (D_L)

Dead load is the load that is permanently and continuously applied to a structure. Typically, dead load refers to the self-weight of the material used in construction or a load that is applied in a permanent nature such as a known location of a piece of heavy equipment or a large island in the kitchen. Unless noted otherwise, the S&A Homes dead load criteria will be used for the wood-framed design of this home. These loads are typical for residential design and were largely derived from ASCE 7 Table C3-1. Dead loads are listed in **Tables 1–3**.

3.3 Live load

Live loading is a gravity loading that is temporary or intermittent in nature. The three live loads considered for the design of this home are floor live (L_L), roof live (R_L), and snow load (S_L). The IRC prescribes the minimum uniformly distributed loads that must be used by designers for residential structures. Such minimum loads listed in **Table 4** will be used for this study.

Sub-component	Weight N/m^2 (lbf/ft^2)
Carpet/vinyl	47.9 (1.0)[a]
19.1 mm (¾ in) plywood	114.9 (2.4)
301.6 mm (11 7/8 in) I-joists[b]	91.0 (1.9)
Mechanical allowance	95.8 (2.0)
12.7 mm (½ in) gypsum ceiling	105.3 (2.2)
Total	≈454.9 (10)

[a]*For floor areas known to have ceramic tile floor covering, increase load to 0.96 kN/m^2 (20 lbf/ft^2).*
[b]*Weight is derived from Weyerhaeuser publication #TJ-4000 for 230 or 360 series joists.*

Table 1.
Floor/ceiling assembly weight.

Sub-component	Weight N/m^2 (lbf/ft^2)
Truss framing	95.8 (2.0)
11.1 mm (7/16 in) sheathing	81.4 (1.7)
Asphalt shingles	114.9 (2.4)
228.6 mm 9 in insulation	86.2 (1.8)
12.7 mm (½ in) gypsum board	105.3 (2.2)
Miscellaneous	95.8 (2.0)
Total	≈ 579.4 (12)

[a]*Engineered wood truss roof system.*

Table 2.
Roof assembly weight.[a]

Sub-component	Weight
Exterior wall assembly[a]	526.7 N/m^2 (11.0 lbf/ft^2)
Interior wall assembly[b]	383.0 N/m^2 (8.0 lbf/ft^2)
Plain concrete	22.8 kN/m^3 (145 lbf/ft^3)
Reinforced concrete	23.6 kN/m^3 (150 lbf/ft^3)

[a]*2 × 6 wood studs at 406.4 mm (16 inch) O.C. with 12.7 mm (½ inch) gypsum wallboard and vinyl siding.*
[b]*Wood or steel studs with 12.7 mm (½ inch) gypsum wallboard on each side.*

Table 3.
Miscellaneous materials.

3.4 Lateral loading

The only lateral load being considered for this study is the wind loading. In the State College area, seismic loading does not typically control the design of structural components. The procedures in ASCE 7 will be used to determine wind loading, e.g., Chapter 28 Envelope Procedure Part 2 can be used for this structure. Chapter 28 requires that the structure meets the definition of a low-rise, enclosed simple diaphragm building that is regular-shaped in accordance with Section 26.2.

Load description	Weight kN/m^2 (lbf/ft^2)
L_L (sleeping rooms)	1.44 (30.0)
L_L (other)	1.92 (40.0)
L_L (habitable attics)	1.44 (30.0)
L_L (attics w/limited storage)[a,b]	0.96 (20.0)
L_L (Attics w/o limited storage)[c]	0.48 (10.0)
Roof live load	0.77 (16.0)
Design roof snow load[d]	1.44 (30.0)

[a]*Attics defined as the unfinished area between the roof and the ceiling of the floor below.*
[b]*Limited storage refers to non-habitable attic space greater than or equal to 1.07 m (42 inch).*
[c]*Add to attic space less than 1.07 m (42 inch).*
[d]*Based on State College area prescriptive requirements. Applied on the horizontal projection rather than along the slope.*

Table 4.
Minimum uniformly distributed live loads.

Parameter	Description
Risk category	II
Basic wind speed (V)	51 m/s 115 mph
Exposure category	B
Topographic factor (K_{zt})	1.0
Mean roof height	7.0 m (23 ft)
Adjustment factor (λ)	1.0
Roof pitch	30 degrees

Table 5.
Wind load parameters.

Zones	Case 1	Case 2	Minimum
A	1.13 (23.6)	1.13 (23.6)	0.77 (16)
B	0.77 (16.1)	0.77 (16.1)	0.38 (8)
C	0.90 (18.8)	0.90 (18.8)	0.77 (16)
D	0.62 (12.9)	0.62 (12.9)	0.38 (8)
E	0.09 (1.8)	0.44 (9.1)	0
F	−0.68 (−14.3)	−0.34 (−7.1)	0
G	0.03 (0.6)	0.38 (7.9)	0
H	−0.59 (−12.3)	−0.24 (−5.0)	0
E_{OH}	−0.40 (−8.3)	−0.40 (−8.3)	0
G_{OH}	−0.45 (−9.5)	−0.45 (−9.5)	0

Values in kN/m^2 (lbf/ft^2).

Table 6.
Simplified design wind pressure (Ps) case A θ = 30.

The wind loads calculated in **Table 6** are based on the parameters listed in **Table 5** and in accordance with **Figure 1**. The simplified design wind pressure magnitudes in **Tables 6** and **7** include both windward and leeward pressures. The combined pressure will be applied to only the windward side of the structure. For this design, two load cases must be evaluated because the roof pitch is between 25 and 30 degrees. Additionally, these two cases must be compared to the minimum load case described in ASCE 7 Section 28.6.4. The case that produces the larger load effect will be used for design of structural members.

3.5 Serviceability criteria

The main serviceability criterion considered in the design of residential homes is deflection. The IRC prescribes the maximum allowable deflection of structural

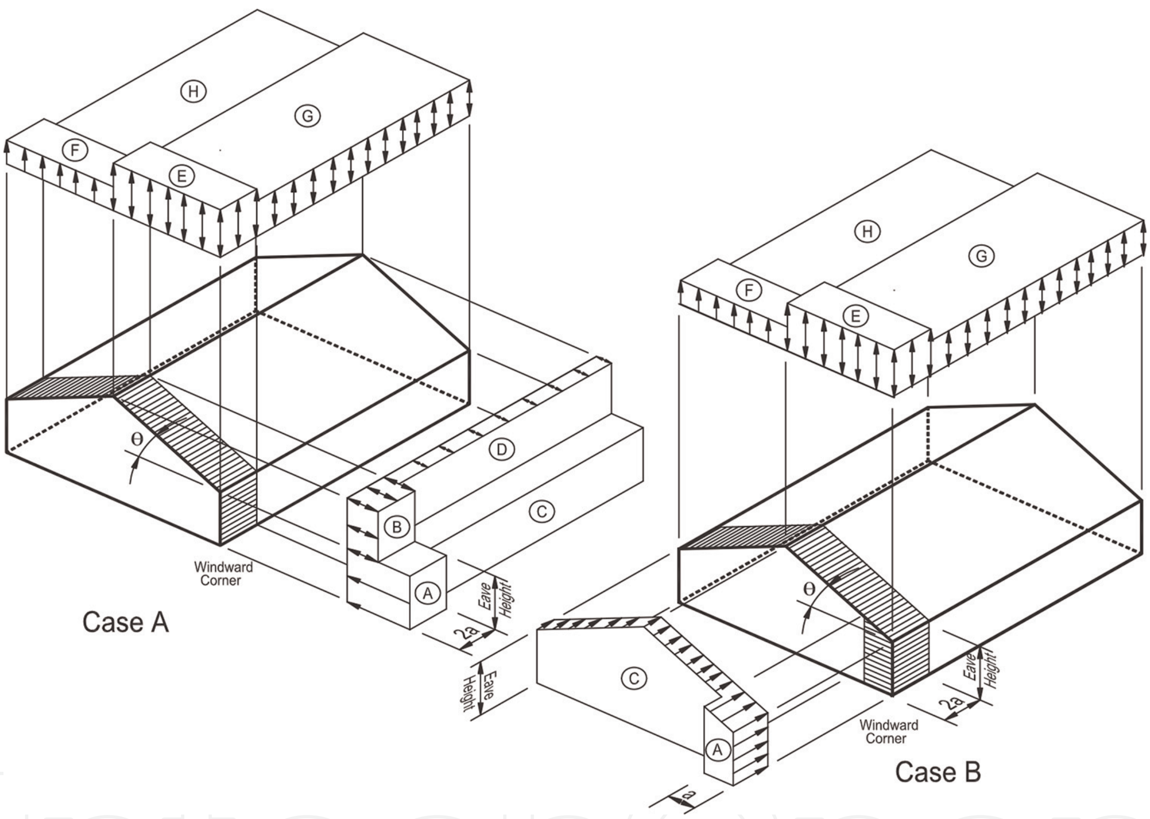


Figure 1.
ASCE 7-10 Chapter 28 wind loading designation (with permission from the ASCE).

Zones	Case 1	Minimum
A	1.01 (21.0)	0.77 (16)
C	0.67 (13.9)	0.77 (16)
E	−1.21 (−25.2)	0
F	−0.68 (−14.3)	0
G	−0.68 (−14.3)	0
H	−0.53 (−11.1)	0

Values in kN/m² (lbf/ft²).

Table 7.
Simplified design wind pressure (Ps) case B θ = 0.

Sub-component	Span ratio
Interior walls and partitions	Height/180
Floors and plaster ceilings[a,b]	Length/360
All other structural members	Length/240
Exterior walls—brittle finish	Length/240

[a]*Limit floor beam deflection to 12.7 mm (½ inch).*
[b]*Limit I-joist deflection ratio to length/480.*

Table 8.
Live load maximum deflection limits.

members and assemblies. Excessive deflections can cause problems for the occupants and potentially damage to nonstructural components such as cladding or fenestration. Excessive interior floor deflections are generally noticed in the form of floor vibration or "spongy" floors. Excessive deflection of roof members can lead to ponding and ultimately moisture issues or overloading of structural members. A portion of Table R301.7 from the IRC that prescribes residential deflection limits is reproduced below in **Table 8**.

3.6 Combination of loads

Both allowable stress design (ASD) and load resistance and factor design (LRFD) load combinations will be utilized for different aspects of the home structural design. For example, the ASD approach will be used for wood design, whereas the LRFD approach will be used for concrete foundation design. Approaches for the designs will be discussed as appropriate. The load combinations that will be used for design are listed below and are reproduced from ASCE 7.

3.6.1 ASD load combinations

1. D
2. D + L
3. D + (Lr or S or R)
4. D + 0.75L + 0.75(Lr or S or R)
5. D + (0.6W or 0.7E)
6. D + 0.75L + 0.75(0.6W) + 0.75(Lr or S or R)
7. 0.6D + 0.6W

3.6.2 LRFD load combinations

1. 1.4D
2. 1.2D + 1.6L + 0.5(Lr or S or R)
3. 1.2D + 1.6(Lr or S or R) + (L or 0.5W)

4. 1.2D + 1.0W + L + 0.5(Lr or S or R)

5. 0.9D + 1.0W

In the above load combination, the notation is defined as follows: D for dead load, L for live load, Lr for roof live load, S for snow load, R for rain load, and W for wind load.

4. Design of residence

Wood is the most popular material used in the USA for the construction of single-family dwellings. An example of residential framing can be seen below in **Figure 2** [6]. Framing lumber is easily obtained in most locations. The units of construction can be easily transported by contractors or homeowners without the need for specialized equipment. Additionally, the erection of a wood-framed structural system is familiar to most and does not require excessive amounts of

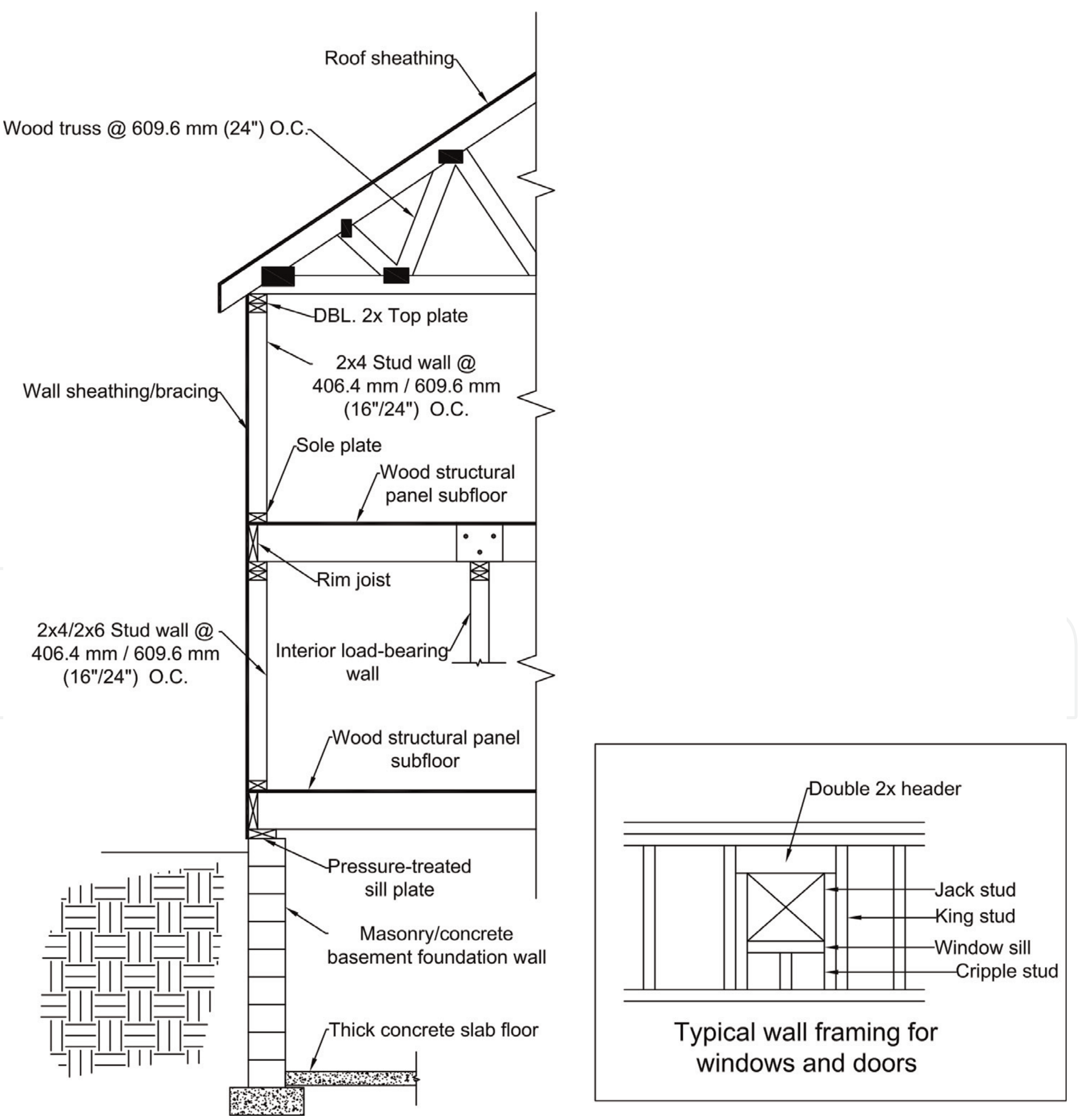


Figure 2.
Section view of typical residential wood-framed home. Note: in this figure, a small rectangle with x inside indicates the cross section of wood member, and DBL stands for double.

specialized knowledge or tools. Lastly, wood-framed construction has been well documented in the USA, and many design aids are available.

As noted before, much of the wood-framed structural design can be accomplished using design aids. The design professional will typically use these design aids to the greatest extent possible and then perform structural analysis and design for any item that is beyond the scope of the design aids. This is the approach that will be used for this study. The design drawings are shown in the Appendix. The associated detailed calculation is not provided due to space limitation; only the necessary results will be mentioned.

4.1 External load transfer (load path)

External loads must be transmitted to ground through the structural system of the building. Two main systems are needed to accomplish this transfer properly: gravity system and the main wind force resisting system (MWFRS). The gravity system transmits the vertical loads through a system of trusses, joists, and beams to foundation, which in turn transmits the load to ground, while the MWFRS transfers lateral wind load to foundation through a system of shear walls and flexible diaphragms. It is important to recognize that the ground must be properly prepared and evaluated to ensure good load transfer. Typically, foundations are placed on virgin soil or engineered (compacted) fill. All organic materials should be removed along with excessive amounts of water.

4.2 Gravity system design

The gravity system in this home starts at the roof and ends in the soil. Vertical loads must have a continuous path to the ground. Generally, the gravity system in this example consists of OSB sheathing, engineered roof trusses, load-bearing stud walls, dimensional lumber headers, engineered I-joist floor system, engineered wood beams, structural steel girders, and a concrete foundation.

4.3 Roof sheathing

The OSB roof sheathing, as illustrated in **Figure 3**, serves to transfer gravity load (i.e., dead, live, and snow loads) and wind suction to roof framing members. The roof sheathing also transfers the lateral wind loading through diaphragm action to the structure. Attachment requirements of the sheathing to roof trusses are governed by the greater of the wind uplift force or the shear transfer requirement of the connection.

According to IRC Table R503.2.1.1(1), 11.1 mm (7/16 inch) roof sheathing (24/16 span rating) is acceptable for this example. The sheathing can be used with or without edge support at 609.6 mm (24 inch) spans with an allowable live load of 1.92 kN/m^2 (40 lbf/ft^2), and a total allowable load of 2.39 kN/m^2 (50 lbf/ft^2), which is less than the 1.44 kN/m^2 (30 lbf/ft^2) snow loading plus 0.57 kN/m^2 (12 lbf/ft^2) roof dead load. It may be possible to use 9.5 mm (3/8 inch) sheathing, but 11.1 mm (7/16 inch) thickness is more readily available and common in the locale. In this example, the sheathing will be specified with panel edge clip support. According to IRC Table R602.3(1), the sheathing is required to be attached to the truss framing with 63.5 mm (2½ inch) 8D common nails spaced at 152.4 mm (6 inch) on center (O.C.) around the edges of the panel and 304.8 mm (12 inch) O.C. at intermediate supports (field). Note that the gable end sheathing connections must be spaced at 152.4 mm (6 inch) O.C. at both the perimeter and intermediate locations.

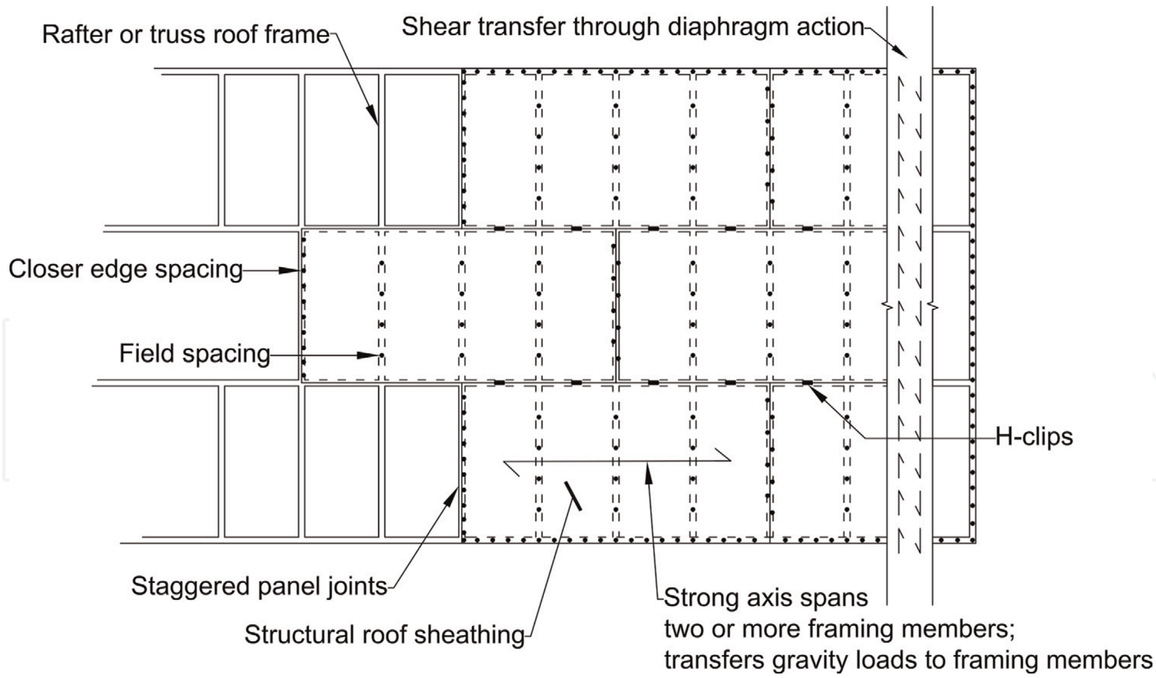


Figure 3.
Roof sheathing illustration.

4.4 Engineered roof trusses

Prefabricated trusses are intended to be used on this residence and required engineering design by the manufacturer. Wood roof trusses must be designed in accordance with IRC Section R802.10. A designer or architect will typically draw the shape of the roof system, and then the truss designer will design the truss system to fit the concept. Typically, it is the responsibility of the home designer to ensure that the gravity and lateral loads from the trusses are properly transferred to the wall below. This involves specifying the connection to wall system below. When the truss drawings are received by the home designer, the loads to the structure, based on the analysis conducted by the truss designer, are typically listed on the engineered truss plans. The designer would use these loads for design. For the example case presented here, however, a set of detailed truss drawings are not available. The assumed loadings described earlier will be used for design. This is typical of an initial home design. A designer will use their assumptions and then verify such assumptions when the final truss plans are received.

4.5 Exterior walls

The gravity load-bearing elements of the wall system presented here are the 2 × 6 dimensional lumber studs and the top and bottom plates (or sole plate). See **Figure 2** for the location of the top and bottom plates. The 2 × 6 designation refers to a wood framing member with a nominal 50.8 mm (2 inch) width and a 152.4 mm (6 inch) depth. The actual measurements of the member are approximately 38.1 mm (1½ inch) wide and 139.7 mm (5½ inch) deep. The top and bottom plates serve to transfer both gravity and lateral loads between floors. The top plate serves three purposes: (1) a chord for the MWFRS, (2) a strut between shear panels in a wall line, and (3) a means to transfer gravity loads to the stud from the joists and trusses.

According to IRC Table 602.3(5), 2 × 6 studs can be used at 609.6 mm (24 inch) O.C.; however, it is more typical for the studs to be spaced at 406.4 mm (16 inch)

O.C. The advantage of this is that when using a double 2 × 6 top plate, the joists or trusses that bear on the wall do not have to bear directly on the stud. If using a single top plate or studs spaced at 609.6 mm (24 inch) O.C., then the joists or trusses must either be directly above the stud or within 25.4 mm (1 inch) of the stud according to IRC Section R602.3.2. It is possible to use 2 × 4 studs spaced at 406.4 mm (16 inch) O.C., but this is not common because of the popularity of using fiberglass batts to meet the International Energy Conservation Code (IECC) [7] envelope insulation requirements. The connections between the studs and the plates are according to IRC Table 603.2(1). The connections are typically nails, and the nail sizes vary between 8D and 16D based on the detail.

4.6 Headers within wall system

Structural header members are used to create openings in a load-bearing wall assembly for fenestration (windows and doors) as shown in **Figure 2**. Dimensional lumber headers are preferred by designers when loading is low. Often times when point loading is present on a header or spans are large, an engineered lumber header, such as an LVL, may become cost-effective. An example of a typical LVL is shown in **Figure 4**. LVLs are also often used in wall systems when smaller depth members are required due to space constraints.

When specifying headers, the designer may choose to specify larger headers in some locations for consistency sake. By minimizing the amount of different beam sizes on the plan, the designer reduces the risk of misplacement of headers. As in the case of the roof sheathing, it may also turn out that some beam sizes may be more readily available, and therefore larger sections may be more economical. For example, a two-ply 2 × 8 beam, with a demand capacity ratio of 0.944 controlled by bearing, is adequate for BM3, but because the entire back wall on the first floor is composed of two-ply 2 × 10 headers and all the other headers in the building are 2 × 6's, it makes sense just to specify a two-ply 2 × 10 beam for this location as well. This eliminates the need to have another beam size on site and provides for the opportunity to use trim pieces from a different header cut to make up this short beam.

4.7 Above-grade floor system

In this home design, an engineered floor system will be used. As shown in **Figure 4**, I-joists have become popular and cost-effective in the residential home construction market. I-joists have several advantages over dimensional lumber

Figure 4.
Typical I-joist and LVL (courtesy Timber Rock Homes).

joists, one of which is a greater span-to-depth ratio. This allows for shallower floor assemblies, longer spans, and higher ceilings. I-joists are generally more stable than dimensional lumber. This almost eliminates the need for bridging in a floor system and ensures consistency of engineering properties.

An I-joist floor system is an engineered product. Typically, a designer will send their floor plan along with preliminary input from the designer to the I-joist manufacturer. The manufacturer will then design the floor system according to the requests of the homeowner and designer. Live load deflections are often limited to L/480 (beam span/480). Because longer spans can be achieved by using an I-joist product, the chances of floor vibration occurring increase, but can be controlled, as designers will often restrict deflection to L/480.

It is common for designers to use span tables to select an initial floor joist size. This will provide a fairly accurate estimate and allow the designer to select a floor assembly depth. The improved stability and increased stiffness of I-joists allow designers to consider larger spacing for the floor joists. It is common to specify I-joists at 487.7 mm (19.2 inch) O.C., whereas it was generally common in the past to specify dimensional lumber joists at 406.4 mm (16 inch). Additionally, lumber joists are only available in certain lengths. This made the need for a splice at an internal bearing wall or beam a very common occurrence. The length of I-joists is generally only limited by transportation and site restrictions. An I-joist package will typically arrive at the site precut and ready to be installed with minimal modification.

As in the case of roof sheathing, floor sheathing serves two purposes. First, it acts in the gravity system to distribute floor loads to the joists. Secondly, it is the primary shear resisting component in the floor diaphragm, which will be discussed subsequently. Typically, the gravity loads govern the thickness choice of subflooring, and the shear requirements dictate connection to joists [8].

Once again IRC Table R503.2.1.1(1) will be used to size the sheathing. In this case, the sheathing will serve as both the underlayment and the subflooring. From the table, either 18.3 mm (23/32 inch) or 19.1 mm (3/4 inch) tongue and groove oriented strand board (OSB) sheathing would be appropriate, whichever is more cost-effective and readily available. It is possible that 15.1 mm (19/32 inch) or 15.9 mm (5/8 inch) sheathing could be used, but spans are restricted to 508 mm (20 inch). Although the joists will be specified at 487.7 mm (19.2 inch), which is less than the limit, it is likely that at least a few joists within the floor system will need to be spaced greater than 508 mm (20 inch). An example is when joist bays are used for heating, ventilating, and air conditioning (HVAC) ductwork, the joists are often spread in those locations to 609.6 mm (24 inch). In this instance, the thinner sheathing would be inadequate. IRC Table 602.3(1) specifies attachment of the sheathing to joists with a 50.8 mm (2 inch) 6D deformed nail or a 63.5 mm (2½ inch) 8D common nails spaced at 152.4 mm (6 inch) O.C. around sheathing edges and 304.8 mm (12 inch) O.C. for intermediate field spacing.

Joists for this project are selected from the Trus Joist #TJ4000 specifier's guide [9]. From the span tables within the guide, TJI110 301.6 mm (11 7/8 inch) joists are adequate for both the first and second floors of this residence. The maximum span in the home is approximately 4.70 m (15 foot–5 inch). The TJI110 301.6 mm (11 7/8 inch) joist can span a maximum of 4.90 m (16 foot–1 inch) considering L/480 deflection limit, 1.92 kN/m^2 (40 lbf/ft^2) live load, and a 0.96 kN/m^2 (20 lbf/ft^2) dead load. The TJI 28.6 mm (1 1/8 inch) engineered rim board will be used for the perimeter of the floor system. The rim board serves to transfer compressive and shear loads from the exterior walls above to foundation below. It also acts to enclose the perimeter of the floor system. Typically, joists are toenailed to sill plates at ends and nailing plates at intermediate points. Metal hardware such as that made by USP [10] or Simpson Strong Tie [11] is used to make any flush beam-to-beam or joist-to-beam connections within the floor system. An example would be the stair trimmer detail shown in **Figure 5**.

A double joist or LVL product can be used to function as stair trimmers in an engineered floor system. When loads are low, double joists are economical, but as loading and span increase, an LVL is sometimes needed. LVLs are sometimes used because the installation is cleaner looking and easier to finish than double joists. Double joists often require padding at connections and sometimes bearing, which is usually OSB, to compensate for the space between the web and flanges. LVLs are conveniently made in the same depths as I-joists, which makes it easy to use within the floor systems.

A benefit of using I-joists over dimensional lumber is that it is easier to put holes through the joists for mechanical runs. Most I-joist manufacturers will have predetermined locations or precut holes in the joists where mechanical penetrations are anticipated. Some guidance is typically specified in the manufacturer literature. Holes in dimensional lumber typically require structural analysis and stress evaluation as they become large relative to the depth of the joist or beam.

4.8 Girder sizing

For this example home design, a central steel girder will be used to collect the floor loads and transfer to pad footings in the center of the basement. It is common for designers to use either steel girders or manufactured lumber girders in homes today. These types of girders are much stronger than dimensional lumber beams and are necessary in many instances because of the longer allowable engineered I-joist spans and homeowner request for open basement floor plans. Both manufactured lumber girders and steel girders must be either specified or the design reviewed by a professional engineer.

Steel girders are often chosen over manufactured lumber girders when girder spans are long, head room in the basement is a premium, or steel is readily available. For this particular builder, the head room in the basement is important because they like to advertise their homes with basements that can be finished in the future.

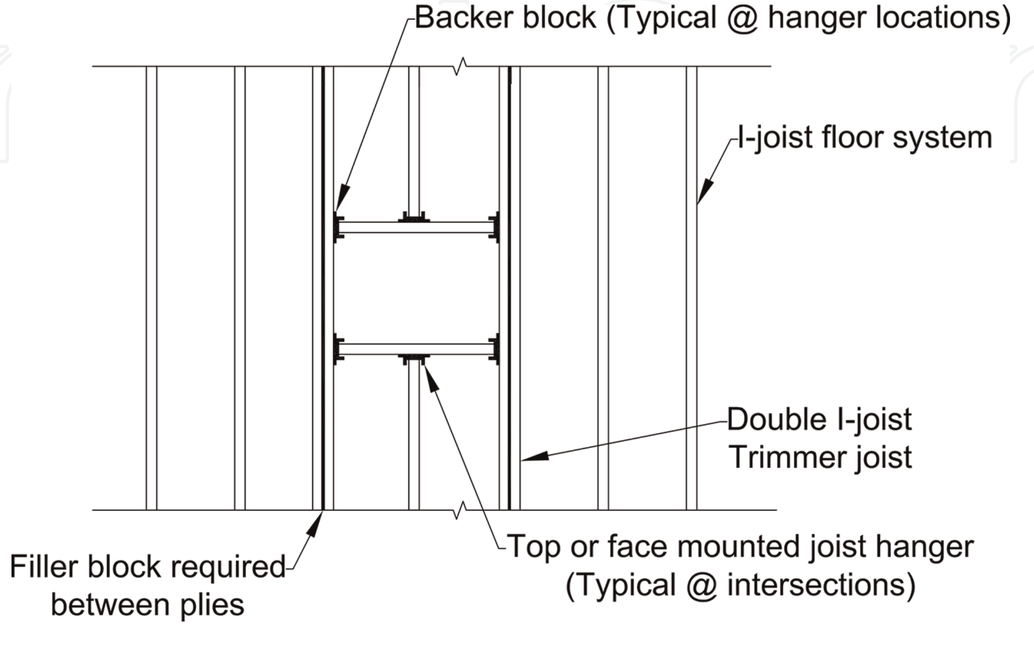


Figure 5.
Stair trimmer detail.

A W8x18 girder works well for them because it's a shallow beam and the flange width is small enough that the beam can fit in a 2 × 6 wall making the girder unnoticeable if the basement is ever finished.

A W8x18 steel girder, with a design moment capacity of 86.5 kN-m (63.8 kip-ft), is more than adequate to resist the internal moment of 31.5 kN-m (23.2 kip-ft) for the controlling load case. It is possible that a smaller girder could have been used, but W8x18 is the minimum size the builder will use. Small sizes tend to have stability issues and can be susceptible to local buckling problems caused by larger point loads. In addition, this is a readily available steel section from the builder's steel supplier.

The design of residential girders involves assumptions regarding the bracing of the beam. The American Institute of Steel Construction (*AISC*) *Steel Construction Manual 14th Ed.(SCM)* in Chapter B3.6, F1 (2) [12] and Appendix 6.3 all require that girders are restrained against rotation about their longitudinal axis at the points of support unless it can be shown that the restraint is not required. The amount of restraint provided by the adjustable column, which is typically four bolts through the bottom flange, may need a detailed analysis because of the slenderness of the columns.

Steel girders in most residential cases are usually ordered in a single length if possible to avoid splices and therefore are continuous over their intermediate supports. Negative moment occurs at the intermediate supports, which puts the bottom flanges in compression in those regions.

If it is assumed that the columns do not provide adequate bottom flange support, then these negative moment regions would be destabilizing, and since inflection points are not typically recognized as a brace point (SCM Appendix 6.3), the unbraced length would have to be taken as the entire beam length of 11.0 m (36 feet), which would require a very large section. Additionally, if no compression flange bracing is assumed at the supports, then the beam fails the concentrated load check in SCM J10.4 for web sidesway buckling. Section J10.4 requires the supports to be adequately braced under these circumstances.

If it is assumed that the column is braced against rotation at the supports by either assuming the column connection is adequate or providing additional bottom flange support, then the unbraced length reduces to the distance between the columns, which in this case is 9′-0″ and the beam passes both strength and concentrated load checks.

Also restraint against rotation should be provided at the ends of the beams, which are seated in the beam pockets. Typically, beam pockets in the concrete wall are oversized to facilitate easy installation of the beams. This creates the opportunity for twisting. SCM Section J10.7 requires all unframed girder ends to have a pair of transverse stiffeners if unrestrained. In this case, a better idea would be to grout the pocket as shown in **Figure 6**, or provide some type of shim, after installation to restrain the end against rotation. It should be noted that the required moisture management and thermal envelope components are not shown for clarity in the figure.

Another consideration for girder sizing is live load pattern loading. Since the girder is a continuous beam having multiple spans, ASCE 7 Section 4.3.3 requires the consideration of pattern loading. In this case, it turns out that applying live loading to spans 1, 2, and 4 only produced the largest internal moment of 31.5 kN-m (23.3 kip-ft) in the beam. **Figure 7** shows the moment diagram for the controlling load combination and the spans that were loaded to produce it.

Pattern loads are considered in the structural analysis software package Enercalc that was used for beam design. Enercalc runs all permutations of live load application and reports the worst-case scenario in envelope format. Data for individual permutations is not able to be extracted. For this example, a separate check was

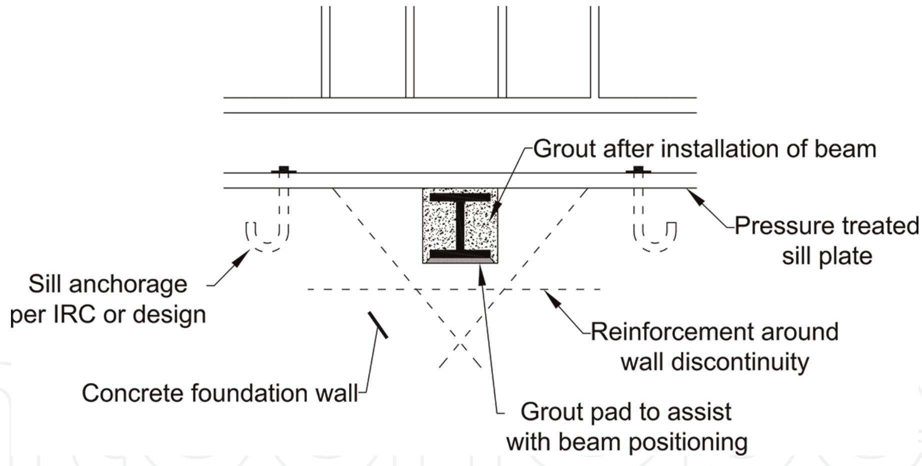


Figure 6.
Steel girder beam pocket detailing.

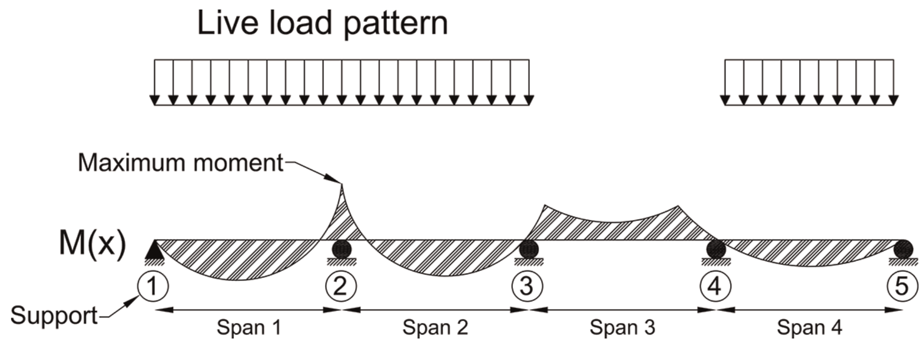


Figure 7.
Moment diagram showing maximum internal moment over support 2.

made using Computers and Structures, Inc. (CSI) SAP2000 finite element modeling software to verify the results of Enercalc and determine the controlling permutation. Results were within 1% of each other between the two analysis packages.

Pattern loading was significant in this example. If only the full intensity live load application was to be considered, then the design moment would have been underestimated by approximately 5%, and the support reactions would have been underestimated by approximately 5% at supports 2, 4, and 12% at support 3. If ignored, this could have led to the undersizing of both adjustable column and pad footing.

4.9 Adjustable columns

Adjustable columns are generally used in residential construction as intermediate supports for basement girders. Adjustable columns are readily available at almost any hardware stores and can be adjusted in height to match site conditions by the contractor. **Figure 8** shows an example of typical adjustable columns. The maximum loading, as reported by the manufacturer, is a factored allowable ASD load capacity (Ra). Reactions determined by ASD load combination can be used to directly size the column from the manufacturers testing data. For this particular home design case, the maximum ASD girder reaction is 80.5 kN (18.1 kip). According to the manufacturers data, an 88.9-mm (3½ inch) and 2.31-mm-thick (11 gauge) column with a height between 2.21 m (7 foot–3 inch) and 2.31 m (7 foot–7 inch) has an allowable load of 95.6 kN (21.5 kip), which is greater than the maximum column axial demand of 80.5 kN (18.1 kip). All three columns

Figure 8.
Typical adjustable column.

will be specified for this maximum loading. This will decrease the chances of misplacing columns.

4.10 Foundation design

A combination of components are used to transfer load from the above-grade portion of the home to the ground. In this home, concrete walls supported by concrete strip footings are used to support the exterior walls and resist lateral earth pressure. Interior loads are transferred by the intermediate girder through columns to concrete pad footings. It is common practice in residential design to specify the foundation walls prescriptively but design the footings. This is the approach that is taken for this study. The American Concrete Institute (ACI) 332-08 [13] and ACI 318-14 [14] are used as references for this design. These documents are adopted by the 2015 IRC and often lead to more economical designs when compared to the requirements of the IRC.

4.11 Foundation walls

Based on soil categorization, the ACI provides prescriptive foundation sizing tables in Appendix A of ACI 332, which are usually appropriate for most situations. For most residential designs, geotechnical exploration and lab testing are cost prohibitive, and therefore soil pressures must be assumed. ASCE 7 provides design lateral soil load that can be used in the absence of site-specific geotechnical information.

For this design, the equivalent soil pressure will be estimated at 2.15 kn/m^2 per linear meter (45 lbf/ft^2 per linear foot). According to ASCE 7 Table 3.2.1, this is representative of a type GC soil (unified soil classification), which is described as a clayey gravel, poorly graded, gravel, and sand mix. Assuming horizontal backfill and a vertical foundation wall, this is roughly equivalent to 19.6 kN/m^3 (125 lbf/ft^3) soil with an internal friction angle of 28 degrees [15].

According to ACI 332 **Table 9**, 21 MPa (3000 psi) is the minimum required compressive strength for foundation walls in the severe weather probability category. Because the concrete will be exposed to weathering, it must be air entrained, having an air content of 6% plus or minus 1.5%.

The concrete foundation wall for the main structure in this example has an unsupported height of 2.44 m (8 foot) and will be subjected to approximately 2.13 m (7 foot) of unsupported backfill when in service (**Figure 9a**). For this situation, considering reinforcing bars with a yield strength of 420 MPa (60,000 psi), ACI 332 Table A.4 allows for the use of a plain concrete (no vertical reinforcing needed) 203.2 mm (8-inch)-thick foundation wall. To minimize shrinkage cracking, however, ACI 332 requires the use of three continuous horizontal bars in the wall. One must be placed within 609.6 mm (24 inch) of the top, one within 609.6 mm (24 inch) of the bottom, and the last one in between the other

Walls	Length required	Length provided	Method
First floor			
N	4.24 (167)	8.23 (324)	CS-WSP
S	3.40 (134)	3.66 (144)	WSP
E	3.20 (126)	3.66 (144)[a]	WSP
W	3.20 (126)	3.66 (144)	WSP
Second floor			
N	1.83 (72)	2.44 (96)	WSP
S	1.83 (72)	2.44 (96)	WSP
E	1.52 (60)	3.66 (144)	WSP
W	1.52 (60)	3.66 (144)	WSP
Garage[b]			
N	1.27 (50)	2.44 (96)	WSP
E	1.32 (52)	1.37 (54)	WSP
W	1.32 (52)	2.44 (96)	WSP

[a]*For WSP methods panel lengths between 0.914 and 1.22 m (36 and 48 inches) are allowed but must be adjusted per IRC Table 602.10.3.*
[b]*The required bracing for the garage/main house common wall will be added directly to the first floor north wall.*

Table 9.
Wall bracing. Values in meters (inches).

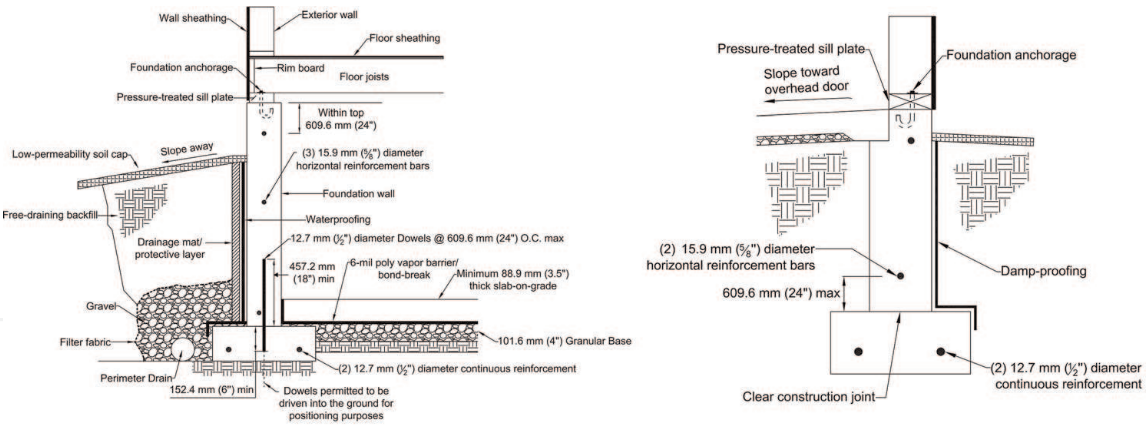


Figure 9.
(a) Typical basement wall and (b) typical garage frost wall.

two. ACI 332 also prescribes 12.7 mm diameter (½ inch) dowel rods at a maximum of 609.6 mm (24 inch) O.C. or a keyway to be provided in this instance since unbalanced backfill height exceeds 1.22 m (4 foot).

The garage wall foundation walls are all 0.91 m (3 feet) in height and have no unbalanced backfill. According to ACI 332, 203.2 mm (8 inch) plain concrete walls are adequate. No vertical reinforcing is necessary, but horizontal reinforcing is still required (**Figure 9b**). The wall height is less than 1.83 m (6 feet), which requires only two 12.7 mm diameter (½ inch) reinforcing bars, one within the top 609.6 mm (24 inch) of the wall height and the other within the bottom 609.6 mm (24 inch) of the wall height. Because the unbalanced backfill is less than 1.22 m (4 feet), Section 6.3.4 allows for the use of a clean construction joint versus dowel rods.

4.12 Wall strip footings

Continuous strip footings will be used to support the exterior foundation walls. The wall footings will be designed (as opposed to prescriptive). No soil testing data is available, so the IRC minimum of 71.8 kN/m^2 (1500 lbf/ft^2) prescribed in Table R401.4.1 will be used for design. The assumption will be made that the footings are not exposed to weathering; therefore, ACI 332 prescribes 17 MPa (2500 psi) minimum compressive strength for the concrete.

For this example, it will be assumed that the load from the exterior wall will act concentrically on the footing. In other words, the footings will be designed for uniform pressure only, and no imbalanced soil pressure due to the presence of a moment will be considered. This is a reasonable assumption because basement walls are typically restrained from translation at the top and bottom by the first floor assembly and the basement slab, respectively. The presence of this restraint allows walls to be designed as a vertical beam with pinned ends (no moment transfer). In addition, the opposing soil exterior lateral loading tends to offset the small amounts of eccentricity created by above-grade wall offsets, so in practice the effects of above-grade wall offsets are generally ignored for wall footing design. **Figure 10** shows an illustration of the analytical model for a typical residential basement wall.

Residential wall footings are typically specified in depths of 152.4 mm (6 inch), 203.2 mm (8 inch), or 254 mm (10 inch), and widths are generally varied in 50.8 mm (2 inch), 76.2 mm (3 inch), or 152.4 mm (6 inch) increments. Both the IRC and ACI 332 allow for the use of 152.4 mm (6-inch)-thick footings (assuming adequate strength), but the developer in this case prefers to use 203.2 mm (8-inch)-thick footings. This allows for some additional safety precaution when plain

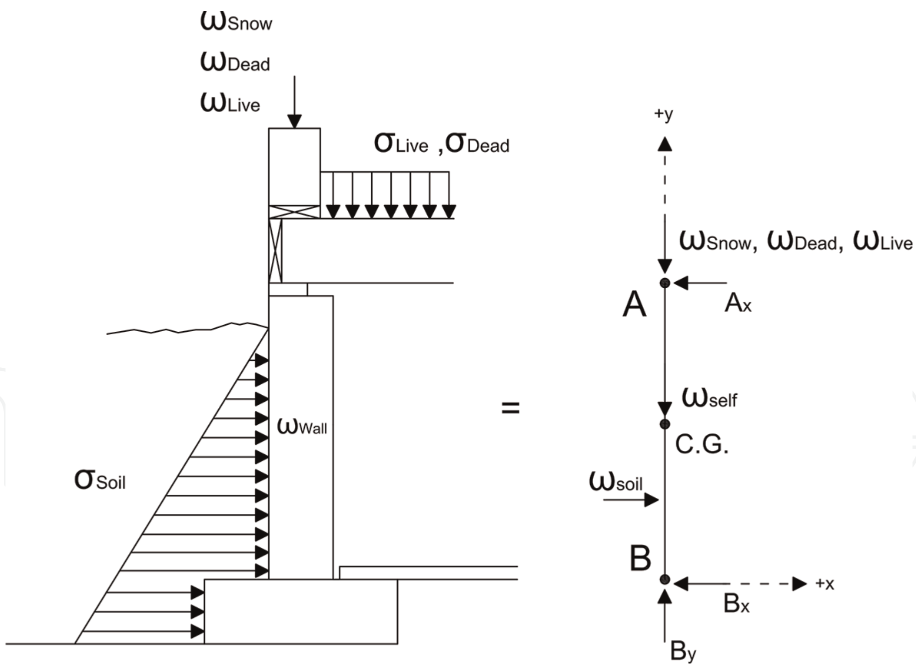


Figure 10.
Free body diagram of a basement wall. Note: the arrows show loads, and small rectangle with x inside indicates the cross section of wood member.

concrete footings are used. When specifying footing widths, this particular developer prefers to use even dimensions in 50.8 mm (2 inch) increments.

In this example, the wall footing design is split into three segments, the main load-bearing walls of the east and west (perpendicular to joist and truss spans), the gable end walls, and the garage walls. Wall footings were designed as plain concrete strip footings according to the requirements of ACI 318, considering the increased modulus of rupture allowed by ACI 332 Chapter 7. Soil bearing pressure controlled all designs. With a soil bearing pressure of approximately 67 kN/m^2 (1400 lbf/ft^2), the bearing walls required 203.2 × 457.2 mm (8 inch by 18 inch) footings. The gable end wall footings and garage footing were able to be reduced to 203.2 × 406.4 mm (8 inch by 16 inch). The wall region beneath the supporting columns for the garage door header controlled the design. Considering ASD load combination 4 and a point load distribution angle of 45 degrees within the concrete wall, the soil pressure beneath the column would be approximately 67 kN/m^2 (1400 lbf/ft^2) as well.

The footings were designed as plain concrete footings. Plain concrete footings are the most economical because of the absence of the steel reinforcing cost. Some developers are comfortable relying on the unreinforced concrete footing to maintain its integrity over the service life of the building, but some prefer to add light reinforcing to help prevent cracking due to unexpected soil discontinuities. ACI 332 Section 6.2.4.1 prescribes the use of two 12.7 mm diameter (½ inch) bars for locations with discontinuities less than 914.4 mm (36 inch) in length.

4.13 Isolated pad footings

Isolated pad footings are typically used to transfer vertical gravity load from interior columns in the basement. In this case, there are three pad footings required to support the interior central steel girder. Interior pad footings are not subjected to weathering, so 17 MPa (2500 psi) concrete compressive strength is adequate. The default value of 71.8 kN/m^2 (1500 lbf/ft^2) is used for the soil bearing capacity, as in the strip footing design.

Reinforced square concrete footings were selected as appropriate for this application. Plain concrete pad footings are sometimes adequate for smaller footings with plan dimension of 609.6 mm (24 inch) or 762 mm (30 inch) square but typically require reinforcement as the plan dimensions of the footing increases. In this case, three 1219.2 mm (4 foot) square footings using four 15.9 mm (5/8 inch) diameter bars in each directions were required. Considering LRFD combination 2, two-way shear (punching shear) with a demand/capacity ratio of 1.30 was the controlling failure mechanism for the concrete footing and required an increase in footing depth from 203.2 mm (8 inch) to 254 mm (10 inch). This reduced the demand/capacity ratio to the acceptable level of 0.698.

4.14 MWFRS design

The typical residential MWFRS system is composed of a system of flexible diaphragms and shear walls. As shown in **Figure 11a**, wind load is transferred from exterior walls perpendicular to the wind direction to structural wood panels, typically OSB or plywood, attached to roof or floor framing. The flexible roof or floor diaphragms, as shown in **Figure 11b**, act similar to a deep beam and distribute the wind load as reactions to the exterior walls parallel to the wind loading (**Figure 11c**) and distribute to the stiff structural shear panels within those walls by direct diaphragm connection or strutting.

The structural wall panels, as shown in **Figure 11d**, provide the necessary shear resistance and transmit the loads vertically (overturning tension and compression

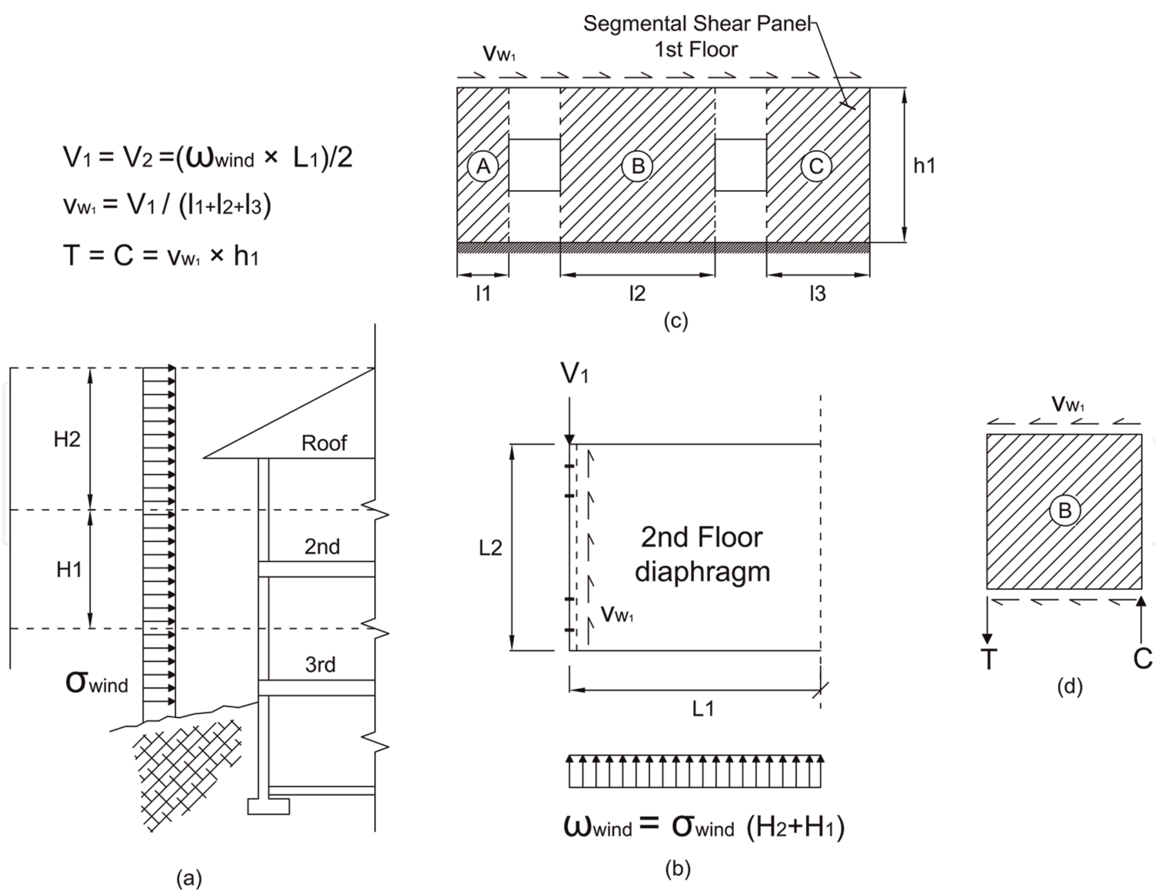


Figure 11.
(a) Wind pressure distributed through external walls to flexible diaphragm. (b) Flexible diaphragm distributes load to parallel walls. (c) An example of a segmental shear wall load distribution approach. (d) Shear wall segment resolution of overturning forces. Note: in this figure, the following notation is used: V for shear force, T for tension force, C for compression force, l and L for Span length, h and H for height, σ for wind pressure, ω for wind load per unit length, and Vw for shear per unit length.

loads at the corners of each structural panel) to the foundation though a system of hold-downs and connections.

Typically, the panels are specified by design aids such as the IRC or the Wood Frame Construction Manual (WFCM). When using the IRC approach, the prescribed nailed connections are assumed to be adequate to transfer the overturning shear forces shown in **Figure 11** to the foundation. If an engineered design or the WFCM prescriptive approach is used to specify shear wall panels, then structural connectors must be specified to transfer these overturning forces. The connection system must have an identifiable load path to the foundation. For this reason, most residential designers use the IRC to specify shear panels and their fastening system. When using a wood truss system as part of the roof diaphragm, such as the one in this home design example, structural connectors are typically specified to transfer the horizontal shear loads and uplift loads resulting from the roof wind loading.

The loads from the shear wall panels and floor diaphragm are transferred to the sole plate by nailed connections and sometimes structural connectors if necessary. The sole plate is attached to the foundation wall with cast-in-place anchors such as J-bolts or post-installed anchorage that must be drilled after the concrete has had time to cure, such as expansion anchors, epoxy anchorage, or screw type. With a prescriptive approach, the prescribed anchor bolts are assumed to adequately transfer both the overturning actions and horizontal actions generated by the wind.

4.15 Overturning and sliding analysis

It's generally good practice to review the whole structure for stability under wind loading and then design the individual components of the lateral force resisting system as required. An overturning and sliding analysis is conducted to determine the required strength of the connections between main assemblies such as the roof-to-wall connections, floor-to-wall connections, and the above-grade building-to-foundation connections.

Many times, homes have attached garages where the garage is not integral to the main living space, such as the one in this example. The garage and the main building can be somewhat treated as separate buildings for the purposes of MWFRS design. The garage can sometimes help resist main building wind loading as long as the wall offsets are not too large; otherwise they must be treated completely separately as far as wall bracing goes. In the east-west direction, the common north wall between the garage and the main structure is generally treated as an exterior wall, and bracing will be prescriptively specified as such, which will act to transfer load from both the garage and the main building.

ASCE 7 Figure 28.6.1 cases A and B were used to determine the magnitude of wind forces applied to the building. The magnitudes of the loads were reported previously in **Tables 6** and 7. The load effects created by the external wind forces were used to specify the hold-downs and shear connectors necessary to maintain continuity of MWFRS load path. The garage was not analyzed, but the procedure would be the same. To simplify the analysis, the end zone loads for case A were applied on both ends to simplify the analysis. To maintain a uniform balanced load in case B (wind applied to the gable end), a weighted average of 0.69 kN/m^2 (14.4 lbf/ft^2) was taken for zones A and C and applied horizontally. An average of zones E and F that was calculated to be -0.95 kN/m^2 (-19.8 lbf/ft^2) was applied vertically to the windward side of the roof, and an average of zones G and H that was calculated to be 0.61 kN/m^2 (12.7 lbf/ft^2) was applied vertically on the leeward side of the roof.

Analysis showed that structural connectors were needed for the roof, but not for the floor-to-floor connections and the foundation connection. Connectors for the truss ends must be able to simultaneously transfer uplift and north-south shear loading as well as shear loading alone in the east-west direction. Simpson Strong Tie (SST) H2.5A hurricane connectors were considered for the truss end-to-top plate connection. This connection resists both shear and uplift. The H2.5A has a shear capacity of 0.58 kN (130 lbf) and uplift capacity of 1.62 kN (365 lbf). The truss end loads are, respectively, 0.18 kN (40 lbf) and 0.27 kN (60 lbf). Applying a unity equation, the demand/capacity ratio is 0.18 kN/0.58 kN + 0.27 kN/1.62 kN = 0.477 < 1.0; therefore, the connector is adequate. An example of a typical truss connector is shown in **Figure 12**. SST A21 angles were considered for the gable end truss-to-top plate connection. This connection is subject to a total shear load of 10.7 kN (2400 lbf) when the wind is applied perpendicular to the gable end. SST A21 has a design capacity of 1.09 kN (245 lbf) per connector; therefore, the required number of connectors will be 10.7 kN/1.09 kN, which gives approximately 10 connectors.

The structure was checked for overturning at the second floor and at the first floor. The weight of the structure was adequate to resist the overturning moment in both locations. Sliding was only checked on the roof to specify the structural connectors. Sliding on the second floor is resisted by the nailed connection between the bottom plate and the floor assembly. Typically, there are sufficient nails engaged to resist the shear force. As for the building-to-foundation connection, there is no reason to expect an extraordinary loading at this junction, so anchor bolts are specified according to IRC Chapter R403.1.6. The I-joist to soleplate toenail connection was not checked in this analysis but should be checked in an actual design.

Figure 12.
Typical truss-to-top plate structural connector.

4.16 Wall bracing

Wall bracing for residential construction typically involves designating sections along the exterior wall length as shear panels. Structural wood panels are used on the exterior side of the wood framing, and gypsum wallboard on the interior provides the shear resistance and load transfer capability. Plywood or OSB is typically used for the wood structural panels. IRC Table 602.3(3) prescribes a 9.5 mm (3/8 inch) minimum structural panel thickness for 406.4 mm (16 inch) O.C. stud spacing; however, the builder prefers a 11.1 mm (7/16-inch)-thick OSB panel, which is required to be fastened to framing using 8D common nails at 152.4 (6 inch) O.C. around the perimeter and 304.8 mm (12 inch) O.C. in the field.

IRC Section R602.10 will be used to specify shear panel length and location along the wall line. Section R602.10 has provisions for various wall bracing methods. The bracing in this home will follow the requirements for the intermittent wood structural panel (WSP) method or one of the continuous sheathing methods. Because this home is categorized in seismic design category A, Section 602.10.1 allows for different methods to be used along different wall lines. Different intermittent methods could even be used along the same wall line in this category, but if using any of the continuous sheathing methods, the whole wall line must be continuously sheathed.

For the design of this home, it was more economical to use the WSP method for the majority of the shear panels. Section R602.10 requires 609.6 mm (24 inch) corner returns or braced panels at the end of each wall. At least one of the corners does not meet this criterion. When this occurs and the designer is using the continuously sheathed wood structural panel (CS-WSP) method, Section 602.10.4.4 requires the use of 3.56 kN (800 lbf) hold-down devices in lieu of a 2 foot corner return. This is often costlier than the extra amount of sheathing required for the WSP method. Another issue to consider when specifying wall bracing is the stud spacing. In this home, the studs are spaced at 406.4 mm (16 inch O.C.); therefore, it is prudent to specify shear panels 406.4 mm (16 inch) increments, even though the requirements may be less. The location of the shear panels is specified in the drawing set located in Appendix A.

4.17 Horizontal floor diaphragms

The floor assembly is treated as a flexible diaphragm when transferring lateral loading. Wind is transferred from a tributary area of the exterior wall to the rim board of the floor assembly and then into the structural sheathing. The floor sheathing then transfers that load to the exterior shear walls (structural panels within the wall system) parallel to the wind direction below the floor assembly. The diaphragm is treated like a deep beam for the purposes of analysis. The reactions are the connections with wall below. The floor assembly deflects, which causes tension and compression forces called chord forces in the walls below, which are perpendicular to the wind loading. The sheathing layout and the attachment of the sheathing to the I-joists have the greatest effect on the strength of the diaphragm. In this case, the floor sheathing and the required nailing were specified from the IRC in the floor assembly section of this report.

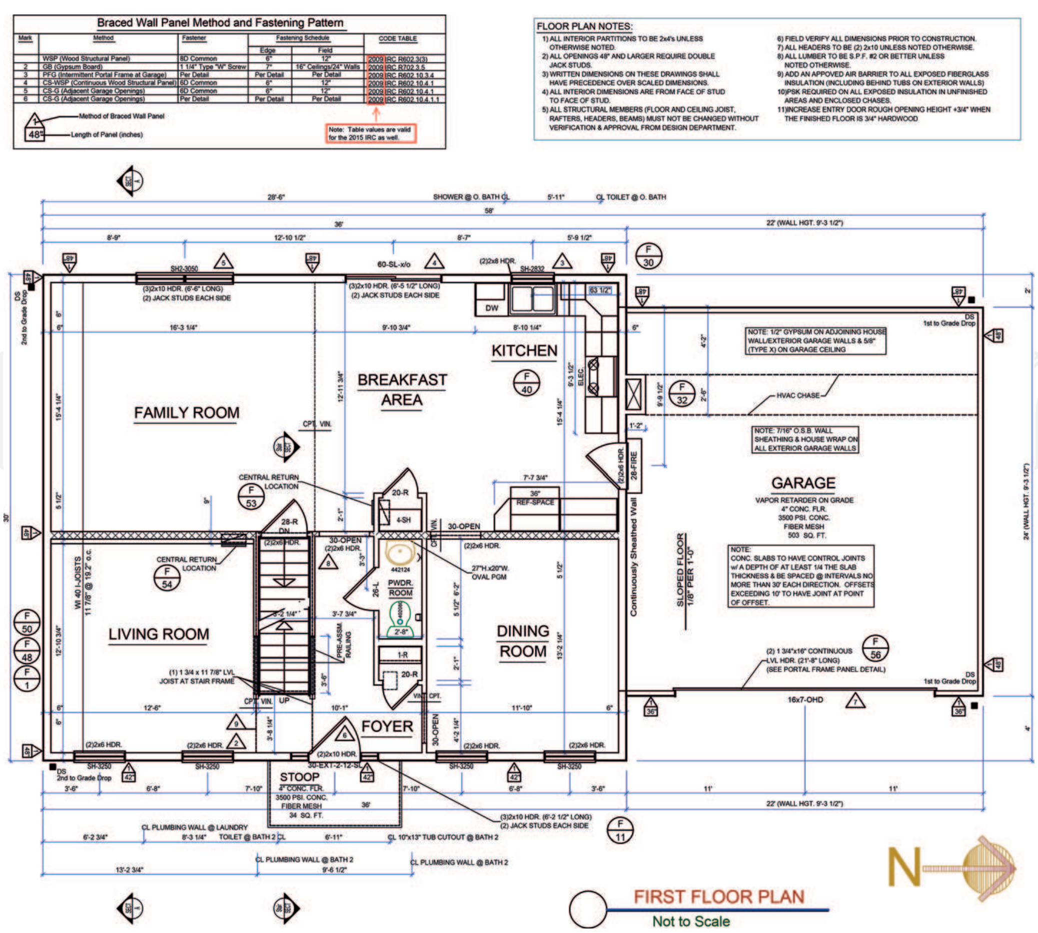
4.18 Connections

Most connections in wood-framed homes are made up of nailed connections. The majority of the connections in a typical home can be found in IRC Table R602.3. The items specified from the IRC in this wood-framed section are based on compliance with this table. In this study, only a few of the typical critical connections for the structural system were specified.

5. Concluding remarks

This chapter presented a complete design of a typical US single-family home made of conventional wood-frame system. Initially, the applicable building and material codes were introduced and relevant provisions discussed. A typical home plan by a PA builder was discussed and explained for detailed design. The process of load selection and load path and load combination was discussed. Then based on application of the resultant loads on typical structural elements, detailed designs for roof sheathing, roof trusses, exterior walls, main wind force resisting system, floor system, girders, columns, and foundation walls and footings were presented and discussed. Where appropriate, tips and guidelines for typical design were offered so that the procedure presented can be followed by designers as appropriate. While other structural systems are becoming increasingly available, the wood-frame system is still the dominating system as in the USA lumber is readily available at highly competitive process. This makes structural systems other than conventional wood-frame less competitive, unless there are special conditions where cost may not be the main determining factor.

Appendix: design drawings



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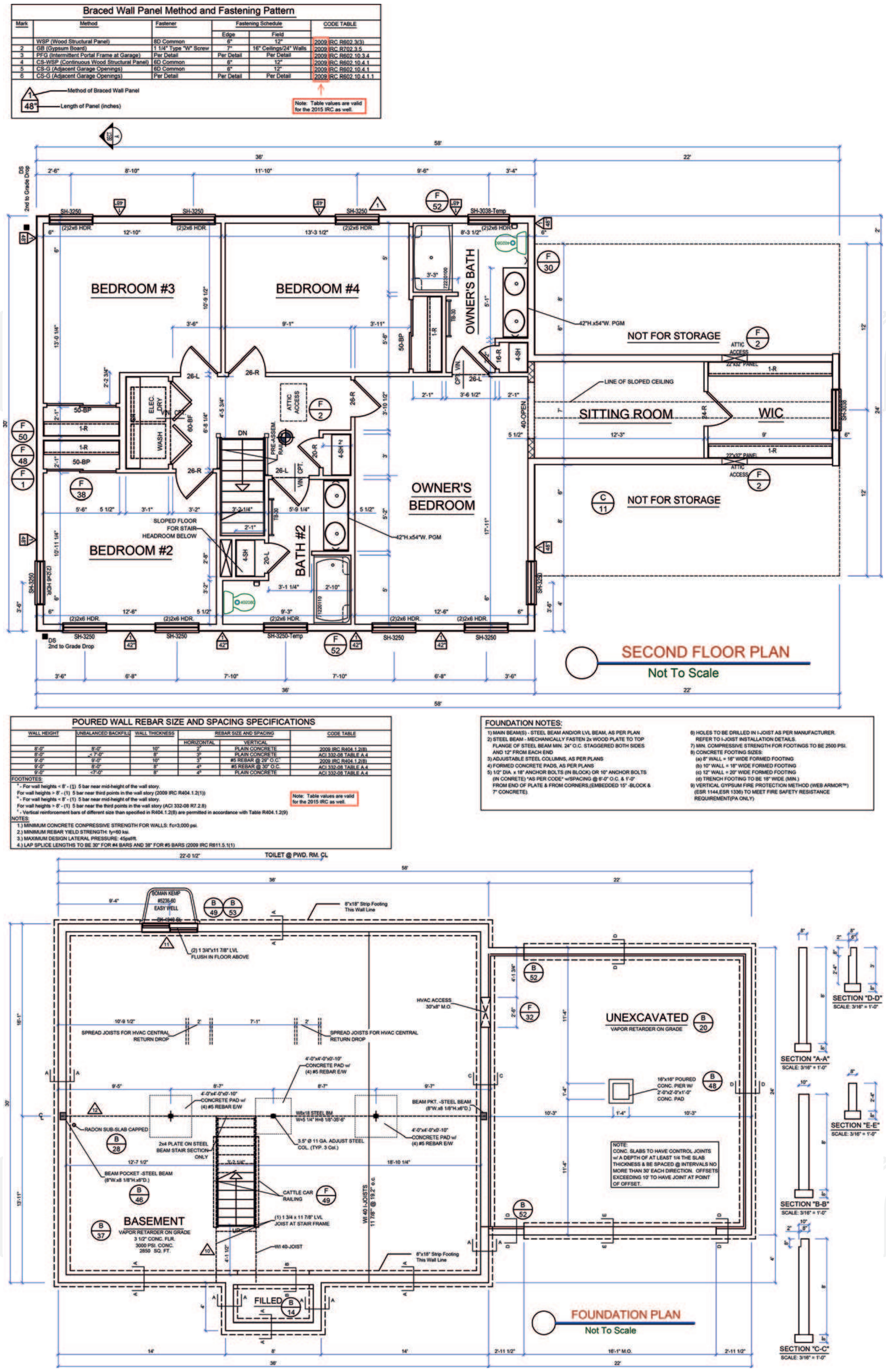

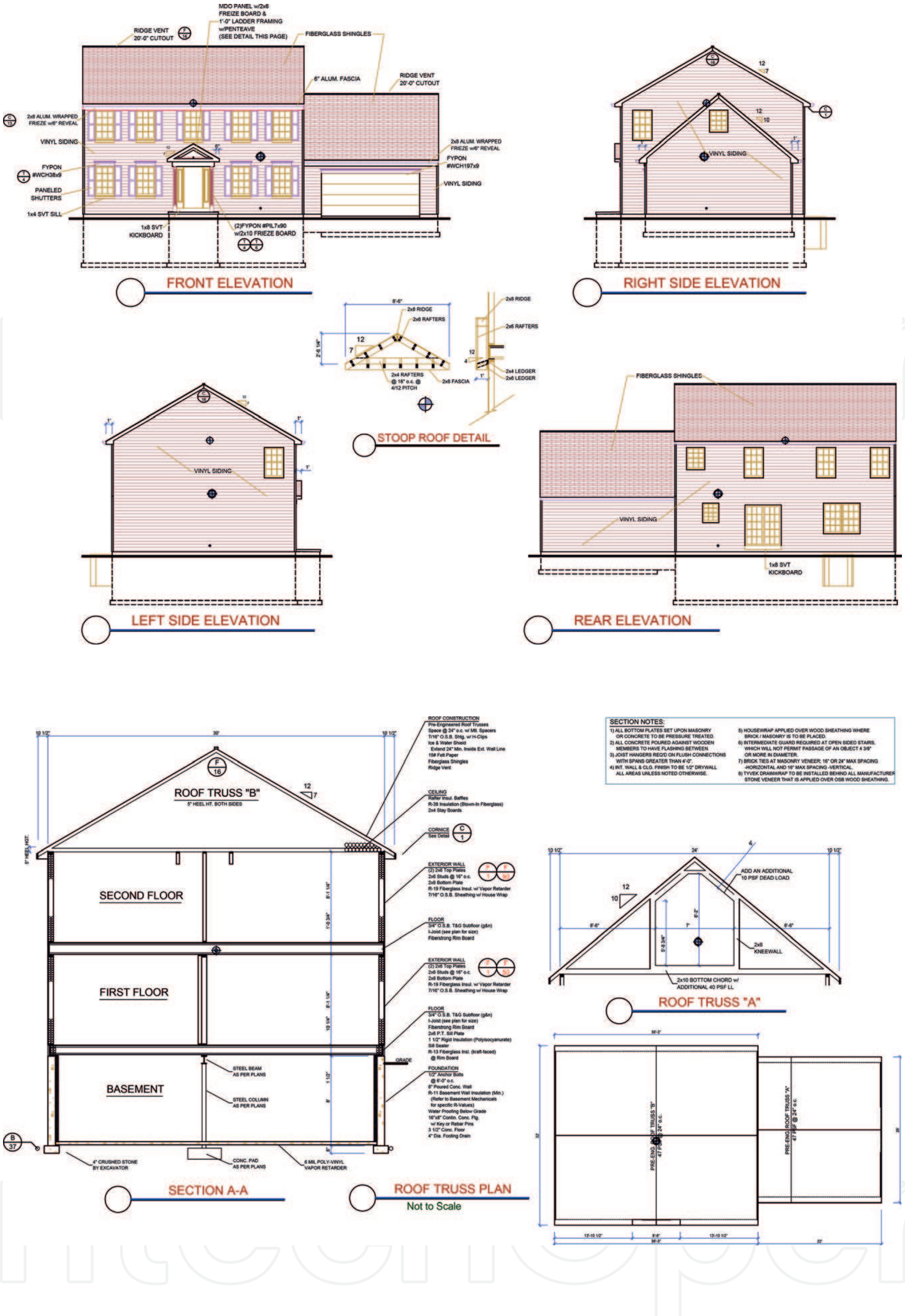
FRONT ELEVATION
RIGHT SIDE ELEVATION
STOOP ROOF DETAIL
LEFT SIDE ELEVATION
REAR ELEVATION
ROOF TRUSS "B"
SECOND FLOOR
FIRST FLOOR
BASEMENT
SECTION A-A
ROOF TRUSS "A"
ROOF TRUSS PLAN
Not to Scale
SECTION NOTES:

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