APPENDIX A

ILLUSTRATIVE EXAMPLE

Purpose

Appendix A contains a design example illustrating the proper application of the different standards and specifications in the *Prescriptive Method*. It provides a step-by-step procedure on how to apply the requirements of the *Prescriptive Method* when designing a home. A typical residential building is used to demonstrate the application of insulating concrete form construction requirements.

Information is presented in both U.S. customary units and International System (SI) units. Reinforcement bar sizes are presented in U.S. customary units; refer to Appendix C for the corresponding reinforcement bar size in SI units.

Building Design Criteria

The example building has the following characteristics:

Building type:	Two-story house (above an unfinished basement) with a center load-bearing beam supporting the first floor and a center load-bearing wall supporting the second floor. Roof trusses bear on exterior walls only.
Building site:	Enclosed building sited in Exposure B
Building width:	30 feet (9.1 m) perpendicular to ridge
Building length:	60 feet (18.3 m) parallel to ridge
Maximum unbalanced backfill height:	6 feet (1.8 m)
Soil-bearing capacity:	2,500 psf (120 kPa)
Basement wall height:	8 feet (2.4 m)
First story wall height:	9 feet (2.7 m)
Second story wall height:	8 feet (2.4 m)
Basement wall ICF type:	6-inch- (152-mm-) thick flat ICF concrete wall
First-story wall ICF type:	4.5-inch- (114-mm-) thick flat ICF concrete wall
Second-story wall ICF type:	4.5-inch- (114-mm-) thick flat ICF concrete wall
Floor joists:	Wood joists spaced at 24 inches (610 mm) on center
Roof framing:	Wood trusses spaced at 24 inches (610 mm) on center
Roof slope:	8:12

The following design criteria are applicable to the example home:

Ground Snow Load psf (kPa)	3-Second Gust Wind Speed (mph) mph (km/hr)	Seismic Design Category	First-Floor Live Load psf (kPa)	Second-Floor Live Load psf (kPa)	Floor Dead Load psf (kPa)	Equivalent Fluid Density pcf (kg/m^3)	Soil-Bearing Capacity psf (kPa)
50 (2.4)	110 (177)	B	40 (1.9)	30 (1.4)	10 (0.5)	30 (481)	2,500 (120)

Building elevations are shown in Figure A1.1 on the next page.

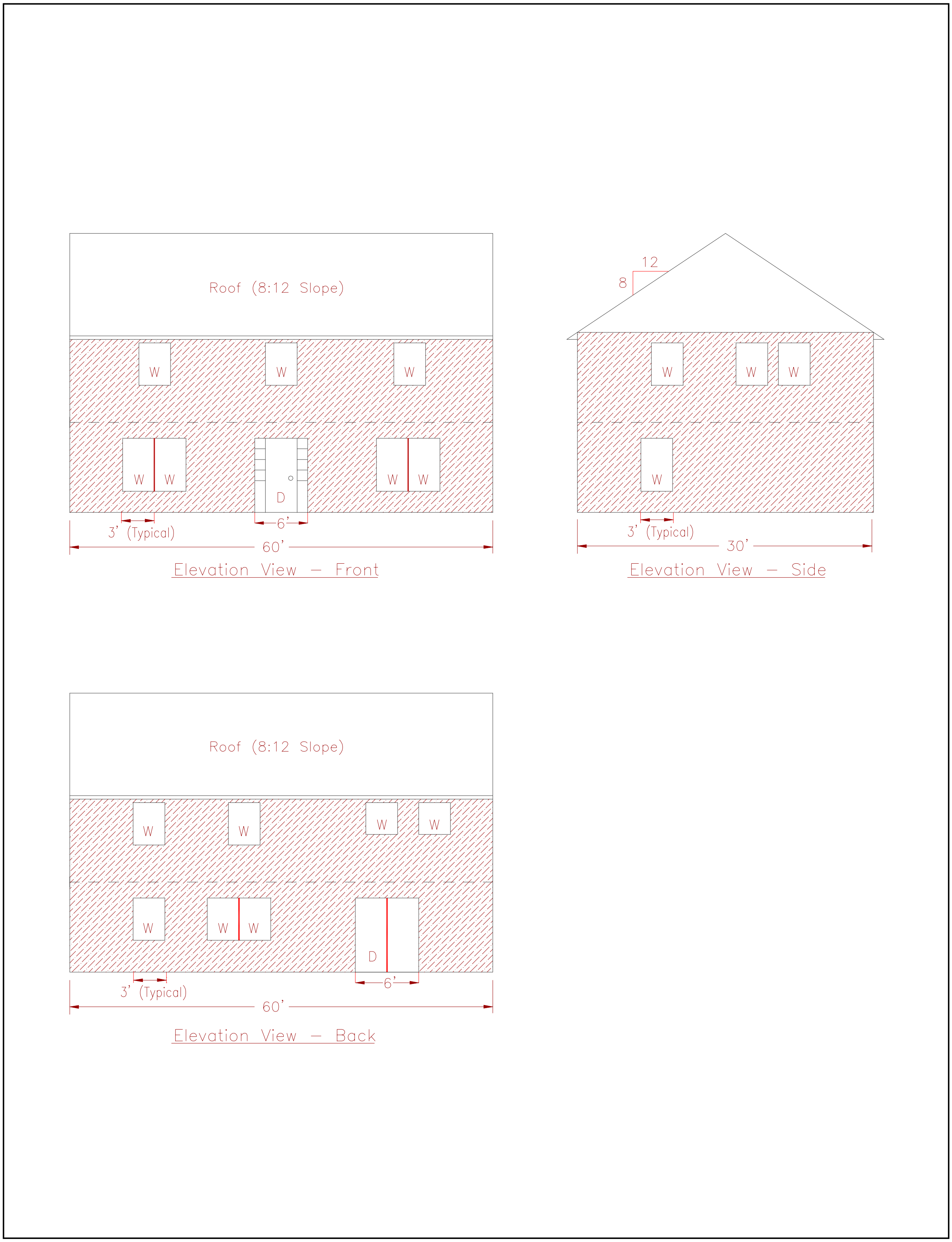


Figure A1.1 Building Elevations

Building Design Requirements Summary

The list below summarizes the requirements that result from applying the *Prescriptive Method* to the given building. A detailed description of the process is given in the following sections. Connection requirements are not highlighted in this Appendix since adequate details and tables are provided in the *Prescriptive Method.* The omission of detailed connection requirements from this Appendix is not, however, intended to diminish the importance of proper anchoring. The user should devote substantial efforts to connection requirements, which cannot be adequately conveyed in one building example.

TABLE A1.1
SUMMARY OF ILLUSTRATIVE EXAMPLE CALCULATIONS
FOR EXTERIOR WALLS

Framing Description	Type	Thickness/ Size	Vertical Reinforcement Requirement		Horizontal Reinforcement Requirement[1]	Reference
Footings	Concrete	16 inches by 6 inches	Grade 40	N/A	N/A	Section 2.0 Section 3.0 Table 3.1
			Grade 60	N/A		
Basement Walls	Flat ICF	6 inches thick	Grade 40	#3@12", #4@22", #5@30", or #6@40"	One No. 4 bar within 12 inches of the top of the wall story and one No. 4 bar near mid-height of the wall story	Section 2.0 Section 3.0 Table 3.3 Table 3.4
			Grade 60	#3@18", #4@33", #5@45", or #6@60"		
First-Story Walls	Flat ICF	4.5 inches thick	Grade 40	#4@34" or #5@48"	One No. 4 bar within 12 inches of the top of the wall story and one No. 4 bar near third points of the wall story	Section 2.0 Section 4.0 Table 4.1 Table 4.2
			Grade 60	#4@48"		
Second-Story Walls	Flat ICF	4.5 inches thick	Grade 40	#4@42"	One No. 4 bar within 12 inches of the top of the wall story and one No. 4 bar near third points of the wall story	Section 2.0 Section 4.0 Table 4.1 Table 4.2
			Grade 60	#4@48"		
First-Story Solid Wall Length	Flat ICF	4.5 inches thick	30' End Wall requires 8.7 feet of solid wall length 60' Side Wall requires 5.6 feet of solid wall length			Section 5.0 Table 5.2B Table 5.2C
Second-Story Solid Wall Length	Flat ICF	4.5 inches thick	30' End Wall requires 6.2 feet of solid wall length 60' Side Wall requires 4.8 feet of solid wall length			Section 5.0 Table 5.2B Table 5.2C

FOR SI: 1 foot = 0.3048 m; 1 inch = 25.4 mm

[1]The minimum horizontal reinforcement requirements are for both Grade 40 and Grade 60 reinforcement.

TABLE A1.2
SUMMARY OF ILLUSTRATIVE EXAMPLE CALCULATIONS
FOR LINTELS

Framing Description	Type	Thickness/ Size	Stirrup/Shear Reinforcement Requirement		Horizontal Reinforcement Requirement	Reference
			Concrete	Requirements		
First-Story 6-foot Lintel[1]	Flat ICF	24 inches deep by 4.5 inches thick	2,500 psi	No. 3 Stirrups @ 11” except for middle 1’ – 8” of span	Minimum Grade 40 - No. 4 Reinforcing bar	Section 5.3 Tables 5.6, 5.7, 5.8A & 5.12
			3,000 psi	No. 3 Stirrups @ 11” except for middle 1’ – 9” of span		
			4,000 psi	No. 3 Stirrups @ 11” except for middle 2’ – 1” of span		
First-Story 3-foot Lintel	Flat ICF	24 inches deep by 4.5 inches thick	2,500 psi 3,000 psi 4,000 psi	Stirrups are not required	Minimum Grade 40 - No. 4 Reinforcing bar	Section 5.3 Tables 5.6, 5.7, 5.8A & 5.12
Second-Story 3-foot Lintel	Flat ICF	12 inches deep by 4.5 inches thick	2,500 psi 3,000 psi 4,000 psi	Stirrups are not required	Minimum Grade 40 - No. 4 Reinforcing bar	Section 5.3 Tables 5.6, 5.7, 5.8A & 5.12

FOR SI: 1 foot = 0.3048 m; 1 inch = 25.4 mm

Footings

The design house is 30 feet (9.1 m) wide, the soil-bearing capacity is 2,500 psf (120 kPa), and the thickness of the basement wall is 6 inches (152 mm). The basement is supporting two ICF stories above. The minimum footing size for these conditions is established from Table 3.1 of the *Prescriptive Method* as 16 inches (406 mm) wide by 6 inches (152 mm) thick. The footings require a minimum concrete compressive strength of 2,500 psi (17 MPa).

TABLE 3.1
MINIMUM WIDTH OF ICF AND CONCRETE
FOOTINGS FOR ICF WALLS[1,2,3]
(excerpt from the *Prescriptive Method*)

MAXIMUM NUMBER OF STORIES[4]	MINIMUM LOAD-BEARING VALUE OF SOIL (psf)				
	2,000	2,500	3,000	3,500	4,000
5.5-Inch Flat, 6-Inch Waffle-Grid, or 6-Inch Screen-Grid ICF Wall Thickness[5]					
One Story[6]	15	12	10	9	8
Two Story[6]	20	**16**	13	12	10

ICF Basement Walls

The building is 30 feet (9.1 m) wide and is subject to lateral soil pressure with an equivalent fluid density of 30 pcf (481 kg/m^3). It has a maximum unbalanced backfill height of 6 feet (1.8 m), and its basement wall is 8 feet (2.4 m) in height. The minimum horizontal wall reinforcement in accordance with Table 3.3 of the *Prescriptive Method* is one No. 4 bar within 12 inches (305 mm) of the top of the wall story and one No. 4 bar near mid-height of the wall story. The ICF basement walls are assumed to be laterally supported at the top of the wall by the floor joists as required in Section 3.0 of the *Prescriptive Method*. In addition, using Table 3.4 of the *Prescriptive Method*, the basement walls require minimum vertical wall reinforcement of one No. 3 bar, one No. 4 bar, one No. 5 bar, or one No. 6 bar spaced at 12 inches (305 mm), 22 inches (559 mm), 30 inches (762 mm), and 40 inches (1.0 m) on center, respectively, for Grade 40 reinforcement. However, if Grade 60 reinforcement is used, basement walls require a minimum vertical reinforcement of one No. 3 bar, one No. 4 bar, one No. 5 bar, or one No. 6 bar spaced at 18 inches (457 mm), 33 inches (838 mm), 45 inches (1.1 m), or 60 inches (1.5 m) on center, respectively.

Basement walls require a minimum compressive strength of concrete of 2,500 psi (17.2 MPa). An increased minimum concrete compressive strength of 3,000 psi (20.6 MPa) or 4,000 psi (27.6 MPa) does not affect the reinforcement requirements for ICF basement walls.

TABLE 3.3
MINIMUM HORIZONTAL WALL REINFORCEMENT FOR ICF BASEMENT WALLS
(excerpt from the *Prescriptive Method*)

Maximum Height of Basement Wall feet (meters)	Location of Horizontal Reinforcement
8 (2.4)	**One No. 4 bar within 12 inches (305 mm) of the top of the wall story and one No. 4 bar near mid-height of the wall story**
9 (2.7)	One No. 4 bar within 12 inches (305 mm) of the top of the wall story and one No. 4 bar near third points in the wall story

TABLE 3.4
MINIMUM VERTICAL WALL REINFORCEMENT FOR 5.5-INCH- (140-MM-)THICK FLAT ICF BASEMENT WALLS [1,2,3]
(excerpt from the *Prescriptive Method*)

MAX. WALL HEIGHT (feet)	MAXIMUM UNBALANCED BACKFILL HEIGHT[4] (feet)	MINIMUM VERTICAL REINFORCEMENT		
		MAXIMUM EQUIVALENT FLUID DENSITY 30 pcf	MAXIMUM EQUIVALENT FLUID DENSITY 45 pcf	MAXIMUM EQUIVALENT FLUID DENSITY 60 pcf
8	4	#4@48”	#4@48”	#4@48”
	5	#4@48”	#3@12”; #4@22”; #5@32”; #6@40”	#3@8”; #4@14”; #5@20”; #6@26”
	6	**#3@12”; #4@22”; #5@30”; #6@40”**	#3@8”; #4@14”; #5@20”; #6@24”	#3@6”; #4@10”; #5@14”; #6@20”

ICF First-Story Walls

The building is 30 feet (9.1 m) wide and is subject to a 3-second gust wind speed of 110 mph (177 km/hr) and a ground snow load of 50 psf (2.4 kPa). The building is enclosed and sited in Exposure Category B. The first-story walls are 9 feet (2.7 m) in height. In accordance with Table 4.1 of the *Prescriptive Method*, the wind pressure used to design the minimum vertical reinforcement requirements for the above grade walls is 29 psf (1.4 kN/m^2). Since interpolation is not permitted in Table 4.2 a wind pressure of 30 psf (1.5 kN/m^2) must be used. Using Table 4.2 of the *Prescriptive Method*, we find that 4.5-inch- (114-mm-) thick first-story flat ICF walls supporting an ICF second story and light-frame roof requires a minimum of one vertical No. 4 or one No. 5 bar spaced at 34 inches (863 mm) and 48 inches (1.2 m) respectively on center for Grade 40 reinforcement or one vertical No. 4 bar spaced 48 inches (1.2 m) on center for Grade 60 reinforcement. Section 4.1 requires horizontal wall reinforcement in the form of one No. 4 rebar within 12 inches (305 mm) from the top of the wall, one No. 4 rebar within 12 inches (305 mm) from the finish floor, and one No. 4 rebar near one-third points throughout the remainder of the wall.

The first-story walls also require a minimum compressive strength of concrete of 2,500 psi (17.2 MPa). An increased minimum concrete compressive strength of 3,000 psi (20.6 MPa) or 4,000 psi (27.6 MPa) does not affect the reinforcement requirements for ICF first-story walls.

TABLE 4.1
DESIGN WIND PRESSURE FOR USE WITH MINIMUM VERTICAL WALL REINFORCEMENT TABLES FOR ABOVE GRADE WALLS[1]
(excerpt from the *Prescriptive Method*)

WIND Speed (mph)	DESIGN WIND PRESSURE (PSF)					
	Enclosed[2]			PARTIALLY ENCLOSED[2]		
	Exposure[3]			Exposure[3]		
	B	C	D	B	C	D
85	18	24	29	23	31	37
90	20	27	32	25	35	41
100	24	34	39	31	43	51
110	**29**	41	48	38	52	61

TABLE 4.2
MINIMUM VERTICAL WALL REINFORCEMENT
FOR FLAT ICF ABOVE-GRADE WALLS [1,2,3,4]
(excerpt from the *Prescriptive Method*)

Design Wind Pressure (Table 4.1)	Maximum Wall Height per Story	Minimum Vertical Reinforcement[4,5]					
		Supporting Roof or Non-Load Bearing Wall		Supporting Light-Frame Second Story and Roof		Supporting ICF Second Story and Light-Frame Roof	
		Minimum Wall Thickness (inches)					
(psf)	(feet)	3.5	5.5	3.5	5.5	3.5	5.5
20	8	#4@48	#4@48	#4@48	#4@48	#4@48	#4@48
	9	#4@48	#4@48	#4@48	#4@48	#4@48	#4@48
	10	#4@38	#4@48	#4@40	#4@48	#4@42	#4@48
30	8	#4@42	#4@48	#4@46	#4@48	#4@48	#4@48
	9	#4@32; #5@48	#4@48	#4@34; #5@48	#4@48	**#4@34; #5@48**	#4@48

ICF Second-Story Walls

Applying the same design wind pressure to the second-story walls as for the first-story walls, second-story walls require a minimum vertical wall reinforcement of one No. 4 bar spaced at 42 inches (1.0 m) on center for Grade 40 reinforcement or one No.4 bar spaced 48 inches (1.2 m) on center for Grade 60 reinforcement for flat walls with a thickness of 4.5 inches (114 mm) supporting a light-frame roof in accordance with Table 4.2. Section 4.1 requires horizontal wall reinforcement in the form of one No. 4 rebar within 12 inches (305 mm) from the top of the wall, one No. 4 rebar within 12 inches (305 mm) from the finish floor, and one No. 4 rebar near one-third points throughout the remainder of the wall.

The second-story walls also require a minimum compressive strength of concrete of 2,500 psi (17.2 MPa). An increased minimum concrete compressive strength of 3,000 psi (20.6 MPa) or 4,000 psi (27.6 MPa) does not affect the reinforcement requirements for ICF walls.

TABLE 4.2
MINIMUM VERTICAL WALL REINFORCEMENT
FOR FLAT ICF ABOVE-GRADE WALLS [1,2,3,4]
(excerpt from the *Prescriptive Method*)

Design Wind Pressure (Table 4.1)	Maximum Wall Height per Story	Minimum Vertical Reinforcement[5,6]					
		Supporting Roof or Non-Load Bearing Wall		Supporting Light-Frame Second Story and Roof		Supporting ICF Second Story and Light-Frame Roof	
		Minimum Wall Thickness (inches)					
(psf)	(feet)	3.5	5.5	3.5	5.5	3.5	5.5
20	8	#4@48	#4@48	#4@48	#4@48	#4@48	#4@48
	9	#4@48	#4@48	#4@48	#4@48	#4@48	#4@48
	10	#4@38	#4@48	#4@40	#4@48	#4@42	#4@48
30	8	**#4@42**	#4@48	#4@46	#4@48	#4@48	#4@48
	9	#4@32; #5@48	#4@48	#4@34; #5@48	#4@48	#4@34; #5@48	#4@48

Minimum Length of ICF Wall without Openings

To determine the minimum percentage of solid wall length required for the example house, Section 5.0, Table 5.1, Table 5.2A, Table 5.2B, and Table 5.2C of the *Prescriptive Method* are used. The design house is a two-story home, 30 feet (9.1 m) wide by 60 feet (18.3 m) long. It has a roof slope of 8:12 and is located in an Exposure B 110 mph (177 km/hr) wind area and Seismic Design Category B. Each above-grade wall story is 4.5 inches (114 mm) thick.

In accordance with Table 5.1 of the *Prescriptive Method*, the velocity pressure used to design the minimum length of ICF without openings for the above grade walls is 23 psf (1.1 kN/m^2).

TABLE 5.1
VELOCITY PRESSURE FOR DETERMINATION OF MINIMUM SOLID WALL LENGTH[1]
(excerpt from the *Prescriptive Method*)

Wind Speed (mph)	VELOCITY PRESSURE (PSF)		
	Exposure[2]		
	B	C	D
85	14	19	23
90	16	21	25
100	19	26	31
110	**23**	32	37

FIRST-STORY WALLS

End Wall Design

Wind

The minimum length of solid end wall is based on the wind velocity pressure from Table 5.1, the building side wall length, and the roof slope. Interpolation is required to obtain the accurate amount of solid end wall length required from Table 5.2B.

For the 7:12 roof pitch and 23 psf (1.1 kN/m^2), interpolating between 9.0 feet (25 psf) and 7.75 feet (20 psf) results in 5 equal increments of 0.25 feet; (9.0 ft – 7.75 ft)/5 = 0.25 ft. Thus, 9.0 feet (25 psf) minus 2 x 0.25 ft (2 psf) results in *8.5 feet* for 23 psf (1.1 kN/m^2).

For the 12:12 roof pitch and 23 psf (1.1 kN/m^2), interpolating between 10.0 feet (25 psf) and 8.75 feet (20 psf) results in 5 equal increments of 0.25 feet; (10.0 ft – 8.75 ft)/5 = 0.25 ft. Thus, 10.0 feet (25 psf) minus 2 x 0.25 ft (2 psf) results in *9.5 feet* for 23 psf (1.1 kN/m^2).

For 8:12 roof pitch, interpolating between 8.5 ft (7:12) and 9.5 ft (12:12) results in 5 equal increments of 0.2 feet; (9.5 ft – 8.5 ft)/5 = 0.2 ft. Thus, 8.5 feet (7:12) plus 0.2 feet indicates that the minimum solid wall length required is *8.7 feet* for the 8:12 roof pitch.

However, a conservative value from the 12:12 roof pitch and 25 psf velocity pressure resulting in a minimum solid wall length of 10 feet (3.0 m) may be used without interpolating.

TABLE 5.2B
MINIMUM SOLID END WALL LENGTH REQUIREMENTS FOR FLAT ICF WALLS (WIND PERPENDICULAR TO RIDGE)[1,2,3,4,5]
(excerpt from the *Prescriptive Method*)

Design Velocity Pressure (psf)			20	25	30	35	40	45	50	60
Wall Category	Building Side Wall Length, L (feet)	Roof Slope	Minimum Solid Wall Length on Building End Wall, W (feet)							
First Story of Two-Story	16	≤ 1:12	4.00	4.25	4.50	4.75	5.00	5.25	5.25	5.75
		5:12	4.50	4.75	5.00	5.25	5.50	5.75	6.00	6.75
		7:12	4.50	5.00	5.25	5.75	6.00	6.25	6.75	7.25
		12:12	5.00	5.25	5.75	6.25	6.50	7.00	7.25	8.25
	24	≤ 1:12	4.50	4.75	5.00	5.25	5.50	5.75	6.00	6.75
		5:12	4.75	5.25	5.50	6.00	6.25	6.75	7.00	7.75
		7:12	5.25	5.75	6.25	6.75	7.00	7.50	8.00	9.00
		12:12	5.50	6.25	6.75	7.25	8.00	8.50	9.00	10.25
	32	≤ 1:12	4.75	5.00	5.50	5.75	6.25	6.50	6.75	7.50
		5:12	5.25	5.75	6.25	6.75	7.25	7.50	8.00	9.00
		7:12	5.75	6.50	7.00	7.75	8.25	9.00	9.50	10.75
		12:12	6.25	7.00	7.75	8.50	9.25	10.00	10.75	12.25
	40	≤ 1:12	5.00	5.50	5.75	6.25	6.75	7.25	7.50	8.50
		5:12	5.50	6.25	6.75	7.25	8.00	8.50	9.00	10.25
		7:12	6.25	7.00	7.75	8.75	9.50	10.25	11.00	12.50
		12:12	7.00	8.00	8.75	9.75	10.75	11.50	12.50	14.25
	50	≤ 1:12	5.50	6.00	6.50	7.00	7.50	8.00	8.50	9.50
		5:12	6.00	6.75	7.50	8.25	9.00	9.75	10.50	11.75
		7:12	7.00	8.00	9.00	10.00	10.75	11.75	12.75	14.50
		12:12	7.75	9.00	10.00	11.25	12.25	13.50	14.75	17.00
	60	≤ 1:12	5.75	6.50	7.00	7.50	8.25	8.75	9.50	10.75
		5:12	6.75	7.50	8.25	9.25	10.00	10.75	11.75	13.25
		7:12	**7.75**	**9.00**	10.00	11.00	12.25	13.25	14.50	16.75
		12:12	**8.75**	**10.00**	11.50	12.75	14.00	15.50	16.75	19.50

Side Wall Design

Wind

The minimum length of solid side wall length is based on the wind velocity pressure from Table 5.1 and the building end wall length. Interpolation between the velocity pressure and building end wall length is required to obtain the accurate amount of solid end wall length required from Table 5.2C.

For the 24 ft building end wall length and 23 psf (1.1 kN/m^2), interpolating between 5.25 feet (25 psf) and 4.75 feet (20 psf) results in 5 equal increments of 0.1 feet; (5.25 ft – 4.75 ft)/5 = 0.1 ft. Thus, 5.25 feet (25 psf) minus 2 x 0.1 ft (2 psf) results in *5.05 feet*.

For the 32 ft building end wall length and 23 psf (1.1 kN/m^2), interpolating between 6.0 feet (25 psf) and 5.5 feet (20 psf) results in 5 equal increments of 0.1 feet; (6 ft – 5.5 ft)/5 = 0.1 ft. Thus, 6.0 feet (25 psf) minus 2 x 0.1 ft (2 psf) results in *5.8 feet*.

For 30 ft building end wall width, interpolating between 5.05 ft (24 ft end wall) and 5.8 ft (32 ft end wall) results in 4 approximate increments of 0.188 feet; (5.8 ft – 5.05 ft)/4 = 0.188 ft. Thus, 5.05 feet (30 ft end wall) plus 3 x 0.188 ft (6 ft end wall length) indicates that the minimum solid wall length required is *5.6 feet*.

However, a conservative value from the 32ft end wall length and 25 psf velocity pressure resulting in a minimum solid wall length of 6.0 feet (1.8 m) may be used without interpolating.

TABLE 5.2C
MINIMUM SOLID SIDE WALL LENGTH
REQUIREMENTS FOR FLAT ICF WALLS
(WIND PARALLEL TO RIDGE)[1,2,3,4,5]
(excerpt from the *Prescriptive Method*)

Design Velocity Pressure (psf)		20	25	30	35	40	45	50	60
Wall Category	Building End Wall Width, W (feet)	Minimum Solid Wall Length on Building Side Wall, L (feet)							
One-Story or Top Story of Two-Story	16	4.00	4.00	4.00	4.00	4.25	4.25	4.50	4.75
	24	4.00	4.25	4.50	4.75	4.75	5.00	5.25	5.50
	32	4.50	4.75	5.00	5.25	5.50	6.00	6.25	6.75
	40	5.00	5.50	5.75	6.25	6.75	7.00	7.50	8.25
	50	5.75	6.25	7.00	7.50	8.25	8.75	9.50	10.75
	60	6.50	7.50	8.25	9.25	10.00	10.75	11.75	13.25
First Story of Two-Story	16	4.25	4.50	4.75	5.00	5.25	5.50	5.75	6.50
	24	**4.75**	**5.25**	5.50	6.00	6.25	6.75	7.00	8.00
	32	**5.50**	**6.00**	6.50	7.00	7.50	8.00	8.75	9.75
	40	6.25	7.00	7.50	8.25	9.00	9.75	10.50	12.00
	50	7.25	8.25	9.25	10.25	11.25	12.25	13.25	15.25
	60	8.50	9.75	11.00	12.25	13.50	15.00	16.25	18.75

The above values are based on concrete with a minimum compressive strength of 2,500 psi (17.2 MPa). Due to the combination of shear and flexural elements in the design of the minimum solid wall length no adjustment factors are available if concrete with a minimum compressive strength of 3,000 psi (20.6 MPa) or 4,000 psi (27.6 MPa) is used.

Remember that Section 5.0 requires a minimum length of 24 inches (610 mm) of solid wall segment, extending the full height of each wall story, at all corners of exterior walls and that 2-foot (610-mm) wall segments shall not exceed the maximum allowable spacing of 18 feet (5.5 m) on center.

SECOND-STORY WALLS

End Wall Design

Wind

The minimum length of solid end wall is based on the wind velocity pressure from Table 5.1, the building side wall length, and the roof slope.

Interpolation is required to obtain the accurate amount of solid end wall length required from Table 5.2A.

For the 7:12 roof pitch and 23 psf (1.1 kN/m^2), interpolating between 6.25 feet (25 psf) and 5.5 feet (20 psf) results in 5 equal increments of 0.15 feet; (6.25 ft – 5.5 ft)/5 = 0.15 ft. Thus, 6.25 feet (25 psf) minus 2 x 0.15 ft (2 psf) results in *5.95 feet*.

For the 12:12 roof pitch and 23 psf (1.1 kN/m^2), interpolating between 7.25 feet (25 psf) and 6.5 feet (20 psf) results in 5 equal increments of 6.5 feet; (7.25 ft – 6.5 ft)/5 = 0.15 ft. Thus, 7.25 feet (25 psf) minus 2 x 0.15 ft (2 psf) results in *6.95 feet*.

For 8:12 roof pitch, interpolating between 5.95 ft (7:12) and 6.95 ft (12:12) results in 5 equal increments of 0.2 feet; (6.95 ft – 5.95 ft)/5 = 0.2 ft. Thus, 5.95 feet (7:12) plus 0.2 feet indicates that the minimum solid wall length required is *6.15 feet*.

However, a conservative value from the 12:12 roof pitch and 25 psf velocity pressure resulting in a minimum solid wall length of 7.25 feet (2.2 m) may be used without interpolating.

TABLE 5.2A
MINIMUM SOLID END WALL LENGTH
REQUIREMENTS FOR FLAT ICF WALLS
(WIND PERPENDICULAR TO RIDGE)[1,2,3,4,5]
(excerpt from the *Prescriptive Method*)

Design Velocity Pressure (psf)			20	25	30	35	40	45	50	60
Wall Category	Building Side Wall Length, L (feet)	Roof Slope	Minimum Solid Wall Length on Building End Wall, W (feet)							
One-Story or Top Story of Two-Story	16	≤ 1:12	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.00
		5:12	4.00	4.00	4.00	4.00	4.00	4.00	4.25	4.50
		7:12	4.00	4.25	4.25	4.50	4.75	4.75	5.00	5.50
		12:12	4.25	4.50	4.75	5.00	5.25	5.50	5.75	6.25
	24	≤ 1:12	4.00	4.00	4.00	4.00	4.00	4.00	4.25	4.50
		5:12	4.00	4.00	4.00	4.25	4.25	4.50	4.50	4.75
		7:12	4.25	4.50	4.75	5.00	5.25	5.50	5.75	6.25
		12:12	4.75	5.00	5.25	5.75	6.00	6.50	6.75	7.50
	32	≤ 1:12	4.00	4.00	4.00	4.00	4.25	4.25	4.50	4.75
		5:12	4.00	4.00	4.25	4.50	4.50	4.75	5.00	5.25
		7:12	4.50	5.00	5.25	5.50	6.00	6.25	6.50	7.25
		12:12	5.00	5.50	6.00	6.50	7.00	7.25	7.75	8.75
	40	≤ 1:12	4.00	4.00	4.25	4.25	4.50	4.50	4.75	5.00
		5:12	4.00	4.25	4.50	4.75	4.75	5.00	5.25	5.50
		7:12	4.75	5.25	5.75	6.00	6.50	7.00	7.25	8.00
		12:12	5.50	6.00	6.50	7.25	7.75	8.25	8.75	10.00
	50	≤ 1:12	4.00	4.25	4.25	4.50	4.75	4.75	5.00	5.50
		5:12	4.25	4.50	4.75	5.00	5.25	5.50	5.75	6.00
		7:12	5.25	5.75	6.25	6.75	7.25	7.75	8.25	9.25
		12:12	6.00	6.75	7.50	8.00	8.75	9.50	10.25	11.50
	60	≤ 1:12	4.00	4.25	4.50	4.75	5.00	5.25	5.25	5.75
		5:12	4.50	4.75	5.00	5.25	5.50	5.75	6.00	6.75
		7:12	**5.50**	**6.25**	6.75	7.50	8.00	8.50	9.25	10.25
		12:12	**6.50**	**7.25**	8.25	9.00	9.75	10.50	11.50	13.00

Side Wall Design

Wind

The minimum length of solid side wall length is based on the wind velocity pressure from Table 5.1 and the building end wall length.

Interpolation is required to obtain the accurate amount of solid wall length required from Table 5.2C. However, since the values used to interpolate for the required minimum length of solid wall are almost equal, the conservative value from the 32 ft end wall length and 25 psf velocity pressure is used resulting in a minimum solid wall length of 4.75 feet (1.4 m)

TABLE 5.2C
MINIMUM SOLID SIDE WALL LENGTH
REQUIREMENTS FOR FLAT ICF WALLS
(WIND PARALLEL TO RIDGE)[1,2,3,4,5]
(excerpt from the *Prescriptive Method*)

Design Velocity Pressure (psf)		20	25	30	35	40	45	50	60
Wall Category	Building End Wall Width, W (feet)	Minimum Solid Wall Length on Building Side Wall, L (feet)							
One-Story or Top Story of Two-Story	16	4.00	4.00	4.00	4.00	4.25	4.25	4.50	4.75
	24	**4.00**	**4.25**	4.50	4.75	4.75	5.00	5.25	5.50
	32	**4.50**	**4.75**	5.00	5.25	5.50	6.00	6.25	6.75
	40	5.00	5.50	5.75	6.25	6.75	7.00	7.50	8.25
	50	5.75	6.25	7.00	7.50	8.25	8.75	9.50	10.75
	60	6.50	7.50	8.25	9.25	10.00	10.75	11.75	13.25
First Story of Two-Story	16	4.25	4.50	4.75	5.00	5.25	5.50	5.75	6.50
	24	4.75	5.25	5.50	6.00	6.25	6.75	7.00	8.00
	32	5.50	6.00	6.50	7.00	7.50	8.00	8.75	9.75
	40	6.25	7.00	7.50	8.25	9.00	9.75	10.50	12.00
	50	7.25	8.25	9.25	10.25	11.25	12.25	13.25	15.25
	60	8.50	9.75	11.00	12.25	13.50	15.00	16.25	18.75

The above values are based on concrete with a minimum compressive strength of 2,500 psi (17.2 MPa). Due to the combination of shear and flexural elements in the design of the minimum solid wall length no adjustment factors are available if concrete with a minimum compressive strength of 3,000 psi (20.6 MPa) or 4,000 psi (27.6 MPa) is used.

Remember that Section 5.0 requires a minimum length of 24 inches (610 mm) of solid wall segment, extending the full height of each wall story, at all corners of exterior walls and that 2-foot (610-mm) minimum wall segments shall not exceed the maximum allowable spacing of 18 feet (5.5 m) on center.

Wall Openings and Lintels

In this example, there are two opening sizes: 6-foot- (1.8-m-) wide openings for the entrance doorway, sliding door, and double windows in the first story and 3-foot- (0.9-m-) wide openings throughout the remainder of the building. Tables 5.6, 5.7, 5.8A, and 5.12 of the *Prescriptive Method* are used in this exercise to determine the lintel type and reinforcement requirements. For the 6-foot- (1.8-m-) and 3-foot- (0.9-m-) wide openings, Table 5.12 of the *Prescriptive Method* requires a lintel over the opening in accordance with Section 5.0.

The 6-foot (1.8 m) wide openings are located in the first-story load-bearing walls of the building. First-story walls were previously stated as 4.5 inch (114 mm) thick flat ICF walls. The building is 30 feet (9.1 m) wide and subject to a ground snow load of 50 psf (2.4 kPa). Since the lintels are located in the first-story wall, they support one ICF story and a roof and ceiling.

Table 5.7 of the *Prescriptive Method* should be consulted first to determine if lintels without stirrups/shear reinforcement are permitted. Table 5.7 illustrates that a 3.5 inch (88.9 mm) lintel thickness and 24 inch (610 mm) lintel depth supporting an ICF story above without stirrups allows for:

30 psf (1.5 kPa) ⇒ lintel span = 5’ – 6” (1.7 m)
70 psf (3.4 kPa) ⇒ lintel span = 5’ – 2” (1.6 m)

By interpolation, for the design snow load of 50 psf (2.4 kPa) the allowable lintel span without stirrups is 5’ – 4” (1.6 m). This span may be increased by multiplying by the following adjustment factors in the footnote of Table 5.7:

2,500 psi (17.2 MPa) Compressive Strength Concrete ⇒ No Adjustment Factor
3,000 psi (20.7 MPa) Compressive Strength Concrete ⇒ Adjustment Factor = 1.05
4,000 psi (27.6 MPa) Compressive Strength Concrete ⇒ Adjustment Factor = 1.10

The allowable span for the specified lintel without stirrups are as follows:

2,500 psi (17.2 MPa) Compressive Strength Concrete ⇒ 5’ – 4”(1.00) = 5’ – 4” (1.6 m)
3,000 psi (20.7 MPa) Compressive Strength Concrete ⇒ 5’ – 4”(1.05) = 5’ – 7” (1.7 m)
4,000 psi (27.6 MPa) Compressive Strength Concrete ⇒ 5’ – 4”(1.10) = 5’ – 10” (1.8 m)

Therefore, lintels with stirrups are required for the 6-foot (1.8 m) openings regardless of concrete compressive strength used.

Consult Table 5.8A of the *Prescriptive Method* to determine the allowable span for the specified lintel. Table 5.8A of the *Prescriptive Method* illustrates that a 4.5-inch (114-mm) lintel thickness and 24-inch (610-mm) lintel depth will suffice since the lintel’s maximum allowed span is 6’-1” (1.9 m) for a design snow load of 70 psf (3.4 kPa).

Determine the center portion of the span, A, where stirrups are not required by using Table 5.12 of the *Prescriptive Method*. Using interpolation, Table 5.12 of the *Prescriptive Method* illustrates that a 4.5-inch (114-mm) lintel thickness and 24-inch (610-mm) lintel depth subjected to a 50 psf (2.4 kPa) snow load is required to have stirrups for all but the center 1'–8" (0.5 m) of the span. The footnotes of Table 5.12 of the *Prescriptive Method* allows the center portion of the span (A) where stirrups are not required to be multiplied by 1.09 for 3,000 psi (20.7 MPa) and 1.26 for 4,000 psi (27.6 MPa) compressive strength concrete. The adjustment factors produce the following center portion of the span where stirrups are not required:

2,500 psi (17.2 MPa) Compressive Strength Concrete ⇒ 1' – 8"(1.00) = 1' – 8" (0.5 m)
3,000 psi (20.7 MPa) Compressive Strength Concrete ⇒ 1' – 8"(1.09) = 1' – 9" (0.5 m)
4,000 psi (27.6 MPa) Compressive Strength Concrete ⇒ 1' – 8"(1.26) = 2' – 1" (0.6 m)

When required, No. 3 stirrups shall be spaced no more than 11 inches (279 mm). In addition, Table 5.6 of the *Prescriptive Method* requires a minimum vertical reinforcement of one No. 4 bar within 12 inches (305 mm) of each side of the 6-foot- (1.8-m-) wide opening for the full height of the wall in conditions where wind speeds are 110 mph (177 km/hr) or less.

The 3-foot (0.9 m) wide openings are located throughout the remainder of the building. First-story and second-story walls were previously stated as 3.5 inch (88.9 mm) thick flat ICF walls. The building is 28 feet (8.5 m) wide and subject to a ground snow load of 50 psf (2.4 kPa).

Table 5.7 of the *Prescriptive Method* should be consulted first to determine if lintels without stirrups are permitted for the 3-foot (0.9 m) first-story lintels. Table 5.7 illustrates that a 4.5 inch (114 mm) lintel thickness and 24 inch (610 mm) lintel depth supporting one ICF story and a roof and ceiling without stirrups allows for:

30 psf (1.5 kPa) ⇒ lintel span = 5' – 6" (1.7 m)
70 psf (3.4 kPa) ⇒ lintel span = 5' – 2" (1.6 m)

By interpolation, for the design snow load of 50 psf (2.4 kPa) the allowable lintel span without stirrups is 5' – 4" (1.6 m). Since the allowable span is greater than the required 3-foot (0.9 m) span length, the applicable building width and concrete compressive strength adjustments need not be applied.

Table 5.7 of the *Prescriptive Method* should be consulted first to determine if lintels without stirrups/shear reinforcement are permitted for the 3-foot (0.9 m) second-story lintels. Table 5.7 illustrates that a 4.5 inch (114 mm) lintel thickness and 12 inch (305 mm) lintel depth supporting a light-frame roof and ceiling without stirrups allows for:

30 psf (1.5 kPa) ⇒ lintel span = 4' – 2" (1.3 m)
70 psf (3.4 kPa) ⇒ lintel span = 4' – 2" (1.3 m)

Since the listed span is greater than the required 3-foot (0.9 m) span length, the applicable building width and concrete compressive strength adjustments need not be applied. Since the opening width is only 3 feet (0.9 m), vertical reinforcement on each side of the opening is not required by Table 5.6.

TABLE 5.6
MINIMUM WALL OPENING REINFORCEMENT
REQUIREMENTS IN ICF WALLS
(excerpt from the *Prescriptive Method*)

WALL TYPE AND OPENING WIDTH, L feet (m)	MINIMUM HORIZONTAL OPENING REINFORCEMENT	MINIMUM VERTICAL OPENING REINFORCEMENT
Flat, Waffle-, and Screen-Grid: L < 2 (0.61)	None Required	None Required
Flat, Waffle-, and Screen-Grid: L ≥ 2 (0.61)	**Provide lintels in accordance with Section 5.3. Top and bottom lintel reinforcement shall extend a minimum of 24 inches (610 mm) beyond the limits of the opening.** **Provide one No. 4 bar within of 12 inches (305 mm) from the bottom of the opening. Each No. 4 bar shall extend 24 inches (610 mm) beyond the limits of the opening.**	**In locations with wind speeds less than or equal to 110 mph (177 km/hr) or in Seismic Design Categories A and B, provide one No. 4 bar for the full height of the wall story within 12 inches (305 mm) of each side of the opening.** **In locations with wind speeds greater than 110 mph (177 km/hr) or in Seismic Design Categories C, D_1, and D_2 provide two No. 4 bars or one No. 5 bar for the full height of the wall story within 12 inches (305 mm) of each side of the opening.**

TABLE 5.7
MAXIMUM ALLOWABLE CLEAR SPANS FOR ICF LINTELS WITHOUT STIRRUPS IN LOAD-BEARING WALLS[1,2,3,4,5,6,7]
NO. 4 OR NO. 5 BOTTOM BAR SIZE
(excerpt from the *Prescriptive Method*)

Minimum Lintel Thickness, T (inches)	Minimum Lintel Depth, D (inches)	Maximum Clear Span (feet – inches)					
		Supporting Light-Frame Roof Only		Supporting Light-Frame Second Story and Roof		Supporting ICF Second Story and Light-Frame Roof [8]	
		Maximum Ground Snow Load (psf)					
		30	70	30	70	30	70
Flat ICF Lintel							
3.5	8	2-6	2-6	2-6	2-4	2-5	2-2
	12	**4-2**	**4-2**	4-1	3-10	3-10	3-7
	16	4-11	4-8	4-6	4-2	4-2	3-10
	20	6-3	5-3	4-11	4-6	4-6	4-3
	24	7-7	6-4	6-0	5-6	**5-6**	**5-2**
5.5	8	2-10	2-6	2-6	2-6	2-6	2-6
	12	4-8	3-8	3-4	3-0	3-0	2-9
	16	6-5	5-1	4-8	4-2	4-3	3-10

TABLE 5.8A
MAXIMUM ALLOWABLE CLEAR SPANS FOR FLAT ICF LINTELS WITH STIRRUPS IN LOAD-BEARING WALLS[1,2,3,4,5,6,7]
(NO. 4 BOTTOM BAR SIZE)
(excerpt from the *Prescriptive Method*)

Minimum Lintel Thickness, T (inches)	Minimum Lintel Depth, D (inches)	Maximum Clear Span (feet – inches)					
		Supporting Light-Frame Roof Only		Supporting Light-Frame Second Story and Roof		Supporting ICF Second Story and Light-Frame Roof [8]	
		Maximum Ground Snow Load (psf)					
		30	70	30	70	30	70
3.5	8	4-9	4-2	3-10	3-4	3-5	3-1
	12	6-8	5-5	5-0	4-5	4-6	4-0
	16	7-11	6-5	6-0	5-3	5-4	4-10
	20	8-11	7-4	6-9	6-0	6-1	5-6
	24	9-10	8-1	7-6	6-7	**6-9**	**6-1**
5.5	8	5-2	4-2	3-10	3-5	3-5	3-1
	12	6-8	5-5	5-0	4-5	4-6	4-1
	16	7-10	6-5	6-0	5-3	5-4	4-10

TABLE 5.12
MIDDLE PORTION OF SPAN, *A*, WHERE STIRRUPS ARE NOT REQUIRED FOR FLAT ICF LINTELS[1,2,3,4,5,6,7]
NO. 4 or NO. 5 BOTTOM BAR SIZE
(excerpt from the *Prescriptive Method*)

Minimum Lintel Thickness, T (inches)	Minimum Lintel Depth, D (inches)	Maximum Center Distance, A (feet – inches)					
		Supporting Light-Frame Roof Only		Supporting Light-Frame Second Story and Roof		Supporting ICF Second Story and Light-Frame Roof [7]	
		Maximum Ground Snow Load (psf)					
		30	70	30	70	30	70
3.5	8	1-2	0-9	0-8	0-6	0-6	0-5
	12	1-11	1-3	1-1	0-10	0-10	0-8
	16	2-7	1-9	1-6	1-2	1-2	1-0
	20	3-3	2-3	1-11	1-6	1-6	1-3
	24	3-11	2-8	2-4	1-10	1-10	1-6
5.5	8	1-10	1-2	1-0	0-9	0-10	0-8
	12	3-0	2-0	1-8	1-4	1-4	1-1
	16	4-1	2-9	2-4	1-10	1-11	1-6

ICF Foundation Wall-to-Footing Connection

Section 6.0 of the *Prescriptive Method* does not require dowels to be installed across the ICF wall-footing interface because the interior floor slab will be poured before backfilling and installed in accordance with Figure 3.3 of the *Prescriptive Method*.

ICF Basement Wall-to-Floor Connection

The design house is 28 feet (8.5 m) wide and has wood floor joists spaced at 24 inches (610 mm) on center. A wood ledger is used to support the floor joists; refer to Section 6.0. A 2-inch (51-mm) nominal wood ledger and 1/2-inch (13-mm) anchor bolts will be used. Assuming that a load-bearing beam is placed beneath the floor at mid-span, the resulting floor clear span is approximately 14 feet (4.3 m). Table 6.1 of the *Prescriptive Method* requires a minimum staggered 1/2-inch- (13-mm-) diameter anchor bolt spaced at 12 inches (305 mm) on center. Alternatively, two 1/2-inch- (13-mm-) diameter anchor bolts spaced at 24 inches (610 mm) on center, staggered 5/8-inch- (15.8-mm-) diameter anchor bolt spaced at 16 inches (406 mm) on center, or two 5/8-inch- (15.8-mm-) diameter anchor bolts spaced at 32 inches (813 mm) on center may also be used.

TABLE 6.1
FLOOR LEDGER-ICF WALL CONNECTION (SIDE-BEARING CONNECTION) REQUIREMENTS[1,2,3]
(excerpt from the *Prescriptive Method*)

Maximum Floor Clear Span[4] (feet)	Maximum Anchor Bolt Spacing[5] (inches)			
	Staggered 1/2-Inch-Diameter Anchor Bolts	Staggered 5/8-Inch-Diameter Anchor Bolts	Two 1/2-Inch-Diameter Anchor Bolts[6]	Two 5/8-Inch-Diameter Anchor Bolts[6]
8	18	20	36	40
10	16	18	32	36
12	14	18	28	36
14	12	16	24	**32**

ICF Wall-To-Roof Connection

Section 6.0, Table 6.3, and Figure 6.8 of the *Prescriptive Method* are used to determine the top of the ICF wall-to-roof connection requirements. It is assumed that the design house has wood trusses in lieu of wood rafters and is located in a 110 mph (177 km/hr) wind area. Table 6.3 of the *Prescriptive Method* requires a minimum of a 1/2-inch (13-mm) anchor bolt spaced at 6 feet (1.8 m) on center. Section 6.3 requires an approved uplift strap to attach the roof to the wood sill plates in accordance with the applicable building code.

TABLE 6.3
TOP SILL PLATE-ICF WALL CONNECTION REQUIREMENTS
(excerpt from the *Prescriptive Method*)

MAXIMUM WIND SPEED (mph)	MAXIMUM ANCHOR BOLT SPACING 1/2-INCH-DIAMETER ANCHOR BOLT
90	6’-0”
100	6’-0”
110	**6’-0”**

APPENDIX B

ENGINEERING TECHNICAL SUBSTANTIATION

INTRODUCTION

The *Engineering Technical Substantiation* is provided as a supplemental information package to document the basis for the development of the *Prescriptive Method*. Structural calculations illustrate the method for determining the reinforcement requirements for the walls and lintels; supplemental equations and reference standards substantiate the examples. The example calculations are not intended to be inclusive of all design considerations for a given application but rather are intended to illustrate the derivation of the requirements in the *Prescriptive Method*.

Information is presented in both U.S. customary units and International System (SI) units except for reinforcement bar sizes which are only presented in U.S. customary units. Refer to Appendix C for the corresponding reinforcement bar size in SI units.

B1.0 GENERAL

B1.1 Load Calculations

Roof Loads

Roof snow loads were calculated using ASCE 7 [B1], multiplying the ground snow load by 0.7. Therefore, the roof snow load was taken as $P = 0.7P_g$, where P_g is the ground snow load in pounds per square foot. Off-balance snow loads were not considered since heavier gravity loads improve the performance of concrete walls used in residential construction. Also, this approach has proven successful based on past performance and experience in residential construction when building and roof spans are relatively small.

Wind Loads

Wind loads were based on 3-second gust wind speeds ranging from 90 to 150 mph (145 to 241 km/hr). Wind pressures were calculated in accordance with ASCE 7 [B1] by using components and cladding coefficients for partially-enclosed and enclosed buildings, interior zone, and a mean roof height of 35 feet (10.7 m). Component and cladding wind loads are used to determine out-of-plane bending and out-of-plane shear in the walls. Main wind force resisting system (MWFRS) loads were also determined in accordance with ASCE 7 [B1]. The distribution of lateral wind loads to components or assemblies comprising the MWFRS was achieved by a standard tributary area method. Wind forces on all windward and leeward surfaces were considered when determining MWFRS loads. Example wind calculations are presented in Section 4 and 5 of the *Technical Substantiation*.

Seismic Loads

Seismic loads were determined in accordance with the International Building Code [B7].

B1.2 ICF Foundation Wall Design Approach

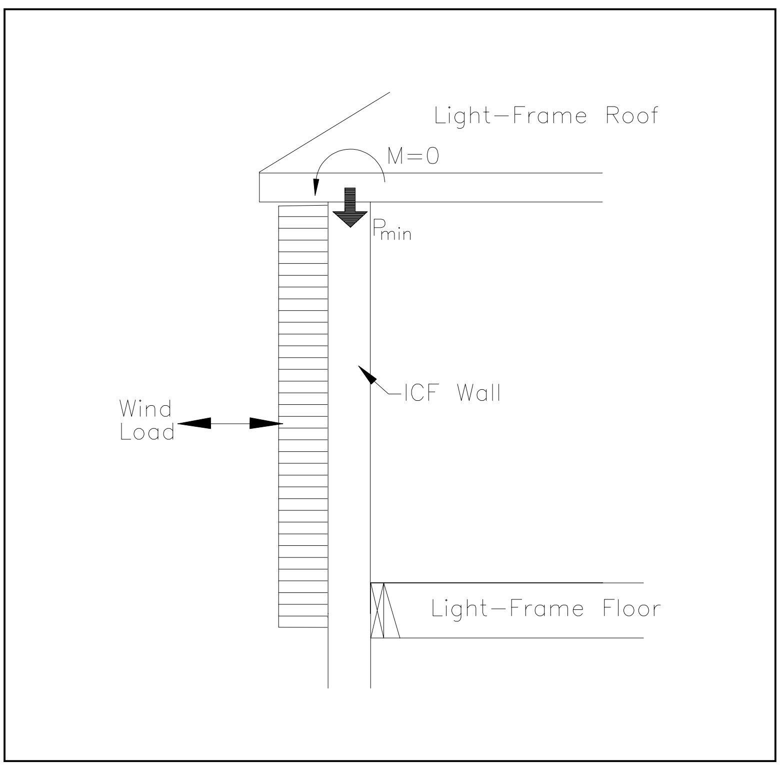
Four different construction cases were investigated to cover a range of residential construction possibilities for the development of the first edition of the *Prescriptive Method*. As expected, the controlling load case was that which induced the lowest practical axial load on the foundation wall in combination with maximum design bending or lateral load. Therefore, a single construction case was investigated in the development of the second edition.

The load case considered in the development of the second edition of the *Prescriptive Method* is conservative in that the no dead, live or gravity loads are considered which would increase the moment capacity, even with considerable eccentricity of axial load toward the outside face of the foundation wall. This method is consistent with the development of the plain concrete and reinforced concrete foundation wall provisions in the *International Residential Code* [B2].

The foundation wall height, unbalanced backfill height, and earth load vary. The vertical wall reinforcements listed in the minimum vertical wall reinforcement tables for basement walls of the *Prescriptive Method* are the results from calculations based on the above construction case and ACI [B3] Load Combinations.

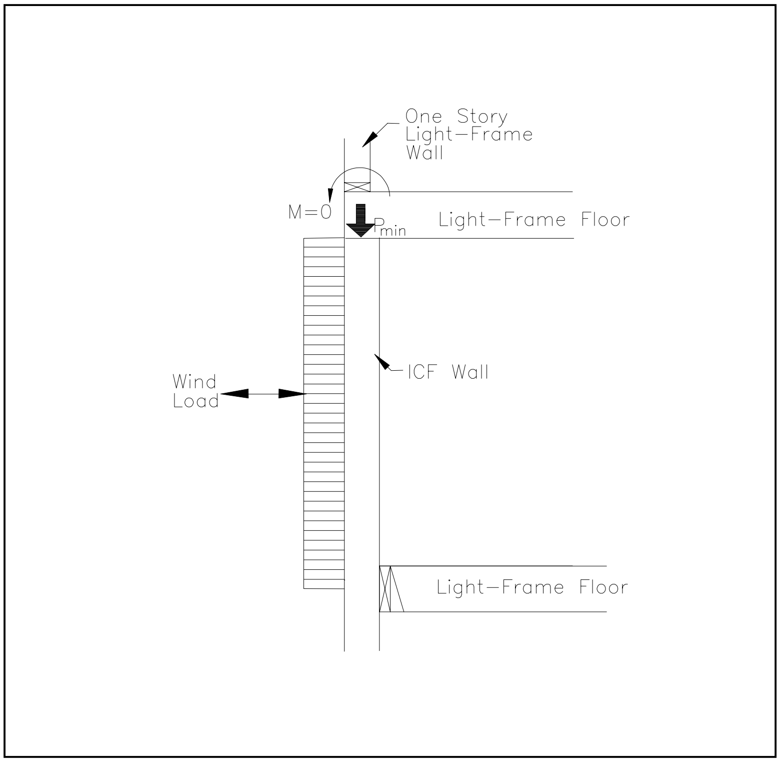
B1.3 ICF Above-Grade Wall Design Approach

For above-grade wall construction, three different construction cases were investigated to cover the range of construction possibilities. The three construction cases are described below.



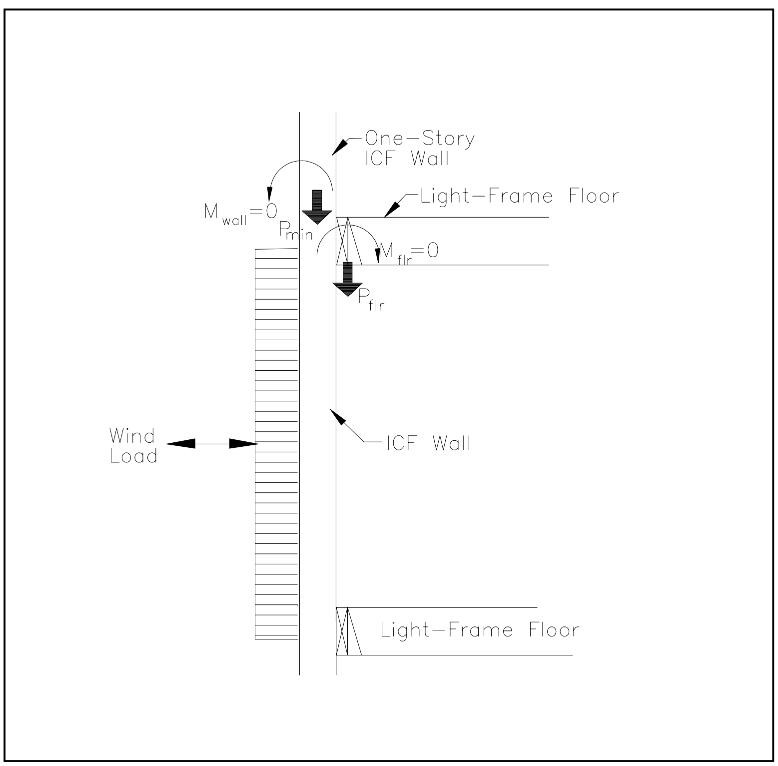
Supporting Light-Frame Roof Only

- Smallest axial design load with light-frame construction due to dead loads
- Assume gable end wall
- Wind loads vary from 20 psf to 80 psf (0.96 kPa to 3.83 kPa)
- No eccentricity, assume roof trusses bearing directly on the wall.



Supporting One-Story Light-Frame Construction and Roof

- Smallest axial design load with light-frame construction due to dead loads
- No eccentricity; assume direct floor bearing on first-story wall
- Wind loads vary from 20 psf to 80 psf (0.96 kPa to 3.83 kPa)



Supporting One-Story ICF Construction and Light-Frame Roof

- Smallest axial design load with ICF construction due to dead loads
- No wall eccentricity; assume ICF wall thickness for second-story wall is equal to ICF wall thickness for first-story wall
- Wind loads vary from 20 psf to 80 psf (0.96 kPa to 3.83 kPa)
- No floor eccentricity

The following values were used to design the above-grade walls; refer to Figure B1.1. The story wall height and wind load vary. The vertical wall reinforcements listed in the minimum vertical wall reinforcement tables in the *Prescriptive Method* are the results of calculations based on the applicable construction case (B1 through B3) and ACI [B3] Load Combinations.

BUILDING GEOMETRY	Units	ABOVE-GRADE CONSTRUCTION CASE		
		B1	B2	B3
Maximum Building Plan Dimension	feet (m)	60 (18)	60 (18)	60 (18)
Roof Slope	----	0:12	12:12	12:12
Roof Tributary Area	feet (m)	Gable End	2 (0.6)	2 (0.6)
First-Floor Tributary Area	feet (m)	N/A	N/A	N/A
Second-Floor Tributary Area	feet (m)	N/A	2 (0.6)	2 (0.6)
First-Story Wall Height	feet (m)	Vary	Vary	Vary
Second-Story Wall Height	feet (m)	N/A	8 (2.4)	8 (2.4)
First-Story Wall Thickness	inches (mm)	Vary	Vary	Vary
Second-Story Wall Thickness	inches (mm)	N/A	3.5 (88.9) light-frame	3.5 (88.9) ICF
Foundation Wall Thickness	psf (kPa)	N/A	N/A	N/A
DEAD LOADS				
First Floor	psf (kPa)	N/A	N/A	N/A
Second Floor	psf (kPa)	N/A	10 (0.48)	10 (0.48)
Roof and Ceiling	psf (kPa)	0	10 (0.48)	10 (0.48)
First-Story Wall	psf (kPa)	Vary	Vary	Vary
Second-Story Wall	psf (kPa)	N/A	8 (0.39)	44 (2.1)
Foundation Wall	psf (kPa)	N/A	N/A	N/A
LIVE LOADS				
First Floor	psf (kPa)	N/A	N/A	N/A
Second Floor	psf (kPa)	N/A	0	0
Roof (ground snow)	psf (kPa)	0	0	0
Attic	psf (kPa)	0	0	0
SEISMIC DESIGN CATEGORY	**----**	A - D_2	A - D_2	A - D_2

Figure B1.1 Above-Grade Wall Loading Conditions and Building Geometry

B1.4 ICF Lintel Design Criteria

The moment and shear capacities were determined in accordance with ACI [B3]. The following design assumptions were used:

Second-Floor Live Load	=	30 *psf*	[1.4 *kPa*]
Attic Live Load	=	20 *psf*	[0.96 *kPa*]
Roof Dead Load	=	15 *psf*	[0.72 *kPa*]
Roof and Floor Clear Span	=	32 *ft*	[9.8 *m*]
ICF Wall Dead Load	=	69 *psf*	[3.3 *kPa*]
Deflection Criterion	=	*L*/240	

No. 4 and No. 5 bars are used for tensile reinforcement. All steel has a minimum tensile strength of 40,000 psi (300 MPa). All concrete has a minimum compressive strength, f'_c, of 2,500 psi (17.2 MPa).

B1.5 Ledger Board Connection Design Criteria

The following design assumptions were used:

Floor Live Load	=	40 *psf*	[1.9 *kPa*]
Floor Dead Load	=	15 *psf*	[0.72 *kPa*]
Wind Load	=	80 *psf*	[3.8 *kPa*]
Wall Height above and below Connection	=	10 *feet*	[3 *m*]
Floor Joist Spacing	=	2 *feet*	[0.61 *m*]

B2.0 PROPERTIES

B2.1 Material Properties

B2.1.1 Steel Reinforcement and Concrete

Assume the following minimums.

F_y = 40,000 *psi for steel reinforcement* [300 *MPa*]
f_c' = 2,500 *psi for concrete* [17 *MPa*]

Assume the following maximums.

F_y = 60,000 *psi for steel reinforcement* [414 *MPa*]
f_c' = 4,000 *psi for concrete* [28 *MPa*]

B2.1.2 Ledger Board

Assume a 1.5-inch x 7.25-inch (38-mm x 184-mm), No. 2 Grade Hem-Fir with the following allowable design properties determined in accordance with the *National Design Specification for Wood Construction (NDS)* [B4].

F_b = 850 *psi* [5.9 *MPa*]
F_b' = 850 *psi* (1.15) (1.6) (1.0)
= 1,564 *psi modified for flat use, wind load duration, and size* [10.8 *MPa*]
F_b' = 850 *psi* (1.0)(1.0)
= 850 *psi modified for live load duration and size* [5.9 *MPa*]
$F_{c\perp}$ = 405 *psi* [2.8 *MPa*]
$F_{c\perp}'$ = 405 *psi* (1.25) = 506 *psi modified for small bearing area* [3.5 *MPa*]
F_v = 75 *psi* [0.52 *MPa*]
F_v' = (75 *psi*) (2) = 150 *psi modified for no splits* [1.03 *MPa*]
S_{xx} = 13.14 in^3 [215 dm^3]
S_{yy} = 2.72 in^3 [44.6 dm^3]
h = 7.25 *inch* [184 *mm*]
b = 1.5 *inch* [38 *mm*]

B2.1.3 Ledger Board Bolts

Assume A36 steel with the following design properties determined in accordance with the *Manual of Steel Construction Allowable Stress Design* [B5].

F_t = 19,100 *psi* [132 *MPa*]
F_y = 36,000 *psi* [248 *MPa*]
F_v = 9,860 *psi threads included in shear plane* [68 *MPa*]

B2.2 Section Properties of Concrete

Due to the variety of available dimensions, a minimum equivalent rectangular section with the dimensions listed in Figure B2.1 is used to generate minimum reinforcement tables in the *Prescriptive Method* for use with as many ICF products as reasonably possible without adversely impacting design economy or practicality.

ICF Wall Type	Nominal Thickness inches (mm)	Minimum Equivalent Thickness inches (mm)	Minimum Equivalent Width inches (mm)	Vertical Core Spacing inches (mm)
Flat	3.5 (89)	3.5 (89)	12 (305)	N/A
	5.5 (140)	5.5 (140)	12 (305)	N/A
	7.5 (191)	7.5 (191)	12 (305)	N/A
	9.5 (241)	9.5 (241)	12 (305)	N/A
Waffle-Grid	6 (152)	5.0 (127)	6.25 (159)	12 (305)
	8 (203)	7.0 (178)	7.0 (178)	12 (305)
Screen-Grid	6 (152)	5.5 (140)	5.5 (140)	12 (305)

Figure B2.1 Equivalent Rectangular Section Dimensions

B3.0 ICF FOUNDATION WALL DESIGN EXAMPLES AND ENGINEERING CALCULATIONS

The following engineering calculations are provided as supplemental information to illustrate the means and methods of analysis followed in developing the requirements included in the *Prescriptive Method*. Structural calculations specifically illustrate the method for calculating the reinforcement requirements and adjustment factors used within the tables. The example calculations are not intended to be inclusive of all design considerations for a given application, but rather are intended to illustrate the derivation of the tables in the *Prescriptive Method*.

B3.1 5.5-Inch- (140-mm-) Thick Flat ICF Basement Wall

For the purposes of illustration, a flat ICF foundation wall is selected from Table 3.4 of the *Prescriptive Method* for foundation walls constructed in soil with an equivalent fluid density of 30 pcf (481 kg/m^3). The foundation wall is 9 feet (2.7 m) high and has 5 feet (1.5 m) of unbalanced backfill. Table 3.4 shows that the foundation wall is required to have a minimum of one No. 4 bar at 48 inches (1.2 m) on center for vertical wall reinforcement. Calculate the capacity and check the adequacy of the 5.5-inch- (140-mm-) thick flat ICF foundation wall.

Using the material properties in Section 2.0 compute the amount of vertical wall reinforcement required. The load case considered is conservative in that no dead, live or gravity loads are considered which would increase the capacity. The controlling condition is when the axial load is minimized and the moment is maximized, therefore, only the dead load for the foundation wall itself is considered. The following example summarizes the nominal and factored loads calculated.

<u>*Dead Loads on Foundation Wall*</u>

First Floor	0 *plf*	[0 *N/m*]
Roof and Ceiling	0 *plf*	[0 *N/m*]
First-Story Wood Wall	0 *plf*	[0 *N/m*]
Foundation ICF Wall	$0.5(9.0\,ft)(5.5in/12in/ft)(150\,pcf) = 309\,plf$ *@ mid-height*	[4.5 *N/m*]

<u>*Live Loads on Foundation Wall*</u>

First Floor	0 *plf*	[0 *N/m*]
Roof and Ceiling	0 *plf*	[0 *N/m*]
First-Story Wood Wall	0 *plf*	[0 *N/m*]

<u>*Foundation Wall Moments*</u>

Dead Load$_{@top}$	0 *in-lb/lf*	[0 *N-m/m*]
Live Load$_{@top}$	0 *in-lb/lf*	[0 *N-m/m*]

$$Moment_{@midht} \quad -\frac{(30\,pcf)(5\,ft)(4.5\,ft)^2}{2} + \frac{(30\,pcf)(4.5\,ft)^3}{6} + 306\,plf(4.5\,ft) = 312\,ft-lb/lf \qquad [1.39\ kN\text{-}m/m]$$

$$Moment_{@x} \quad -\frac{(30\,pcf)(5\,ft)(2.85\,ft)^2}{2} + \frac{(30\,pcf)(2.85\,ft)^3}{6} + 306\,plf(2.85\,ft) = 377\,ft-lb/lf \qquad [1.68\ kN\text{-}m/m]$$

x = 2.85 feet (0.87 m) location of the maximum moment; a simply supported beam model is used.

Parallel Shear

The unbalanced backfill height is greater than 4 feet (1.2 m). Assume all basement walls in the building have 5 feet (1.5 m) of unbalanced backfill height. Parallel shear is neglected as common practice since the foundation walls are generally restrained by the soil lateral pressure on all sides and opening areas in foundation walls are generally low.

Perpendicular Shear

Refer to Section 7.0 for variable definitions.

$$V_{top} = \frac{ql^3}{6L} = \frac{(30\,pcf)(5\,ft)^3}{6(9\,ft)} = 69\,plf \qquad [1.0\ kN/m]$$

$$V_{bottom} = \frac{ql^2}{2} - V_{top} = \frac{(30\,pcf)(5\,ft)^2}{2} - 69\,plf = 306\,plf \qquad [4.2\ kN/m]$$

$$V_{midht} = qlx - \frac{qx^2}{2} - V_{bottom} = (30\,pcf)(5\,ft)(4.5\,ft) - \frac{(30\,pcf)(4.5\,ft)^2}{2} - 306\,plf = 66\,plf \qquad [0.96\ \text{kN/m}]$$

Load Combinations

The following load combinations are from ACI 318 Chapter 9 [B3]:

(1) U = 1.4D + 1.7 L
(2) U = 1.4D + 1.7 L + 1.7 H
(3) U = 0.9D + 1.3 W
(4) U = 0.9D + 1.7 H

All foundation design cases are controlled by load combination number (4) since it produces the smallest possible axial load and maximum bending load.

Check Perpendicular Shear

The critical factored perpendicular shear load, V_u, experienced by the foundation wall occurs at the bottom of the wall story due to ACI 318 Load Combination (4).

$$V_u = 1.7(306\,plf) = 520\,plf \qquad [7.1\ kN/m]$$

$$\phi V_n = 0.65\left(\frac{4}{3}\right)\sqrt{f'_c}\,b_w h = 0.65\left(\frac{4}{3}\right)\sqrt{2{,}500\,psi}\,(12\,in)(5.5\,in) = 2{,}860\,plf \qquad [41.7\ kN/m]$$

$$V_u \le \phi V_n \quad OK$$

Check Compression and Tension

$M_u = 1.7(377\ \textit{ft-lb/lf})(12\ \textit{in/ft}) = 7{,}690\ \textit{in-lb/ft}$ [2.9 *kN-m/m*]

$P_u = 0.9(423\ \textit{plf}) = 381\textit{plf}$ [5.3 *kN/m*]

Determine Points for the Interaction Diagrams

Interaction Diagram for Plain Structural Concrete

Point 1 – Pure Compression

$$P_o = 0.60 f'_c\left[1-\left(\frac{h}{32t}\right)^2\right]A_c = 0.6(2{,}500\,psi)\left[1-\left(\frac{9\,ft(12in/ft)}{32(5.5in)}\right)^2\right](5.5in)(12in) = 61{,}722lbf$$ [275 *kN*]

$$\phi\,P_n = 0.65(61{,}722) = 40{,}119lbf$$ [178 *kN*]

$$\phi\,M_n = 0$$

Point 2 – Pure Bending

$$\phi\,M_n = \phi\,5\sqrt{f'_c}S = 0.65(5)\sqrt{2{,}500\,psi}\left(\frac{(12in)(5.5in)^2}{6}\right) = 9{,}831lb-in$$ [1.1 *kN-m*]

$$\phi\,P_u = 0$$

Point 3 – Balanced Condition

The balanced condition represents the dividing point between compression controls and tension controls regions of the strength interaction diagram. According to ACI 22.5.3 [B3] plain structural concrete members subject to combined axial load and bending shall be proportioned such that on the compression face:

$$P_u/\phi\,P_n + M_u/\phi\,(0.85)(f'_c)S \le 1$$

and on the tension face:

$$M_u/S - P_u/A_g \le 5\phi\,\sqrt{f'_c}$$

Hence,

$$M_u = \frac{\phi\, P_n + 5\phi\,\sqrt{f_c'}\,A_c}{\left(\dfrac{A_c}{S} + \dfrac{\phi\, P_n}{\phi\,(0.85)(f_c')(S)}\right)}$$

$$= \frac{40{,}119lb + 5(0.65)\sqrt{2{,}500psi}\,(5.5in)(12in)}{\left(\dfrac{(5.5in)(12in)}{(12in)(5.5in)^2/6} + \dfrac{40{,}119lb}{(0.65)(0.85)(2{,}500psi)\dfrac{(12in)(5.5in)^2}{6}}\right)} = 32{,}364lb - in \qquad [3.64\ \text{kN-}m]$$

$$P_u = \frac{M_u}{S}A_c - 5\phi\,\sqrt{f_c'}\,A_c = \frac{32{,}364lb - in(5.5in)(12in)}{(12in)(5.5in)^2/6} - 5(0.65)\sqrt{2{,}500psi}\,(5.5in)(12in) = 24{,}581lbf \qquad [109\ kN]$$

The hand calculations above were consolidated into spreadsheet form and factored according to ACI 318 [B3]. The factored moments were also magnified according to ACI 318 Section 10.12 [B3] for structurally reinforced concrete walls (see Section B4 of the *Technical Substantiation* for an example of magnified moments). With the factored axial loads and maximum factored moments, multiple reinforcement schedules were plotted on a factored P_u-M_u interaction diagram.

Figure B3.1 shows the factored loads for the 5.5-inch (140 mm) thick plain ICF wall in this example (see plotted point for “30 pcf”). It also contains the plain structural concrete curve. The requirements of the foundation wall system are moment controlled and thereby lie on the lowest portion of the curve. For determining the requirements, a “zoom-in” on the moment controlled region of the diagram is shown in Figure B3.2.

Figure B3.2 indicates that the required strength for the 5.5-inch- (140-mm-) thick flat basement wall that is 9 feet (2.7 m) high and has 5 feet (1.5 m) of unbalanced backfill in soil with an equivalent fluid density of 30 pcf (481 kg/m^3) is satisfied by specifying the wall to meet plain structural concrete requirements.

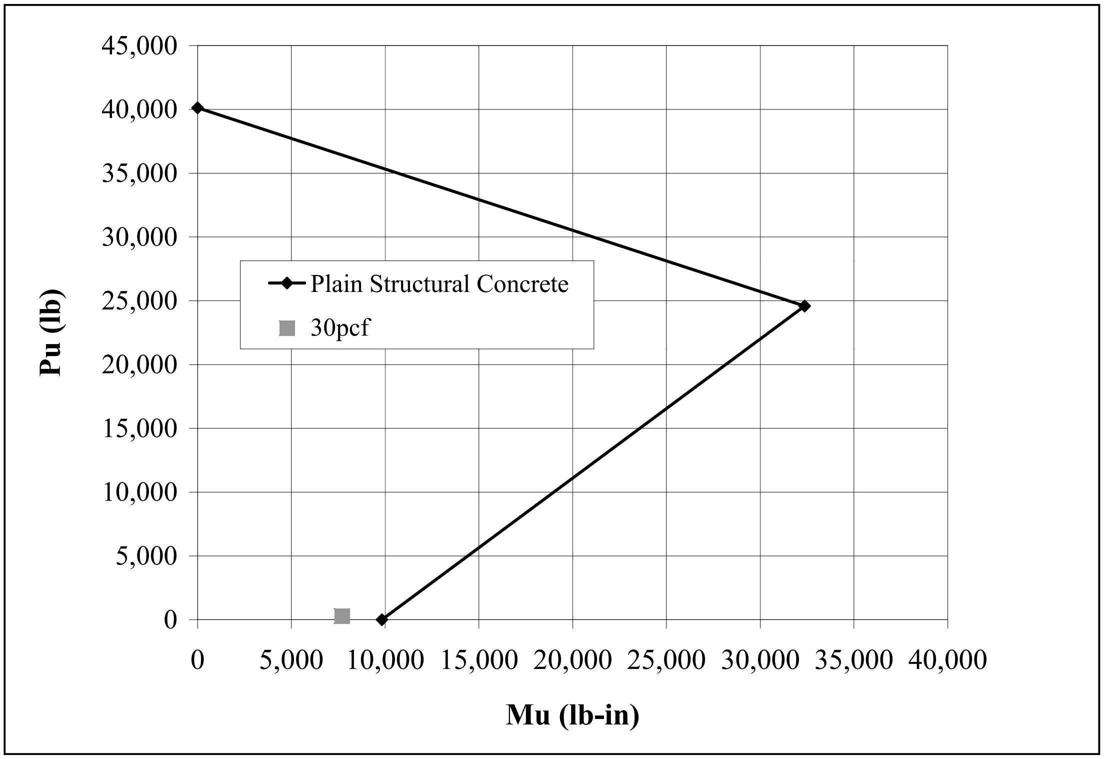


Figure B3.1 P_u-M_u Diagram for the 5.5 in Basement Wall

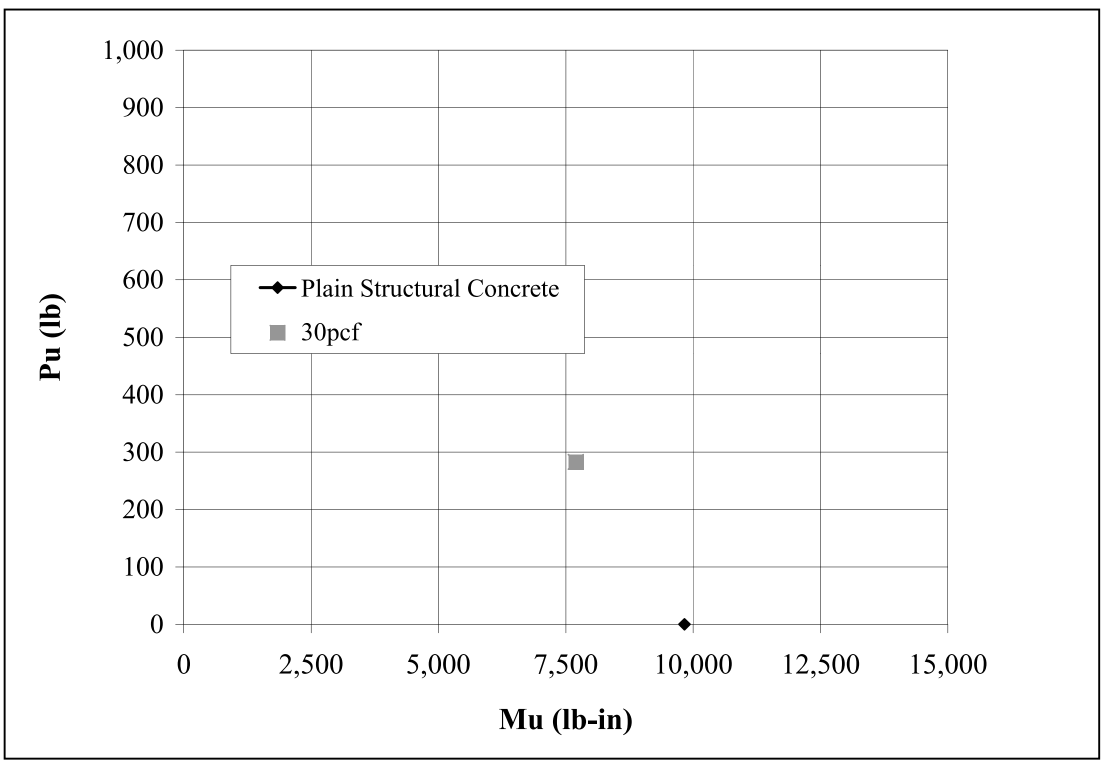


Figure B3.2 Zoom of P_u-M_u Diagram for the 5.5 in Basement Walls

Check Deflection

For below-grade walls, a deflection limit of *L/240* for service live loads is used. To calculate wall deflection, effective section properties, $E_c I_g$, of the uncracked concrete section are used. For simplicity, when calculating the maximum deflection, assume the earth load acts on the entire wall height.

$$\Delta_{maximum} = \frac{0.01304(0.5)ql^5}{E_c I_g} = \frac{0.01304(0.5)(30\,pcf)(1\,ft)(9\,ft)^5}{(2{,}850{,}000\,psi)\left(\frac{(12in)(5.5\,in)^3}{12}\right)}\left(\frac{1{,}728in^3}{ft^3}\right) = 0.042in \qquad [1.07\ mm]$$

$$\Delta_{allowable} = \frac{L}{240} = \frac{(9\ ft)\left(\frac{12\,in}{ft}\right)}{240} = 0.45\,in \qquad [11\ mm]$$

$$\Delta_{actual} \le \Delta_{allowable} \qquad OK$$

Determine Reinforcement

Although the wall is acceptable as a structural plain concrete wall in this case, a nominal amount of reinforcement is specified for the 5.5 inch (140 mm) thick below grade walls. Tests [B6] have shown that horizontal and vertical wall reinforcement spacing limited to 48 inches (1.2 m) on center results in reliable performance for ICFs; therefore, one Grade 40 (276 MPa) or Grade 60 (414 MPa), No. 4 bar at 48 inches (1.2 m) on center is specified for minimum vertical wall reinforcement. One No. 4 bar at 48 inches (1.2 m) on center is similarly specified as a minimum horizontal wall reinforcement. At least one continuous horizontal reinforcing bar should be placed within the top 12 inches (305 mm) of the wall story.

This procedure was repeated based on all possible construction permutations included in Chapter 3 of the *Prescriptive Method* including 60 ksi (414 MPa) reinforcing steel, 3,000 psi (20.7 MPa) concrete, and 4,000 psi (27.6 MPa) concrete. However, when reinforced concrete was required by first analyzing plain concrete P_u–M_u curves, a similar set of P_u–M_u curves was created to determine minimum amounts of reinforcement required. These analyses yielded the allowable 50-percent spacing increase when 60 ksi (414 MPa) reinforcing steel is used. However, there was no appreciable benefit in using higher strength concrete for this application.

TABLE 3.4
MINIMUM VERTICAL WALL REINFORCEMENT FOR
5.5-INCH- (140-MM-) THICK FLAT ICF BASEMENT WALLS [1,2,3,4,5]
(excerpt from the *Prescriptive Method*)

Max. Height of Basement Wall (feet)	Maximum Unbalanced Backfill Height[6] (feet)	Minimum Vertical Reinforcement		
		Maximum Equivalent Fluid Density 30 pcf	Maximum Equivalent Fluid Density 45 pcf	Maximum Equivalent Fluid Density 60 pcf
8	4	#4@48”	#4@48”	#4@48”
	5	#4@48”	#3@12”; #4@22”; #5@32”; #6@40”	#3@8”; #4@14”; #5@20”; #6@26”
	6	#3@12”; #4@22”; #5@30”; #6@40”	#3@8”; #4@14”; #5@20”; #6@24”	#3@6”; #4@10”; #5@14”; #6@20”
	7	#3@8”; #4@14”; #5@22”; #6@26”	#3@5”; #4@10”; #5@14”; #6@18”	#3@4”; #4@6”; #5@10”; #6@14”
9	4	#4@48”	#3@32”; #4@48”	#4@48”
	5	**#4@48”**	#3@12”; #4@20”; #5@28”; #6@36”	#3@8”; #4@14”; #5@20”; #6@22”
	6	#3@10”; #4@20”; #5@28”; #6@34”	#3@6”; #4@12”; #5@18”; #6@20”	#4@8”; #5@14”; #6@16”
	7	#3@8”; #4@14”; #5@20”; #6@22”	#4@8”; #5@12”; #6@16”	#4@6”; #5@10”; #6@12”
	8	#3@6”; #4@10”; #5@14”; #6@16”	#4@6”; #5@10”; #6@12”	#4@4”; #5@6”; #6@8”

B4.0 ICF ABOVE-GRADE WALL DESIGN EXAMPLES AND ENGINEERING CALCULATIONS

The following engineering calculations are provided as supplemental information to illustrate the means and methods used in the development of the requirements included in the *Prescriptive Method*. Example calculations are used to illustrate the method of determining reinforcement requirements and adjustment factors used within the tables. The example calculations are not intended to be inclusive of all design considerations, but rather are intended to clearly illustrate the derivation of values in the *Prescriptive Method*.

B4.1 Calculating Wind Pressures

An enclosed building sited where wind speeds are 100 mph (161 km/hr) in Exposure Category B is considered. Table 4.1 shows that the above-grade wall design wind pressure is 24 psf (1.15 kPa). This wind pressure is used to select the appropriate vertical reinforcement requirements for out-of-plane bending for the various ICF systems at the stated conditions for this example. The following calculations show how the design wind pressures in Table 4.1 are calculated for the example building conditions using ASCE 7 [B1]. A mean roof height of 35 ft (10.7 m) was used for all table values.

B4.1.1 Determine the Velocity Pressure

$$q_h = 0.00256 K_z K_{zt} K_d V^2 I = (0.00256)(0.73)(1.0)(1.0)(100 mph)^2 (1.00) = 18.69 psf$$

Note: A K_d factor of 1.0 is used for concrete design in accordance with the International Building Code, Section 1605.2.1 [7].

B4.1.2 Determine the Net External and Internal Pressure Coefficient

$$\sum\left(GC_p - GC_{pi}\right) = (-1.1 - 0.18) = -1.28$$

Note: GC_p is based on Zone 4 values. Negative pressure creates the worst-case load magnitude. A GC_{pi} value of –0.55 is used to determine pressures for partially enclosed buildings in lieu of –0.18 above.

B4.1.3 Determine the Design Wind Pressure

$$p = q_h \sum\left(GC_p - GC_{pi}\right) = (18.69\ psf)(-1.28) = -23.9\ psf$$

B4.2 Out-of-Plane Seismic Loads

According to the *International Building Code* [B7], bearing walls and shear walls shall be designed for an out-of-plane force that is the greater of 10 percent of the wall weight or the quantity given by

$$F_p = 0.40 I_E S_{DS} W_w$$

where:

I_E = Occupancy importance factor (1.00 for residential construction)
S_{DS} = The short period site design spectral response acceleration coefficient
W_w = The weight of the wall

Figure B4.1 summarizes the out-of-plane seismic loads for Seismic Design Categories C, D_1, and D_2. Since the out-of-plane seismic loads for Seismic Design Category C are less than the lowest design wind pressure (20 psf) in Tables 4.2 through Table 4.4 of the *Prescriptive Method*, wind governs the design in all cases. By inspecting Tables 4.2 to 4.5 of the Prescriptive Method at the out-of-plane seismic load for Seismic Design Categories D_1 and D_2, it is clear that the minimum vertical reinforcement requirement of one No. 5 bar at 18 inches (457 mm) on center provides more than adequate out of plane bending resistance for Seismic Design Categories D_1 and D_2.

Wall Type	Weight (psf)	Seismic Design Category C		Seismic Design Category D_1		Seismic Design Category D_2	
		S_{DS}	F_p (psf)	S_{DS}	F_p (psf)	S_{DS}	F_p (psf)
3.5-in Flat	43.75	0.5g	8.8	0.83g	14.5	1.17g	20.5
5.5-in Flat	68.75	0.5g	13.8	0.83g	22.8	1.17g	32.2
6-in Waffle	56.00	0.5g	11.2	0.83g	18.6	1.17g	26.2
8-in Waffle	76.00	0.5g	15.2	0.83g	25.2	1.17g	35.6
6-in Screen	53.00	0.5g	10.6	0.83g	17.6	1.17g	24.8

Figure B4.1
Out-of-Plane Seismic Loads for Seismic Design Category C, D_1, and D_2

B4.3 6-Inch- (152-mm-) Thick Waffle-Grid ICF Above-Grade Wall

A waffle-grid ICF above-grade wall is selected from Table 4.3 of the *Prescriptive Method* for above-grade walls constructed in an area where the design wind pressure is 30 psf (1.44 kPa). The above-grade wall is 9 feet (2.7 m) high and supports a light-frame roof only. Table 4.3 shows that the above-grade wall requires one #4 bar at 48 in (1.2 m) on center. Table 4.5 shows that the above-grade wall requires horizontal reinforcement in the form of one No. 4 bar at third points in the wall story and one No. 4 bar within 12 inches (305 mm) of the top of the wall story. Calculate the capacity and check the adequacy of the 6-inch- (152-mm-) thick waffle-grid ICF above-grade wall.

Using the values in Figure B1.1 and the material properties in Section 2.0, the amount of vertical wall reinforcement required was determined as follows.

The wall moments are calculated by multiplying the axial load by the assumed eccentricities of the roof, wall above, and floor as applicable. The controlling condition is when the gravity load is minimized and the moment is maximized.

Dead Loads

Roof and Ceiling	0 *plf*	[0 *kN/m*]
First-Story ICF Wall	0.5(9.0 *ft*)(55 *psf*) = 248 *plf* @ *mid-height*	[3.6 *kN/m*]

Live Loads

Roof and Ceiling	0 *plf*	[0 *kN/m*]
Attic	0 *plf*	[0 *kN/m*]

First-Story Wall Moments

Dead Load $_{@top}$	0 *in-lb/lf*	[0 *N-m/m*]
Live Load $_{@top}$	0 *in-lb/lf*	[0 *N-m/m*]
Wind Load $_{@midht}$	$\frac{30\,psf(9\,ft)^2(1\,ft)}{8} = 303.8\,ft - lb/lf = 3{,}645\,in - lb/lf$	[1.3 *kN-m/m*]

Parallel Shear

Refer to Section 5.0 for parallel shear calculations.

Perpendicular Shear

Refer to Section 7.0 for variable definitions.

$$V_u = \frac{ql}{2} = \frac{(30\,psf)(9\,ft)}{2} = 135\,plf\left(\frac{ft}{1 vertical core}\right) = 135\,lb/post \qquad [433\ kN/post]$$

Load Combinations

The following ACI 318 and IBC Load Combinations were used for design purposes [B3] [B7]:

(1) U = 1.4D + 1.7L
(2) U = 0.75 (1.4D + 1.7L + 1.7W)
(3) U = 1.05D + 1.28L + 1.0E
(4) U = 0.9D + 1.3W
(5) U = 0.9D + 1.0E

All out-of-plane bending design cases were controlled by load combination (4) for wind loads and load combination (5) for earthquake loads.

The hand calculations above were consolidated into spreadsheet form and factored following ACI 318 [B3] and IBC [B7] guidelines. The factored moments were also magnified following the requirements found in ACI 318 section 10.12 [B3]. The maximum moment in the wall was determined at the mid-point of the wall. With the maximum factored axial loads and maximum factored moments the construction cases were plotted on a factored P_u-M_u curve constructed following conventional reinforced concrete practices and ACI recommended resistance factors. The following sections provide example calculations and an example P_u-M_u curve.

Check Perpendicular Shear

The critical factored perpendicular shear load, V_u, experienced by the first-story wall occurs at the bottom of the wall story due to ACI [B3] Load Combination (4). Consider a reinforced section.

$V_u = (1.3)(135\,lb/post) = 176\,lb/post$ [0.78 *kN/post*]

$\phi V_n = 0.85(2)\sqrt{f'_c}\,bh = 0.85(2)\sqrt{2{,}500\,psi}(6.25in)(5in) = 2{,}656lb/post$ [11.8 *kN/post*]

$V_u \leq \phi V_n \quad OK$

Check Compression and Tension

The critical maximum moment, M_u, experienced by the first-story wall occurs at mid-height due to ACI [B3] Load Combination (4). The corresponding total factored axial load, P_u, is also taken at mid height of the first story-wall based on ACI [B3] Load Combination (4).

$M_u = (1.3)(3{,}645\,in-lb/ft)(ft/post) = 4{,}739\,in-lb/post$ [0.54 *kN-m/post*]

$P_u = (0.9)(248plf) = 223plf = 223lb/post$ [1.0 *kN/post*]

Determine Magnified Moment

With one exception, the equations below are taken from ACI 10.12 [B3]. The equation for *EI*, as listed in ACI 10.12.3 [B3], is applicable to wall sections that contain a double layer of reinforcement. Given that ICFs contain only one layer of reinforcement, the equation for *EI* noted below is used instead [B8].

$M_u = (1.3)(3{,}645in-lb/lf) = 4{,}739in-lb/post$ [0.54 *kN-m/post*]

$P_u = (0.9)(248plf) = 223plf = 223lb/post$ [1.0 *kN/post*]

$E_c = 57{,}000\sqrt{f_c'} = 57{,}000\sqrt{2{,}500psi} = 2{,}850{,}000psi$ [19.7 *GPa*]

From ACI Section 10.12.3 [B3], assume $\beta_d = 0.6$, therefore;

$$EI = 0.25E_cI_g = 0.25(2{,}850{,}000psi)\left(\frac{(6.25in)(5in)^3}{12}\right) = 46{,}386{,}718psi$$ [320 *GPa*]

$M_{2,min} = P_u(0.6+0.03h) = 223lb(0.6+0.03(5in)) = 167.3in-lb$ [18.9 *N-m*]

$C_m = 1.0$ *for members with transverse loads between supports*

$$P_c = \frac{\pi^2 EI}{(kl_u)^2} = \frac{\pi^2(46{,}386{,}718psi)}{[(1.0)(9ft)^2](12in/ft)^2} = 39{,}211lb$$ [174 *kN*]

$$\delta_{ns} = \frac{C_m}{1-\left(\frac{P_u}{0.75P_c}\right)} \geq 1.0 = \frac{1.0}{1-\left(\frac{223lb}{0.75(39{,}211lb)}\right)} = 1.008$$

$M_{ns} = \delta_{ns}M_u = 1.008(4{,}739in-lb) = 4{,}777in-lb$ [0.53 *kN-m*]

Determine Points for the Interaction Diagrams

Interaction Diagram for one No. 4 bar spaced 48 in (1.2 m) on center

Point 1 – Pure Compression

$$P_o = 0.85 f_c'(A_{concrete} - A_s) + f_y A_s$$

$$= 0.85(2{,}500\,psi)(6.25in(5in) - 0.2in^2(12in/48in)) + 40{,}000\,psi(0.2in^2)(12in/48in) = 68{,}300lbf \qquad [967\ kN/m]$$

$$P_{n\max} = 0.8P_o = 0.8(68{,}300lbf) = 54{,}640lbf \qquad [774\ kN/m]$$

$$P_u = \phi\ P_{n\max} = 0.7(54{,}640lbf) = 38{,}248lbf \qquad [542\ kN/m]$$

$$\phi\,M_n = 0$$

Point 2 – Balanced Condition

The balanced condition represents the dividing point between compression controls and tension controls regions of the strength interaction diagram. It is defined by the simultaneous occurrence of a strain of 0.003 in the extreme fiber of the concrete and the strain $\varepsilon_y = f_y/E_s$ on the tension steel.

Distance to the neutral axis:

$$x_b = \frac{87{,}000d}{f_y + 87{,}000} = \frac{87{,}000(5in/2)}{40{,}000\,psi + 87{,}000} = 1.71in \qquad [43.4\ mm]$$

Compression force:

$$C_c = 0.85 f_c' \beta\ x_b b = 0.85(2{,}500\,psi)(0.85)(1.71in)(6.25in) = 19{,}304lb \qquad [85.9\ kN]$$

Tension force:

$$T = A_s f_y = (0.2in^2)(12in/48in)(40{,}000\,psi) = 2{,}000lb \qquad [8.9\ kN]$$

Factored Balanced Axial Load:

$$\phi\ P_b = \phi(C_c - T) = 0.7(19{,}304lb - 2{,}000lb) = 12{,}113lb \qquad [53.9\ kN]$$

Factored Balanced Moment:

$$\phi M_b = \phi C_c\left(d - \frac{\beta\ x_b}{2}\right) = 0.7(19{,}304lb)\left(\frac{5in}{2} - \frac{0.85(1.71in)}{2}\right) = 23{,}962lb - in \qquad [2.7\ kN\text{-}m]$$

Point 3 – Pure Bending

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.2in^2)(12in/48in)(40{,}000psi)}{0.85(2{,}500psi)(6.25in)} = 0.15in \qquad [3.8\ mm]$$

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2} \right) = 0.9(0.2in^2)(12in/48in)(40{,}000psi)\left(\frac{5in}{2} - \frac{0.15in}{2} \right) = 4{,}365lb - in \qquad [482\ N\text{-}m]$$

$$\phi P_u = 0$$

Additional points were determined in the compression controls region (between Point 1 and Point 2) and the tension controls region (between Point 2 and Point 3) to populate the interaction diagrams. This procedure was duplicated for numerous reinforcement schedules to obtain the most efficient design possible.

Determine Reinforcement

Plot the magnified moment and the corresponding total factored axial load on the interaction diagram for a 6-inch (152-mm) thick waffle-grid wall. This diagram was constructed for each possible configuration using a range of reinforcement ratios that extend below the ACI minimum. Figure B4.2 is a close up of the lowest portion of the curve since the requirements of the wall system for out-of-plane bending are in the tension controlled region. It should be noted that increasing the gravity load in this region of the P_u-M_u will increase the wall strength against transverse loads.

The interaction diagrams shown are based on 2,500 psi (17.2 MPa) concrete, 40,000 psi (276 MPa) reinforcing steel, and the reinforcing steel being centered in the wall cross section. This procedure was repeated for all possible permutations including 3,000 psi (20.6 MPa) concrete, 4,000 psi (27.6 MPa) concrete, and 60,000 psi (414 MPa) reinforcing steel to determine the relationship between the differing material properties.

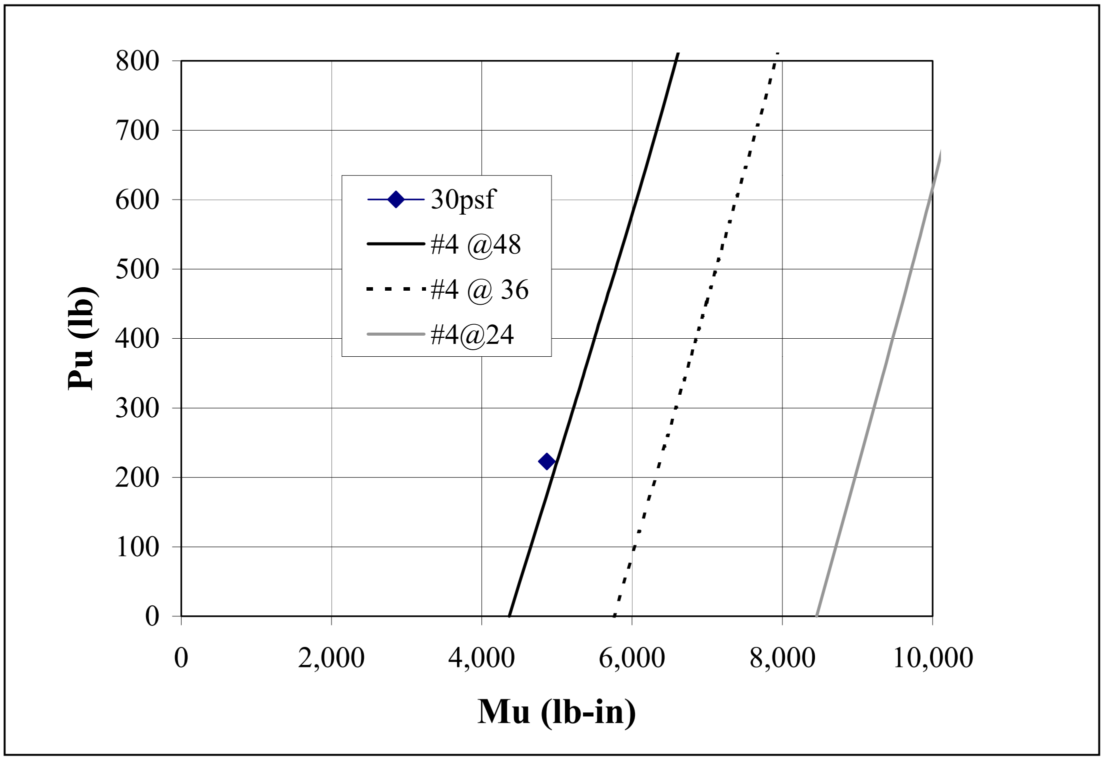


Figure B4.2 Factored Loads on the $P_u - M_u$ Diagram for Construction Case B1

<u>Check Deflection</u>

For above-grade walls, a deflection limit of *L/240* for total service loads is used. To calculate wall deflection at service load levels, effective section properties of the assumed cracked concrete section are based on $0.1E_cI_g$.

Assume a gypsum board interior finish exposed to view. The deflection calculations below are based on wind loads. Refer to Section 7.0 for the maximum deflection equation.

$$\Delta_{actual} = \frac{5(30psf)(1ft)(9ft)^4\left(\frac{1{,}728in^3}{ft^3}\right)}{(0.1)(384)(2{,}850{,}000psi)\left(\frac{(6.25in)(5in)^3}{12}\right)} = 0.24in \qquad [6.1\ mm]$$

$$\Delta_{allowable} = \frac{(9ft)\left(\frac{12in}{ft}\right)}{240} = 0.45in \qquad [11.4\ mm]$$

$$\Delta_{actual} < \Delta_{allowable} \qquad OK$$

4.2.2 Construction Case Summary

From Table 4.3 of the *Prescriptive Method*, we obtain one vertical Grade 40 (300 MPa), No. 4 bar at 48 inches (1.2 m) on center, and from Section 4.5 one horizontal No. 4 bar near third points in the wall story. Note that in Seismic Design Categories D_1 and D_2 the minimum vertical and horizontal reinforcement shall be one No. 5 rebar at a maximum spacing of 18 inches (457 mm) on center and the minimum concrete compressive strength shall be 3,000 psi (20.5 MPa) [B3] [B7].

TABLE 4.3
MINIMUM VERTICAL WALL REINFORCEMENT
FOR WAFFLE-GRID ICF ABOVE-GRADE WALLS [1,2,3,4]
(excerpt from the *Prescriptive Method*)

Design Wind Pressure (Table 4.1)	Maximum Wall Height per Story	Minimum Vertical Reinforcement[5]					
		Supporting Light-Frame Roof Only		Supporting Light-Frame Second Story and Roof		Supporting ICF Second Story and Light-Frame Roof	
		Minimum Wall Thickness (inches)					
(psf)	(feet)	6	8	6	8	6	8
20	8	#4@48	#4@48	#4@48	#4@48	#4@48	#4@48
	9	#4@48	#4@48	#4@48	#4@48	#4@48	#4@48
	10	#4@48	#4@48	#4@48	#4@48	#4@48	#4@48
30	8	#4@48	#4@48	#4@48	#4@48	#4@48	#4@48
	9	**#4@48**	#4@48	#4@48	#4@48	#4@48	#4@48
	10	#4@36; #5@48	#4@48	#4@36; #5@48	#4@48	#4@36; #5@48	#4@48

B5.0 ICF WALL OPENING DESIGN EXAMPLES AND ENGINEERING CALCULATIONS

B5.1 Calculating In-Plane Shear Due to Wind

For illustration of the design methods used, a building sited in Exposure C where wind speeds are 90 mph (145 km/hr) is selected from Table 5.1 of the *Prescriptive Method* for above-grade walls. Table 5.1 shows that the design velocity pressure is 21 psf (1.0 kPa). This velocity pressure is used to select the minimum solid wall length requirements for in-plane shear for the various ICF systems. The following calculations determine the velocity pressure using the main wind force resisting systems for a building sited in Exposure C where the wind speeds are 90 mph (145 km/hr). A mean roof height of 35 ft (10.7 m) was used for all cases.

For the purpose of illustrating the determination of building lateral loads or in-plane shear wall loads due to winds, consider a one-story house with a 6:12 roof pitch for determining the shear loads on the building. Consider a building plan of 40ft (12.2 m) – perpendicular to ridge (end wall length) – by 60ft (18.3 m) – parallel to ridge (side wall length). Lateral wind loads are assigned to the exterior wall lines by the tributary area method. Torsion is not considered since the horizontal diaphragm is considered flexible in comparison with the concrete shear walls in using the more conservative tributary area load distribution method. Therefore, only Case A is considered in Figure 6-4 of ASCE 7-98 [B1] as is typical for residential design practice.

B5.1.1 Determine the Velocity Pressure

$$q_h = 0.00256 K_z K_{zt} K_d V^2 I = (0.00256)(1.01)(1.0)(1.0)(90mph)^2(1.00) = 20.94 psf \quad [998.8 \text{ Pa}]$$

The value above corresponds to the value of 21 psf (1.0 kN/m^2) in the *Prescriptive Method* (Table 5.1).

Note: A K_d factor of 1.0 is used for concrete design in accordance with the *International Building Code*, Section 1605.2.1 [B7].

B5.1.2 Determine the In-Plane Shear Load

Using Figure 6-4 from ASCE 7-98 [B1].

a: Smaller of 10 percent of least building plan dimension (0.1*40ft =4ft) or 0.4*height of the building (0.4*35ft=14ft), but not less than 4% of least horizontal dimension (0.04(40ft) = 1.6 ft) or 3ft (0.9m).

Therefore a = 4ft [1.2 m]

For 6:12 roof pitch, the roof angle = tan^{-1}(6/12) = 26.6 degrees. Determine the external pressure coefficient through interpolation of Table 6-4 of ASCE 7-98 [B1]. The results of interpolation are shown below.

Pitch	θ	Zones							
		1	2	3	4	1E	2E	3E	4E
6:12	26.6	0.550	-0.099	-0.447	-0.391	0.728	-0.190	-0.585	-0.535
0:12	0.0	0.400	X	X	-0.290	0.610	X	X	-0.430

Source: ASCE 7-98 Table 6-4 [B1]

B5.1.2.1 Wind Perpendicular to Ridge (building orientation of 0 degrees)

<u>Determine Net External Pressure Coefficients * Surface Area</u>

Walls

$$\sum\left(GC_{pf}\right) * Surface\ \ Area$$
$$= 0.5\big[10\,ft\big[0.550\,(22\,ft) + 0.728(8\,ft)\big] - 10\,ft\big[-0.391(22\,ft) - 0.535(8\,ft)\big]\big]$$
$$= 154.03\,ft^2$$

Roof

$$\sum\left(GC_{pf}\right) * Surface\ \ Area$$
$$= \left(\frac{20\,ft}{\cos 26.6}\right)\big(-0.099(22\,ft) - 0.190(8\,ft) + 0.447(22\,ft) + 0.585(8\,ft)\big)\sin 26.6$$
$$= 108.32\,ft^2$$

Since the contribution from the roof is positive, it is included. This is true for all roof slopes greater than approximately 5:12 in accordance with Figure 6-4 of ASCE 7-98 [B1].

<u>Determine Wind Load (40 ft End Wall)</u>

$$V_{parallel} = q_h \sum\left(GC_{pf}\right) x\, Surface\ Area = (20.94\,psf)(154.03\,ft^2 + 108.32\,ft^2) = 5{,}494\,lb \qquad [24.4\text{ kN}]$$

$$V_u = 1.3\left(5{,}494\,lb\right) = 7{,}142\,lb \qquad [31.7\text{ kN}]$$

B5.1.2.2 Wind Parallel to Ridge (building orientation rotated 90 degrees)

<u>Determine Net External Pressure Coefficients *x* Surface Area</u>

Walls

$$\sum\left(GC_{pf}\right) * Surface\ \ Area$$
$$= 0.5\big[10\,ft\big[0.400\,(12\,ft) + 0.610(8\,ft)\big] - 10\,ft\big[-0.290(12\,ft) - 0.430(8\,ft)\big]\big] \qquad [7.7\text{ m}^2]$$
$$= 83.0\,ft^2$$

Gable end portion of wall. Use roof slope equal to zero to determine coefficients.

$$\sum\left(GC_{pf}\right) * Surface\ \ Area$$
$$= \frac{1}{2}\left(\frac{6}{12}\right)(8\,ft)(8\,ft)(0.610 + 0.430)\ + \left(\frac{6}{12}\right)(8\,ft)(12\,ft)(0.400 + 0.290)$$
$$+ \frac{1}{2}\left(\frac{6}{12}\right)(12\,ft)(12\,ft)(0.400 + 0.290)$$
$$= 74.6\,ft^2$$

Total external pressure coefficients weighted by tributary surface area:

$$=83.0ft^2 + 74.6ft^2 = 157.6ft^2 \qquad [14.6\ m^2]$$

Determine Wind Load (60 ft Side Wall)

$$V_{parallel} = q_h \sum\left(GC_{pf}\right) = (20.94\,psf)(157.6\,ft^2) = 3{,}300\,lb \qquad [14.7\ kN]$$

$$V_u = 1.3(3{,}300\,lb) = 4{,}290\,lb \qquad [19.1\ kN]$$

B5.2 Minimum Length of Solid Wall Along Exterior 6-Inch- (152-mm-) Thick Waffle-Grid ICF Above-Grade Wall

A waffle-grid ICF above-grade wall is selected from Table 5.3A of the *Prescriptive Method* for above-grade walls. The building is constructed in an area where the wind speeds equate to a velocity pressure of 21 psf (1.0 kPa) in Table 5.1 and is located in Seismic Design Category B. The above-grade wall is 9 feet (2.7 m) high and supports a light-frame roof only. Consider a building where the side wall length (parallel to ridge) is 60 ft (18.3 m) and the building end wall length (perpendicular to ridge) is 40 ft (9.1 m). Assume the building has a 6:12 roof slope. Through interpolation, Table 5.3A of the *Prescriptive Method* shows that a minimum length of solid wall of 5.7 ft (1.7 m) is required. Calculate the capacity and check the adequacy of the 6-inch- (152-mm-) thick waffle-grid ICF load-bearing above-grade wall for parallel shear.

The tables in Sections 3.0 and 4.0 are based on ICF walls without door or window openings. This simplified approach rarely arises in residential construction since walls generally contain windows and doors. The amount of openings affects the lateral (racking) strength of the building, particularly for wind and seismic loading conditions. The *Prescriptive Method* provides recommendations for the amount and placement location of additional reinforcement required around openings. It also addresses the minimum amount of solid wall required to resist racking loads from wind and moderate seismic forces.

The values for the minimum required length of solid wall along exterior wall lines listed in Table 5.2 through Table 5.4 of the *Prescriptive Method* were calculated using the main wind force resisting wind loads in accordance with ASCE 7 [B1]. The ICF walls were checked using resistance models for multiple building dimensions.

A shear model following the methods outlined in the *Uniform Building Code* (UBC) regarding shear wall design was used. This method linearly varies the resistance of a wall from a cantilevered beam model at an aspect ratio (height over width) of 4 to a solid shear wall for all segments greater than 2. All walls are required to have a minimum 2 ft solid wall adjacent to all corners. This methodology was also confirmed in the research conducted at the NAHB Research Center, Inc. entitled *In-Plane Shear Resistance of Insulating Concrete Form Walls* [B9].

Therefore, for conservative analysis purposes, the two foot corner elements were considered flexural elements and included their resistance in determining the shear wall capacity. The amount of solid wall required includes the contribution of minimum 2ft (0.6 m) corners plus additional length of solid wall to meet the design wind or seismic load.

B5.2.1 ASCE 7 Wind Loads

The in-plane shear loads were determined previously in Section 5.1 of the Technical Substantiation.

$$V_u = 1.3(5{,}494lb) = 7{,}142lb \qquad [31.8 \text{ kN}]$$

B5.2.2 Concrete Resistance

<u>Shear Elements</u>

$$\phi V_c = \phi 2\sqrt{f'_c}hbx = 0.85(2)\sqrt{2{,}500psi}(5in)(6.25in/ft)x = (2{,}656lb/ft)x \qquad [1.9 \text{ kN/m}]x$$

<u>Flexural Elements (Corners)</u>

Calculate the depth of the compressive stress block.

$$a = \frac{A_s f_y}{0.85 f'_c h} = \frac{(0.20in^2)(40ksi)}{(0.85)(2.5ksi)(6.25in)} = 0.60in \qquad [19.1 \text{ mm}]$$

Calculate the nominal moment strength.

$$M_n = A_s f_y\left(d - \frac{a}{2}\right) = (0.20in^2)(40ksi)\left(24in - 4in - \frac{0.6in}{2}\right)(1000lb/kip) = 157{,}600in - lb \qquad [17.7 \text{ MN–m}]$$

Calculate the factored shear force from the bending moment:

$$\phi V_m = \phi M_u / h = (0.9)(157{,}600in - lb)/(9ft)(12in/ft) = 1{,}313lb \qquad [5.8 \text{ kN}]$$

Determine Required Wall Length

The total shear in the wall line is resisted by the 2 corners and "*x*" length of shear panels.

$$V_u \leq 2(\phi V_m) + x(\phi V_c)$$

$$x = \frac{V_u - 2(\phi V_m)}{\phi V_c} = \frac{7{,}142 lb/ft - 2(1{,}313 lb)}{2{,}656 lb/ft} = 1.7\,ft$$ [0.5 m]

Total wall length = 2 ft corner + 2 ft corner + 1.7 ft = 5.7 ft as shown in Table 5.3A through interpolation of the highlighted values.

The required minimum solid wall lengths were determined in this manner and are listed in Table 5.2 through 5.4.

TABLE 5.3A
MINIMUM SOLID END WALL LENGTH
REQUIREMENTS FOR WAFFLE-GRID ICF WALLS
FOR WIND PERPENDICULAR TO RIDGE [1,2,5,4,5]
(excerpt from the *Prescriptive Method*)

Design Velocity Pressure (psf)			20	25	30	35	40	45	50	60
Wall Category	**Building Side Wall Length, L (feet)**	**Roof Slope**	**Minimum Solid Wall Length on Building End Wall**							
One Story/ Top Story of Two Story	**16**	**≤ 1:12**	4.00	4.00	4.00	4.00	4.00	4.00	4.00	4.25
		5:12	4.00	4.00	4.00	4.00	4.25	4.25	4.50	4.75
		7:12	4.00	4.25	4.50	4.75	5.00	5.25	5.50	6.00
		12:12	4.50	4.75	5.00	5.50	5.75	6.00	6.50	7.00
	24	**≤ 1:12**	4.00	4.00	4.00	4.00	4.25	4.25	4.50	4.75
		5:12	4.00	4.00	4.25	4.25	4.50	4.75	4.75	5.25
		7:12	4.50	4.75	5.25	5.50	5.75	6.25	6.50	7.25
		12:12	5.00	5.50	6.00	6.50	7.00	7.50	7.75	8.75
	32	**≤ 1:12**	4.00	4.00	4.00	4.25	4.50	4.50	4.75	5.00
		5:12	4.00	4.25	4.50	4.75	4.75	5.00	5.25	5.75
		7:12	5.00	5.25	5.75	6.25	6.75	7.00	7.50	8.50
		12:12	5.50	6.25	6.75	7.50	8.00	8.75	9.25	10.50
	40	**≤ 1:12**	4.00	4.00	4.25	4.50	4.75	5.00	5.00	5.50
		5:12	4.25	4.50	4.75	5.00	5.25	5.50	5.75	6.25
		7:12	5.25	5.75	6.25	7.00	7.50	8.00	8.50	9.50
		12:12	6.25	7.00	7.75	8.50	9.25	10.00	10.75	12.25
	50	**≤ 1:12**	4.00	4.25	4.50	4.75	5.00	5.25	5.50	6.00
		5:12	4.50	4.75	5.00	5.25	5.75	6.00	6.25	7.00
		7:12	5.75	6.50	7.25	7.75	8.50	9.25	9.75	11.00
		12:12	6.75	7.75	8.75	9.50	10.50	11.50	12.50	14.25
	60	**≤ 1:12**	4.25	4.50	4.75	5.00	5.25	5.75	6.00	6.50
		5:12	**4.75**	**5.25**	5.50	5.75	6.25	6.50	7.00	7.75
		7:12	**6.25**	**7.25**	8.00	8.75	9.50	10.25	11.00	12.75

B5.3 Minimum Percentage of Solid Wall Length Along Exterior Above-Grade Walls for Seismic Design Categories C, D_1, and D_2

The amount of openings in a wall affects the lateral or in-plane shear (racking) strength of the building, particularly for wind and seismic loading conditions. The *Prescriptive Method* provides recommendations for the amount and placement location of additional reinforcement required around openings. It also addresses the minimum amount of solid wall required to safely resist racking loads from wind and seismic forces. In addition, minimum amounts of solid wall as a percentage of wall length were increased above calculated amounts to ensure that walls behaved as cantilevered structural concrete walls rather than a moment frame. This conservative adjustment, in part, is a result of in-plane shear tests of ICF walls with varying opening amounts [B9].

The values for the base percentage of solid wall length along exterior wall lines listed in Table 5.5 of the *Prescriptive Method* were calculated using seismic load provisions in accordance with the IBC [B7]. The ICF walls were checked using three resistance models for a 2:1 length-to-width building aspect ratio (the higher load for endwalls was used as the basis of loading for all walls). Each model considered a 15 psf (0.7 kPa) roof dead load, 10 psf (0.5 kPa) floor dead load, 10 psf (0.5 kPa) interior wall dead load, a 70 psf (3.3 kPa) ground snow load, and the weight of the ICF walls. Multiple models were used to bound the range of plausible designs due to the uncertainty of the performance of building under high seismic loads. The minimum reinforcement requirement for Seismic Design Categories C, D_1, and D_2 were used as required in ACI 318 Chapter 21 [B3] and the IBC [B7] for ordinary reinforced concrete shear walls (Seismic Design Category C) and special reinforced concrete shear walls (Seismic Design Categories D_1 and D_2).

Model 1

Model 1 considered the walls in Seismic Design Category C as ordinary reinforced concrete with a Response Modification Coefficient (R) of 4.5 and the walls in Seismic Design Category D_1 and D_2 as special reinforced concrete with a Response Modification Coefficient (R) of 5.5. The resistance of the walls were considered to have two-2 ft (0.6 m) wide flexural controlled elements at the corners of the building and a shear controlled element (minimum 4 ft length) somewhere along the wall line of interest.

Model 2

Model 2 considered the walls in Seismic Design Category C as ordinary reinforced concrete with a Response Modification Coefficient (R) of 4.5 and the walls in Seismic Design Category D_1 and D_2 as special reinforced concrete with a Response Modification Coefficient (R) of 5.5. The resistance of the walls were considered to be provided only by 2 ft (0.6 m) wide by 8 ft (2.4 m) tall flexural controlled elements (cantilevers) along the wall line.

Model 3

Model 3 considered a near elastic design using a Response Modification Coefficient (R) of 2.5 Seismic Design Categories C, D_1, and D_2 (even though the walls are reinforced as required for ordinary or special reinforced concrete walls). The resistance of the walls were considered to have shear controlled elements along the wall line.

All models considered a minimum concrete compressive strength of 2,500 psi (17.3 MPa) for Seismic Design Category C and 3,000 psi (20.7 MPa) for Seismic Design Category D_1 and D_2. A strength reduction factor for shear of 0.60 was used for Seismic Design Categories D_1 and D_2 in accordance with ACI 318 Section 9.3.4 [B3]. Figure B5.3.1 summarizes the loads on the building and Figure B5.3.2 summarizes the required percentages from each model. The models were used to bound the design of percent solid wall length for different levels of conservatism. The amount of reinforcement remains the same for each model and is in compliance with the provisions of ACI 318 – Chapter 21 for Special Reinforced Concrete Walls [B3]. The same resistance calculations were used to determine the required resistance of the wall line as those described in Section B5.2.2.

Wall Type	Wall Supporting Light Frame Roof Only			Wall Supporting Light Frame Second Story and Light Frame Roof			Wall Supporting ICF Second Story and Light Frame Roof		
	Model 1	Model 2	Model 3	Model 1	Model 2	Model 3	Model 1	Model 2	Model 3
Seismic Design Category C									
Flat, 3.5	4,439	4,439	7,990	7,302	7,302	13,144	10,250	10,250	18,450
Flat, 5.5	5,485	5,485	9,873	8,294	8,294	14,929	13,400	13,400	24,120
Flat, 7.5	6,575	6,575	11,835	9,225	9,225	16,605	16,660	16,660	29,988
Waffle, 6	4,952	4,952	8,914	7,687	7,687	13,837	11,788	11,788	21,218
Waffle, 8	5,806	5,806	10,451	8,584	8,584	15,451	14,352	14,352	25,834
Screen, 6	4,824	4,824	8,683	7,558	7,558	13,604	11,405	11,405	20,529
Seismic Design Category D_1									
Flat, 3.5	6,102	6,102	13,424	10,037	10,037	22,081	14,090	14,090	30,998
Flat, 5.5	7,570	7,570	16,654	11,447	11,447	25,183	18,495	18,495	40,689
Flat, 7.5	9,039	9,039	19,886	12,680	12,680	27,896	22,900	22,900	50,380
Waffle, 6	6,807	6,807	14,975	10,565	10,565	23,243	16,204	16,204	35,649
Waffle, 8	7,981	7,981	17,558	11,799	11,799	25,958	19,729	19,729	43,404
Screen, 6	6,631	6,631	14,588	10,389	10,389	22,856	15,676	15,676	34,487
Seismic Design Category D_2									
Flat, 3.5	8,543	8,543	18,795	14,052	14,052	30,914	19,726	19,726	43,397
Flat, 5.5	10,599	10,599	23,318	16,026	16,026	35,257	25,893	25,893	56,965
Flat, 7.5	12,654	12,654	27,839	17,752	17,752	39,054	32,060	32,060	70,532
Waffle, 6	9,530	9,530	20,966	14,792	14,792	32,542	22,686	22,686	49,909
Waffle, 8	11,174	11,174	24,583	16,519	16,519	36,342	27,620	27,620	60,764
Screen, 6	9,283	9,283	20,423	14,545	14,545	31,999	21,946	21,946	48,281

Figure B5.3.1 In-Plane Seismic Loads for Seismic Design Categories C, D_1, and D_2 for the Models Investigated

Wall Type	Wall Supporting Light Frame Roof Only			Wall Supporting Light Frame Second Story and Light Frame Roof			Wall Supporting ICF Second Story and Light Frame Roof		
	Model 1	Model 2	Model 3	Model 1	Model 2	Model 3	Model 1	Model 2	Model 3
Seismic Design Category C									
Flat, 3.5	1.3%	9.6%	7%	4.0%	15.9%	12%	6.7%	22.3%	17%
Flat, 5.5	1.4%	11.8%	6%	3.1%	17.8%	9%	6.1%	28.8%	14%
Flat, 7.5	1.5%	14.1%	5%	2.7%	19.8%	7%	5.9%	35.7%	13%
Waffle, 6	2.9%	12.5%	11%	6.3%	19.4%	17%	11.5%	29.8%	27%
Waffle, 8	2.5%	14.6%	8%	4.7%	21.6%	12%	9.4%	36.1%	21%
Screen, 6	2.4%	11.0%	11%	6.0%	17.2%	18%	11.0%	25.9%	27%
Seismic Design Category D_1									
Flat, 3.5	3.7%	13.3%	16%	8.4%	21.8%	27%	13.3%	30.6%	37%
Flat, 5.5	3.4%	16.3%	13%	6.4%	24.6%	19%	11.8%	39.8%	31%
Flat, 7.5	3.3%	19.4%	11%	5.4%	27.2%	16%	11.2%	49.1%	28%
Waffle, 6	6.8%	17.2%	24%	12.9%	26.7%	38%	22.0%	40.9%	58%
Waffle, 8	5.5%	20.1%	18%	9.5%	29.7%	27%	17.7%	49.6%	45%
Screen, 6	6.2%	15.1%	24%	12.5%	23.6%	38%	21.4%	35.6%	58%
Seismic Design Category D_2									
Flat, 3.5	6.6%	18.6%	23%	13.3%	30.5%	37%	20.1%	42.8%	52%
Flat, 5.5	5.8%	22.8%	18%	9.9%	34.5%	27%	17.5%	55.7%	44%
Flat, 7.5	5.4%	27.1%	16%	8.2%	38.0%	22%	16.3%	68.7%	40%
Waffle, 6	11.2%	24.1%	34%	19.7%	37.3%	53%	32.5%	57.3%	81%
Waffle, 8	8.8%	28.1%	25%	14.4%	41.5%	38%	25.8%	69.4%	63%
Screen, 6	10.6%	21.1%	34%	19.5%	33.0%	54%	31.9%	49.8%	81%

Figure B5.3.2 Minimum Required Percentages of Solid Wall Length for Seismic Design Categories C, D_1, and D_2 for the Models Investigated

The values in Figure B5.3.2 were examined for each wall type, model and Seismic Design Category. The wall type has little affect on the percentage of solid wall length required. Although the resistance increases as the thickness of the walls increase, so does the weight and therefore seismic load on the wall line. Therefore, a single minimum percentage was chosen for each category regardless of wall type. The percentage solid wall length found in the *Prescriptive Method* Table 5.5 also factors in the results of full-scale ICF wall tests with openings [B9]. This testing indicates unfavorable failure modes related to cracks that initiated at corners of openings for walls with small percentages of solid wall length. Therefore, the simplified values in Table 5.5 were generally chosen to be conservative relative to the testing and analysis to ensure adequate strength and ductility and to prevent moment frame behavior of walls intended to behave as cantilevered structural concrete walls. In addition, the added requirement of a minimum shear wall segment length of 4 feet was applied to buildings in Seismic Design Categories D_1 and D_2.

B5.4 ICF Lintel Design Examples and Engineering Calculations

The following engineering calculations are provided as supplemental information to illustrate the means and methods for the development of the tables included in the *Prescriptive Method*. Structural calculations illustrate the method for calculating the reinforcement requirements and adjustment factors used within the tables. The example calculations are not intended to be inclusive of all design considerations for a given application, but rather are intended to illustrate the derivation of the tables in the *Prescriptive Method*.

B5.4.1 Flat ICF Lintel Design without Stirrups in a Load-Bearing ICF Wall

A flat ICF lintel is selected from Table 5.7 of the *Prescriptive Method* for lintels supporting a light frame second floor and a light-frame roof and subjected to a 30 psf (1.4 kPa) ground snow load. The lintel's nominal thickness is 5.5 inches (140 mm), with a depth of 20 inches (508 mm). A building with a 32 ft (9.8 m) floor and roof clear span is considered. Table 5.7 shows that the lintel without stirrups and a minimum No. 4 bar has a maximum clear span of 6'-0" (1.8 m). Calculate the capacity and check the adequacy of the 5.5-inch x 20-inch (140-mm x 508-mm) flat ICF lintel.

B5.4.1.1 Maximum Allowable Span Due to Bending Moment

$$M_u = \phi M_n$$

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2} \right)$$

$$\phi = 0.9 \ \textit{(strength reduction factor)}$$

Calculate the reinforcement ratio for one No. 4 bottom bar.

$$\rho = \frac{A_s}{A_c} = \frac{0.20\,in^2}{(5.5\,in)(18\,in)} = 0.0020$$

$$\rho_b = \frac{0.85 f'_c \beta_1}{f_y} \left(\frac{87{,}000}{f_y + 87{,}000} \right) = \frac{0.85(2{,}500\,psi)(0.85)}{40{,}000\,psi} \left(\frac{87{,}000}{40{,}000\,psi + 87{,}000} \right) = 0.0309$$

$$\rho_{max} = 0.75\rho_b = 0.75(0.0309) = 0.0232$$

$$\rho_{min} = 0.0012$$

$$\textit{Since } \rho_{max} \ge \rho_b \ge \rho_{min} \textit{ OK.}$$

Calculate the depth of the compressive stress block.

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.20in^2)(40ksi)}{(0.85)(2.5ksi)(5.5in)} = 0.62\,in \qquad [15.7\ mm]$$

Calculate the nominal moment strength.

$$M_n = A_s f_y \left(d - \frac{a}{2} \right) = (0.2in^2)(40ksi)\left(18in - \frac{0.62in}{2} \right) = 142in - kip \qquad [15.9\ kN\text{-}m/m]$$

Calculate the factored bending moment.

$$M_u = \phi M_n = (0.9)(142 in - kip)/12 = 10.7 ft - kip \qquad [14.4\ kN\text{-}m]$$

Calculate the load on the lintel.

Live Loads			
	Snow = (0.7)(30 *psf*)	= 21 *psf*	[1.01 *kPa*]
	Floor	= 30 *psf*	[1.44 *kPa*]
	Attic	= 20 *psf*	[0.96 *kPa*]
	Total Live	= 71 *psf*	[3.40 *kPa*]
Dead Loads	*Roof*	= 15 *psf*	[0.72 *kPa*]
	Floor	= 10 *psf*	[0.48 *kPa*]
	Total Dead	= 25 *psf*	[1.20 *kPa*]
	Wall	= 80 *plf*	[1.16 *kPa*]

Factored Load = ((71 *psf*)(1.7) + (25 *psf*)(1.4))(32 *ft*)/(1000)(2)
+ (80 *plf*)(1.4)/(1000)+ (5.5 in)(20 in)(0.150 kcf)(1.4)/(144in^2/ft^2) =2.76 *klf* [40.6 *kN/m*]

Calculate the allowable span. Since the lintel is monolithic with the wall both ends are considered fixed.

$$M = \frac{wl^2}{12} \Rightarrow l = \sqrt{\frac{12M}{w}} = \sqrt{\frac{(12)(10.6 ft - kip)}{2.76 klf}} = 6.78 ft = 6' - 9" \qquad [2.1\ m]$$

B5.4.1.2 Maximum Allowable Span Due to Shear

$V_u = \phi V_n$

$V_n = V_c + V_s$

$V_s = 0$ since there are no stirrups present

$\phi = 0.85$ *(strength reduction factor)*

Calculate the load on the lintel (see calculations in Section B5.4.1.1)

Factored Load = 2.76 *klf* [40.6 *kN/m*]

Determine shear strength of the concrete. The following design recommendations summarized in the report entitled *Testing and Design of Lintels Using Insulating Concrete Forms* [B11] were used in the lintel shear calculations throughout the *Prescriptive Method*.

Flat ICF Lintels

(1) Require a minimum amount of tensile reinforcing steel, $A_{s,min}$, determined by the lesser of:

- $A_{s,min} = 0.2\ \text{in}^2$ [129.0 mm^2]
- $A_{s,\min} = \dfrac{3\sqrt{f_c'}\,b_w d}{f_y} \geq \dfrac{200 b_w d}{f_y}$ ACI Equation 10-3

(2) If the area of tensile reinforcing steel, A_s, is greater than the minimum calculated using ACI 318 Equation 10-3 and the beam has a span-to-depth ratio (l_n/D) of less than 5, calculate the shear capacity using the deep beam provisions, ACI 318 Equation 11-29. Use ACI Equations 11-3 or 11-5 to determine the shear capacity for all other scenarios.

(3) Use shear reinforcement (stirrups) as required by *ACI 318-99*. The study [B11] was unable to draw conclusive recommendations regarding any change to limits for shear reinforcement requirements, particularly when shear reinforcement is necessary in longer span members.

Waffle-Grid ICF Lintels

(1) Require a minimum amount of tensile reinforcing steel, $A_{s,min}$, determined by the lesser of:

- $A_{s,min} = 0.2\ \text{in}^2$ [129.0 mm^2]
- $A_{s,\min} = \dfrac{3\sqrt{f_c'}\,b_w d}{f_y} \geq \dfrac{200 b_w d}{f_y}$ ACI Equation 10-3

(2) If the beam has a span-to-depth ratio (l_n/D) of less than 5, calculate the shear capacity using the deep beam provisions, ACI 318 Equation 11-29, with an effective web thickness of 3.2 in (81.3 mm).

If the beam has a span-to-depth ratio (l_n/D) of greater than 5, calculate the shear capacity with an effective web thickness of 2.6 in (66.0 mm) using ACI 318 Equation 11-3 or an effective web thickness of 2.0 in (50.8 mm) in ACI Equation 11-5.

(3) Use shear reinforcement (stirrups) as required by *ACI 318-99*. The study [B11] was unable to draw conclusive recommendations regarding any change to limits for shear reinforcement requirements, particularly when shear reinforcement is necessary in longer span members.

Screen-Grid ICF Lintels

(1) Require a minimum amount of tensile reinforcing steel, $A_{s,min}$, determined by the lesser of:

- $A_{s,min} = 0.2\ \text{in}^2$ [129.0 mm^2]
- $A_{s,\min} = \dfrac{3\sqrt{f_c'}b_w d}{f_y} \geq \dfrac{200 b_w d}{f_y}$ ACI Equation 10-3

(2) If the beam has a span-to-depth ratio (l_n/D) of less than 5, calculate the shear capacity using the deep beam provisions, ACI 318 Equation 11-29, with an effective web thickness of 2.0 in (81.3 mm) for lintels with depths 24 in (609.6 mm) or greater, and an effective web thickness of 0.9 in (22.9 mm) for lintels with depths less than 24 in (609.6 mm).

If the beam has a span-to-depth ratio (l_n/D) of greater than 5, calculate the shear capacity with an effective web thickness of 2.2 in (55.8 mm) using ACI 318 Equation 11-3 or ACI Equation 11-5.

According to the above design methodology two scenarios should be checked:

Scenario 1: The span-to-depth ratio (l_n/D) is greater than or equal to 5 (not a deep beam according to ACI 318) or the amount of tensile reinforcement ($A_{s,min}$) is less than:

$$A_{s,\min} < MAX\left(\frac{3\sqrt{f_c'}b_w d}{f_y}, \frac{200 b_w d}{f_y}\right) \qquad \text{ACI Equation 10-3}$$

Scenario 2: The span-to-depth ratio (l_n/D) is less than 5 (deep beam according to ACI) and the amount of tensile reinforcement ($A_{s,min}$) is greater than or equal to:

$$A_{s,\min} \geq MAX\left(\frac{3\sqrt{f_c'}b_w d}{f_y}, \frac{200 b_w d}{f_y}\right) \qquad \text{ACI Equation 10-3}$$

If the area of tensile reinforcing steel, A_s, is greater than the minimum calculated using ACI 318 Equation 10-3 and the beam has a span-to-depth ratio (l_n/D) of less than 5 (Scenario 2), calculate the shear capacity using the deep beam provisions, ACI 318 Equation 11-29. Use ACI Equations 11-3 or 11-5 to determine the shear capacity for all other scenarios [B10].

Check the area of steel.

$A_s = 0.2\ in^2$ [1.3 cm^2]

$$\frac{3\sqrt{f'_c}b_w d}{f_y} = \frac{3\sqrt{2{,}500\,psi}(5.5in)(18in)}{40{,}000\,psi} = 0.371 in^2 \qquad [2.4\ cm^2]$$

$$\frac{200 b_w d}{f_y} = \frac{200(5.5in)(18in)}{40{,}000\,psi} = 0.495 in^2 \qquad [3.2\ cm^2]$$

Since $A_{s,min} < 0.495$ in^2 analyze using Scenario 1.

Use ACI Equation 11-3 to determine shear capacity of the section.

$$V_c = 2\sqrt{f'_c}b_w d = 2\sqrt{2{,}500\,psi}(5.5in)(18in) = 9.9 kips \qquad [44.0\ kN]$$

$$\frac{1}{2}\phi V_c = (0.85)(9.9kip)/2 = 4.2kips \qquad [18.8\ kN]$$

Determine the maximum shear force.

$$V_u = \frac{wl}{2} = \frac{(2.76klf)l}{2} = (1.38klf)l$$

Determine the critical shear force.

$$Critical\ V_u = \frac{wl}{2} - wd = (1.38klf)l - (2.76klf)(18in)/(12in/ft) = (1.38klf)l - 4.14k$$

Solve for the span.

$$\frac{1}{2}\phi V_c = Critical\,V_u \Rightarrow 4.2kips = (1.38klf)l - 4.1kips \Rightarrow l = 6.01ft = 6'{-}00'' \qquad [1.83\text{m}]$$

B5.4.1.3 Maximum Allowable Span Due to Deflection

Calculate the load on the lintel.

Live Loads			
	Snow = (0.7)(30 *psf*)	= 21 *psf*	[1.01 *kPa*]
	Floor	= 30 *psf*	[1.44 *kPa*]
	Attic	= 20 *psf*	[0.96 *kPa*]
	Total Live	= 71 *psf*	[3.40 *kPa*]
Dead Loads	*Roof*	= 15 *psf*	[0.72 *kPa*]
	Floor	= 10 *psf*	[0.48 *kPa*]
	Total Dead	= 25 *psf*	[1.20 *kPa*]
	Wall	= 80 *plf*	[1.16 *kPa*]

Unfactored load (used in deflection calculations)

$$=(71\ psf + 25\ psf)(32\ ft)/(2) + (5.5\ in)(20\ in)(150\ pcf)/(144 in^2/ft^2) = 1{,}651 plf \qquad [24.0\ kN/m]$$

Deflection limit of lintel

$$\Delta = \frac{l}{240}$$

Deflection of lintel

$$\Delta = \frac{wl^4}{(0.1)384EI}$$

Calculate moment of inertia, *I.*

$$I = \frac{bh^3}{12} = \frac{5.5(20)^3}{12} = 3{,}667in^4 \qquad [1.52 \times 10^5\ cm^4]$$

Calculate the allowable span.

$$\frac{l}{240} = \frac{wl^4}{(0.1)384EI} \Rightarrow l = \sqrt[3]{\frac{(0.1)384EI}{240w}} =$$

$$= \sqrt[3]{\frac{(0.1)(384)(3{,}122{,}000\,psi)(3{,}667in^4)}{(240)(1{,}651\,plf)(144in^2)}} = 19.75\,ft = 19'\text{–}9''$$

B5.4.1.4 Governing Design

A 5.5-inch x 20-inch (140-mm x 508-mm) flat ICF lintel in a load-bearing wall with one No. 4 or one No. 5 bottom bar may span a maximum of 6’-0” (1.8 m) due to shear limitations with no stirrups required. From Table 5.7 of the *Prescriptive Method* we also obtain a maximum clear span of 6’-0” (1.8 m).

TABLE 5.7
MAXIMUM ALLOWABLE CLEAR SPANS FOR
ICF LINTELS WITHOUT STIRRUPS IN LOAD-BEARING WALLS[1,2,3,4,5,6]
NO. 4 OR NO. 5 BOTTOM BAR SIZE
(excerpt from the *Prescriptive Method*)

Minimum Lintel Thickness, T (inches)	Minimum Lintel Depth, D (inches)	Maximum Clear Span (feet – inches)					
		Supporting Light-Frame Roof Only		Supporting Light-Frame Second Story and Roof		Supporting ICF Second Story and Light-Frame Roof [7]	
		Maximum Ground Snow Load (psf)					
		30	70	30	70	30	70
Flat ICF Lintel							
3.5	8	2-6	2-6	2-6	2-4	2-5	2-2
	12	4-2	4-2	4-1	3-10	3-10	3-7
	16	4-11	4-8	4-6	4-2	4-2	3-10
	20	6-3	5-3	4-11	4-6	4-6	4-3
	24	7-7	6-4	6-0	5-6	5-6	5-2
5.5	8	2-10	2-6	2-6	2-6	2-6	2-6
	12	4-8	3-8	3-4	3-0	3-0	2-9
	16	6-5	5-1	4-8	4-2	4-3	3-10
	20	8-2	6-6	**6-0**	5-4	5-5	5-0
	24	9-8	7-11	7-4	6-6	6-7	6-1

B5.4.2 Waffle-Grid ICF Lintel Design with Stirrups in a Load-Bearing ICF Wall

A waffle-grid ICF lintel is selected from existing Table 5.9B of the *Prescriptive Method* for lintels supporting light-frame roofs and subjected to a 70 psf (3.3 kPa) ground snow load. The lintel's nominal thickness is 6 inches (152.4 mm), with a depth of 20 inches (508 mm). Table 5.9B shows that the lintel has a maximum clear span of 9' – 1" (2.8 m). Calculate the capacity for the 6-inch x 20-inch (152.4-mm x 508 mm) waffle-grid ICF lintel assuming a 32 ft (9.8 m) building width.

B5.4.2.1 Maximum Allowable Span Due to Bending Moment

$$M_u = \phi M_n$$

$$\phi\, M_n = \phi\, A_s f_y \left(d - \frac{a}{2} \right)$$

$\phi = 0.9$ *(strength reduction factor)*

Calculate the depth of the compressive stress block.

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.31 in^2)(40 ksi)}{(0.85)(2.5 ksi)(5 in)} = 1.17 in \qquad [29.7\ mm]$$

Calculate the nominal moment strength.

$$M_n = A_s f_y \left(d - \frac{a}{2}\right) = (0.31 in^2)(40\ ksi)\left(18 in - \frac{1.17 in}{2}\right) = 216 in - kip \qquad [24.4\ mm]$$

Calculate the factored bending moment.

$$M_u = \phi M_n = (0.9)(216 in - kip) / 12 = 16.2\ ft - kip \qquad [22\ kN\text{-}m]$$

Calculate the load on the lintel.

Live Loads			
	Snow = (0.7)(70 *psf*)	= 49 *psf*	[2.3 *kPa*]
	Attic	= 20 *psf*	[0.96 *kPa*]
	Total Live	= 69 *psf*	[3.4 *kPa*]
Dead Loads	*Roof*	= 15 *psf*	[0.72 *kPa*]
	Total Dead	= 15 *psf*	[0.72 *kPa*]

Factored Load = ((69 *psf*)(1.7) + (15 *psf*)(1.4))(32 *ft*)/(1000)(2) + (20 *in*/16 *in*)(0.500 ft^2)(0.150 kcf)(1.4) =2.34 *klf* [34 *kN/m*]

Note: 0.50 sf (0.05 m^2) of concrete fills one linear foot of waffle-grid form.

Calculate the allowable span. Since the lintel is monolithic with the wall both ends are considered fixed.

$$M = \frac{wl^2}{12} \Rightarrow l = \sqrt{\frac{12M}{w}} = \sqrt{\frac{(12)(16.2 ft - kip)}{2.34 klf}} = 9.11 ft = 9'-1"$$

B5.4.2.2 Maximum Allowable Span Due to Deflection

Calculate the load on the lintel.

Live Loads			
	Snow = (0.7)(70 *psf*)	= 49 *psf*	[2.3 *kPa*]
	Attic	= 20 *psf*	[0.96 *kPa*]
	Total Live	= 69 *psf*	[3.4 *kPa*]
Dead Loads	*Roof*	= 15 *psf*	[0.72 *kPa*]
	Total Dead	= 15 *psf*	[0.72 *kPa*]

Unfactored load (used in deflection calculations)

=(69 *psf* + 15 *psf*)(32 *ft*)/(2) + (20 *in*/16 *in*)(0.50 *sf*)(150 *pcf*) = 1,438 *plf* [20.9 *kN/m*]

Deflection limit of lintel.

$$\Delta = l/240$$

Deflection of lintel

$$\Delta = \frac{wl^4}{(0.1)384EI}$$

Calculate moment of inertia, *I*.

Calculate $\bar{y}$ from bottom of lintel

$$\bar{y} = \frac{\sum A_i \bar{y}_i}{\sum A_i} = \frac{(5\,in)(4\,in)(18\,in) + (2\,in)(13\,in)(9.5\,in) + (5\,in)(3\,in)(1.5\,in)}{(5\,in)(4\,in) + (2\,in)(13\,in) + (5\,in)(3\,in)} = 10.32\,in$$

Calculate I .

$$I = \sum\left(\frac{b_i h_i^3}{12} + Ad^2\right) = \left(\begin{array}{l} \dfrac{(5\,in)(4\,in)^3}{12} + (5\,in)(4\,in)(7.7\,in)^2 \\ + \dfrac{(2\,in)(13\,in)^3}{12} + (2\,in)(13\,in)(0.82\,in)^2 \\ + \dfrac{(5\,in)(3\,in)^3}{12} + (5\,in)(3\,in)(8.82\,in)^2 \end{array}\right) = 2{,}768\,in^4 \qquad [1.15 \times 10^5\ cm^4]$$

Calculate the allowable span.

$$\frac{l}{240} = \frac{wl^4}{(0.1)384EI} \Rightarrow l = \sqrt[3]{\frac{(0.1)384EI}{5(240)w}} =$$

$$= \sqrt[3]{\frac{(0.1)(384)(3{,}122{,}000\,psi)\left(2{,}768in^4\right)}{(240)(1{,}438\,plf)(144in^2)}} = 18.8\,ft = 18'\!-\!9''$$

Since $18'\!-\!8'' > 9'\!-\!1''$, *bending moment governs span.*

B5.4.2.3 Increased Span Length for 60ksi Reinforcing Steel

$$M_u = \phi M_n$$
$$\phi\, M_n = \phi\, A_s f_y \left(d - \frac{a}{2} \right)$$
$$\phi = 0.9 \text{ (strength reduction factor)}$$

Calculate the depth of the compressive stress block.

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.31 in^2)(60 ksi)}{(0.85)(2.5 ksi)(5 in)} = 1.75 in \qquad [44.4\ mm]$$

Calculate the nominal moment strength.

$$M_n = A_s f_y \left(d - \frac{a}{2} \right) = (0.31 in^2)(60 ksi)\left(18 in - \frac{1.75 in}{2} \right) = 318.5 in - kip \qquad [36.0\ kN\text{-}m]$$

Calculate the factored bending moment.

$$M_u = \phi M_n = (0.9)(318.5 in - kip)/12 = 23.9 ft - kip \qquad [32.5\ kN\text{-}m]$$

Calculate the load on the lintel.

Live Loads			
	Snow = (0.7)(70 *psf*)	= 49 *psf*	[2.3 *kPa*]
	Attic	= 20 *psf*	[0.96 *kPa*]
	Total Live	= 69 *psf*	[3.4 *kPa*]
Dead Loads	*Roof*	= 15 *psf*	[0.72 *kPa*]
	Total Dead	= 15 *psf*	[0.72 *kPa*]

Factored Load = ((69 *psf*)(1.7) + (15 *psf*)(1.4))(32 *ft*)/(1000)(2)
+ (20 *in*/16 *in*)(0.500 ft^2)(0.150 kcf)(1.4) =2.34 *klf* [34 *kN/m*]

Note: 0.50 sf (0.05 m^2) of concrete fills one linear foot of waffle-grid form.

Calculate the allowable span. Since the lintel is monolithic with the wall both ends are considered fixed.

$$M = \frac{wl^2}{12} \Rightarrow l = \sqrt{\frac{12M}{w}} = \sqrt{\frac{(12)(23.9 ft - kip)}{2.34 klf}} = 11.1 ft = 11'-1"$$

Since 18'–8" > 11'–1", *bending moment governs span.*

Determine percentage increase of 60ksi reinforcing steel vs. 40ksi reinforcing steel.

Percentage Increase $= \frac{11.08 - 9.08}{9.08}(100) = 22.0\% > 20\%$ allowed from Table 5.9B of the *Prescriptive Method.*

B5.4.2.4 Determine Stirrup Requirements

$V_u = \phi V_n$
$V_n = V_c + V_s$
$\phi = 0.85$ *(strength reduction factor)*

Determine the maximum shear force on the lintel.

$$V_u = \frac{wl}{2} = \frac{(2.34klf)(9.08ft)}{2} = 10.6kips \qquad [47.1\ kN]$$

$$Critical\ V_u = V_u - wd = 10.6kips - (2.34k/ft)(18in)/(12in/ft) = 7.09kips \qquad [31.5\ kN]$$

Determine if lintel meets deep beam requirements.

$\frac{l_n}{d} = \frac{(9.08ft)(12)}{18} = 6.05 > 5$ therefore, according to the above design recommendations analyze using ACI Equation 11-3.

Determine if stirrups are required. An effective web width of 2.6 inches (66.0 mm) is suggested according to the above design recommendation for the 6-in waffle-grid [B10].

$$V_c = 2\sqrt{f'_c}b_w d = 2\sqrt{2{,}500psi}(2.6in)(18in) = 4.68kips \qquad [20.8\ kN]$$

$$\frac{1}{2}\phi V_c = (0.85)(4.68kip)/2 = 1.99kips \qquad [8.8\ kN]$$

Since 7.09 kips > 1.99 kips, stirrups are required.

Check required spacing of No. 3 vertical stirrup (A_v = 0.22 in^2 (142 mm^2)).

$$Required\ s = \frac{A_v f_y d}{V_u} = \frac{(0.22in^2)(40ksi)(18in)}{(7.09kip - 4.68kip)} = 65.7in \qquad [1.7\ \text{m}]$$

Maximum allowable spacing is $\frac{d}{2} = \frac{18in}{2} = 9in$

Since 9 in < 65.7 in., the maximum allowable spacing governs

Determine the middle portion of the span, A, where stirrups are not required.

$$A = span - 2\left(\frac{V_u - \frac{1}{2}\phi V_c}{w}\right) = 9.08\,ft - 2\left(\frac{10.6kips - 1.99kips}{2.34kip/ft}\right) = 1.72\,ft = 1\,ft - 8in \qquad [0.5\text{ m}]$$

B5.4.2.5 Governing Design

A 6-inch x 20-inch (152-mm x 508-mm) waffle-grid ICF lintel in a load-bearing wall with one No. 5 Grade 40 bottom bar may span a maximum of 9'-1" (2.8 m) due to bending limitations. However, if a No. 5 Grade 60 bottom bar is used a maximum span of 1.2(9'-1") = 10.9 ft = 10'-8" (3.3 m) is permitted according to the footnote in Table 5.9B of the *Prescriptive Method*. Stirrups are required in the lintel except for the middle portion of the span equaling 1'-8" (0.5 m). Table 5.13 of the Prescriptive Method also requires stirrups except for the middle portion of the span equaling 1'-8" (0.5 m).

TABLE 5.9B
MAXIMUM ALLOWABLE CLEAR SPANS FOR
WAFFLE-GRID ICF LINTELS IN LOAD-BEARING WALLS[1,2,3,4,5,6,7]
NO. 5 BOTTOM BAR SIZE
(excerpt from the *Prescriptive Method*)

Minimum Lintel Thickness, T[8] (inches)	Minimum Lintel Depth, D (inches)	Maximum Clear Span (feet – inches)					
		Supporting Light-Frame Roof Only		Supporting Light-Frame Second Story and Roof		Supporting ICF Second Story and Light-Frame Roof [8]	
		Maximum Ground Snow Load (psf)					
		30	70	30	70	30	70
6	8	5-4	4-8	4-5	4-1	4-5	3-10
	12	8-0	6-9	6-3	5-6	6-3	5-1
	16	9-9	8-0	7-5	6-6	7-5	6-1
	20	11-0	**9-1**	8-5	7-5	8-5	6-11
	24	12-2	10-0	9-3	8-2	9-3	7-8

TABLE 5.13
MIDDLE PORTION OF SPAN (A) WHERE STIRRUPS ARE NOT REQUIRED FOR WAFFLE-GRID ICF LINTELS[1,2,3,4,5,6,7,8]
NO. 4 or NO. 5 BOTTOM BAR SIZE
(excerpt from the *Prescriptive Method*)

Minimum Lintel Thickness, T[9] (inches)	Minimum Lintel Depth, D (inches)	Maximum Center Distance, A (feet – inches)					
		Supporting Light-Frame Roof Only		Supporting Light-Frame Second Story and Roof		Supporting ICF Second Story and Light-Frame Roof [10]	
		Maximum Ground Snow Load (psf)					
		30	70	30	70	30	70
6 or 8	8	0-10	0-7	0-5	0-4	0-5	0-4
	12	1-5	0-11	0-9	0-7	0-8	0-6
	16	1-11	1-4	1-1	0-10	0-11	0-9
	20	2-6	**1-8**	1-5	1-1	1-2	0-11
	24	3-0	2-0	1-9	1-4	1-5	1-2

5.4.3 Screen-Grid ICF Lintel Design in a Load-Bearing ICF Wall

A screen-grid ICF lintel is selected from Table 5.10A of the *Prescriptive Method* for lintels supporting an ICF second story and a light-frame roof and subjected to a 30 psf (1.4 kPa) ground snow load. The lintel's nominal thickness is 6 inches (152 mm), with a depth of 24 inches (610 mm). Table 5.10A shows that the lintel has a maximum clear span of 6'-11" (2.1 m). Calculate the bending capacity and deflection limit for the 6-inch x 24-in (152-mm x 610-mm) screen-grid ICF lintel. Assume a 32 ft (9.8 m) building width.

B5.4.3.1 Maximum Allowable Span Due to Bending Moment

$$M_u = \phi M_n$$

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2} \right)$$

$$\phi = 0.9 \ \textit{(strength reduction factor)}$$

Calculate the depth of the compressive stress block.

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.20in^2)(40ksi)}{(0.85)(2.5ksi)(5.5in)} = 0.68in \qquad [17.3\ mm]$$

Calculate the nominal moment strength.

$$M_n = A_s f_y \left(d - \frac{a}{2} \right) = (0.20in^2)(40ksi)\left(22in - \frac{0.68in}{2} \right) = 173.3in - kip \qquad [19.5\ kN\text{-}m]$$

Calculate the factored bending moment.

$$M_u = \phi M_n = (0.9)(173.3 in - kip)/12 = 13.0 ft - kip \quad [17.7\ kN\text{-}m]$$

Calculate the load on the lintel.

Live Loads			
	Snow = (0.7)(30 *psf*)	= 21 *psf*	[1.00 *kPa*]
	Attic	= 20 *psf*	[0.96 *kPa*]
	Floor	= 30 *psf*	[1.44 *kPa*]
	Total Live	= 71 *psf*	[3.41 *kPa*]
Dead Loads	*Roof*	= 15 *psf*	[0.72 *kPa*]
	Floor	= 10 *psf*	[0.48 *kPa*]
	Total Dead	= 25 *psf*	[1.20 *kPa*]
	Wall	= (53 *psf*)(8 *ft*) = 424 *plf*	[6.2 *kN/m*]

Factored Load = ((71 *psf*)(1.7) + (25 *psf*)(1.4))(32 *ft*)/(1000)(2) + (0.424 klf)(1.4)
+ (24 *in*/12 *in*)(0.053 *ksf*)(1.4) = 3.23 *klf* [47.0 *kN/m*]

Note: 0.053 ksf is the weight of the concrete for screen-grid lintels.

Calculate the allowable span; assume both ends are fixed.

$$M = \frac{wl^2}{12} \Rightarrow l = \sqrt{\frac{12M}{w}} = \sqrt{\frac{(12)(13.0 ft - kip)}{3.23 klf}} = 6.95 ft = 6'\text{–}11" \quad [2.1\ m]$$

B5.4.3.2 Maximum Allowable Span Due to Deflection

Calculate the load on the lintel.

Live Loads			
	Snow = (0.7)(30 *psf*)	= 21 *psf*	[1.00 *kPa*]
	Attic	= 20 *psf*	[0.96 *kPa*]
	Floor	= 30 *psf*	[1.44 *kPa*]
	Total Live	= 71 *psf*	[3.41 *kPa*]
Dead Loads	*Roof*	= 15 *psf*	[0.72 *kPa*]
	Floor	= 10 *psf*	[0.48 *kPa*]
	Total Dead	= 25 *psf*	[1.20 *kPa*]
	Wall	= (53 *psf*)(8 *ft*) = 424 *plf*	[6.2 *kN/m*]

Factored Load = ((71 *psf*)(1.7) + (25 *psf*)(1.4))(32 *ft*)/(1000)(2) + (0.424 klf)(1.4)
+ (24 *in*/12 *in*)(0.053 *ksf*)(1.4) = 3.23 *klf* [47.0 *kN/m*]

Unfactored load (used in deflection calculations)

=(71 *psf* + 25 *psf*)(32 *ft*)/(2) + (424 p*lf*) + (24 *in*/12 *in*)(53 p*sf*) = 2,066 *plf* [6.9 *kN/m*]

Deflection limit of lintel.

$$\Delta = l/240$$

Deflection of lintel.

$$\Delta = \frac{wl^4}{(0.1)384EI}$$

Calculate moment of inertia, *I*.

Calculate $\bar{y}$ from bottom of lintel.

$$\bar{y} = \frac{\sum A_i \bar{y}_i}{\sum A_i} = \frac{(5in)(2.5in)(22.75in) + (5in)(5in)(12.0in) + (5in)(2.5in)(1.25in)}{(5in)(2.5in) + (5in)(5in) + (5in)(2.5in)} = 12.00in \qquad [305\ mm]$$

Calculate I .

$$I = \sum\left(\frac{b_i h_i^3}{12} + Ad^2\right) = \begin{pmatrix} \frac{(5in)(2.5in)^3}{12} + (5in)(2.5in)(10.75in)^2 \\ + \frac{(5in)(5in)^3}{12} \\ + \frac{(5in)(2.5in)^3}{12} + (5in)(2.5in)(10.75in)^2 \end{pmatrix} = 2{,}954.2in^4 \qquad [1.23 \text{ x } 10^5\ mm4]$$

Calculate the allowable span.

$$\frac{l}{240} = \frac{wl^4}{(0.1)384EI} \Rightarrow l = \sqrt[3]{\frac{(0.1)384EI}{(240)w}} =$$

$$= \sqrt[3]{\frac{(0.1)(384)(3{,}122{,}000\,psi)\left(2{,}954.2in^4\right)}{(240)(2{,}066\,plf)(144in^2)}} = 17.0\,ft = 17'\text{–}0''$$

Since $17'\text{–}0'' > 6'\text{–}11''$, *bending moment governs span.*

B5.4.3.3 Increased Span Length for 60ksi Reinforcing Steel

$$M_u = \phi M_n$$

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2}\right)$$

$\phi = 0.9$ *(strength reduction factor)*

Calculate the depth of the compressive stress block.

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.20in^2)(60ksi)}{(0.85)(2.5ksi)(5.5in)} = 1.03in \qquad [26.2\ mm]$$

Calculate the nominal moment strength.

$$M_n = A_s f_y \left(d - \frac{a}{2} \right) = (0.20in^2)(60ksi)\left(22in - \frac{1.03in}{2} \right) = 257.8in - kip \qquad [29.1\ kN\text{-}m]$$

Calculate the factored bending moment.

$$M_u = \phi M_n = (0.9)(257.8in - kip)/12 = 19.3ft - in \qquad [26.2\ kN\text{-}m]$$

Calculate the load on the lintel.

Live Loads			
	Snow = (0.7)(30 *psf*)	= 21 *psf*	[1.00 *kPa*]
	Attic	= 20 *psf*	[0.96 *kPa*]
	Floor	= 30 *psf*	[1.44 *kPa*]
	Total Live	= 71 *psf*	[3.41 *kPa*]
Dead Loads	*Roof*	= 15 *psf*	[0.72 *kPa*]
	Floor	= 10 *psf*	[0.48 *kPa*]
	Total Dead	= 25 *psf*	[1.20 *kPa*]
	Wall	= (53 *psf*)(8 *ft*) = 424 *plf*	[6.2 *kN/m*]

Factored Load = ((71 *psf*)(1.7) + (25 *psf*)(1.4))(32 *ft*)/(1000)(2) + (0.424 klf)(1.4) + (24 *in*/12 *in*)(0.053 *ksf*)(1.4) = 3.23 *klf* [47.0 *kN/m*]

Calculate the allowable span; assume both ends are fixed.

$$M = \frac{wl^2}{12} \Rightarrow l = \sqrt{\frac{12M}{w}} = \sqrt{\frac{(12)(19.3ft - kip)}{3.23klf}} = 8.5ft = 8'\text{–}6''$$

Since $17'\text{–}0'' > 8'\text{–}6''$, *bending moment governs span for 60 ksi steel.*

Determine percentage increase of 60 ksi reinforcing steel vs. 40 ksi reinforcing steel.

Percentage Increase $= \frac{8.50 - 6.92}{6.92}(100) = 22.8\% > 20\%$ allowed from Table 5.10A of the *Prescriptive Method*.

5.4.4 Flat ICF Lintel Design in a Non Load-Bearing ICF Wall Without Stirrups

A flat ICF lintel in a non load-bearing wall is selected from Table 5.14 of the *Prescriptive Method* for lintels supporting an ICF second story. The lintel thickness is 5.5 inches (140 mm), with a depth of 12 inches (305 mm). Table 5.14 shows the lintel to have a maximum clear span of 7 feet (2.1 m). Calculate the capacity and check the adequacy of the 5.5-inch x 12-inch (140-mm x 305-mm) flat concrete lintel in a non load-bearing ICF wall.

B5.4.4.1 Maximum Allowable Span Due to Bending Moment

$$M_u = \phi M_n$$

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2}\right)$$

$$\phi = 0.9 \ \textit{(strength reduction factor)}$$

Calculate the reinforcement ratio for one No. 4 bar horizontal tensile steel.

$$\rho = \frac{A_s}{bd} = \frac{0.2 in^2}{(5.5in)(10in)} = 0.0036$$

$$\rho_b = \frac{0.85 f'_c \beta_1}{f_y}\left(\frac{87{,}000}{f_y + 87{,}000}\right) = \frac{0.85(2{,}500\,psi)(0.85)}{40{,}000\,psi}\left(\frac{87{,}000}{40{,}000\,psi + 87{,}000}\right) = 0.0309$$

$$\rho_{max} = 0.75\rho_b = 0.75(0.0309) = 0.0232$$

$$\rho_{min} = 0.0012$$

$$\textit{Since } \rho_{max} \geq \rho_b \geq \rho_{min} \ \textit{OK}$$

Calculate the depth of the compressive stress block.

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.2\,in^2)(40\,ksi)}{(0.85)(2.5\,ksi)(5.5\,in)} = 0.684\,in \qquad [15.9\ mm]$$

Calculate the nominal moment strength.

$$M_n = A_s f_y\left(d - \frac{a}{2}\right) = (0.20\,in^2)(40\,ksi)\left(10\,in - \frac{0.684\,in}{2}\right) = 77.3\,in - kip \qquad [8.8\ kN\text{-}m]$$

Calculate the factored bending moment.

$$M_u = \phi M_n = (0.9)(77.3\,in - kip)/12 = 5.8\,ft - kip \qquad [7.9\ kN\text{-}m]$$

Calculate the load on the lintel.

Dead Loads	*ICF Wall Above*	= 69 *psf*	[3.3 *kPa*]
	Total Dead	= 69 *psf*	[3.3 *kPa*]

Factored Load = 1.4 ((0.69 *ksf*)(8 *ft*) + (5.5 *in*)(12 *in*)(0.150 *kcf*)/(144 in^2)) = 0.87 *klf* [12.7 *kN/m*]

Calculate the allowable span; assume both ends are fixed.

$$M = \frac{wl^2}{12} \Rightarrow l = \sqrt{\frac{12M}{w}} = \sqrt{\frac{(12)(5.8\ ft-kip)}{0.87\ klf}} = 8.9\ ft = 8'-11'' \qquad [2.7\ m]$$

B5.4.4.2 Maximum Allowable Span Due to Shear

$V_u = \phi V_n$
$V_n = V_c + V_s$
$V_s = 0$ since there are no stirrups present
$\phi = 0.85$ *(strength reduction factor)*

Calculate the load on the lintel (see calculations in Section 5.4.4.1)

Factored Load $= 0.87\ klf$ [12.7 *kN/m*]

Check the area of steel.

$A_s = 0.2\ in^2$ [1.3 cm^2]

$$\frac{3\sqrt{f'_c}b_w d}{f_y} = \frac{3\sqrt{2{,}500\,psi}(5.5in)(10in)}{40{,}000\,psi} = 0.206\,in^2 \qquad [1.3\ cm^2]$$

$$\frac{200 b_w d}{f_y} = \frac{200(5.5in)(10in)}{40{,}000\,psi} = 0.275\,in^2 \qquad [1.8\ cm^2]$$

Since $A_{s,min} < 0.495$ in^2 analyze using Scenario 1 outlined in Section 5.4.1.2 of the *Technical Substantiation*.

Use ACI Equation 11-3 to determine shear capacity of the section.

$$V_c = 2\sqrt{f'_c}b_w d = 2\sqrt{2{,}500\,psi}(5.5in)(10in) = 5.5kips \qquad [24.5\ kN]$$

$$\frac{1}{2}\phi V_C = (0.85)(5.5kip)/2 = 2.34kips \qquad [10.2\ kN]$$

Determine the maximum shear force.

$$V_u = \frac{wl}{2} = \frac{(0.87\,klf)l}{2} = (0.435klf)l$$

Determine the critical shear force.

$$Critical\ V_u = \frac{wl}{2} - wd = (0.435klf)l - (0.87klf)(10in)/(12in/ft) = (0.435klf)l - 0.725k$$

Solve for the span.

$$\frac{1}{2}\phi V_c = Critical\, V_u \Rightarrow 2.34 kips = (0.435 klf)l - 0.725 kips \Rightarrow l = 7.04\, ft = 7'\!-\!00 \qquad [2.1\text{ m}]$$

B5.4.4.3 Maximum Allowable Span Due to Deflection

Calculate the load on the lintel.

Dead Load	*ICF Wall Above*	= 69 *psf*	[3.3 *kPa*]
	Total Dead	= 69 *psf*	[3.3 *kPa*]

Unfactored Load (used in deflection calculations)

$$=(69\, psf)(8\, ft) + (5.5\, in)(12\, in)(150\, pcf)/144\, in^2 = 621\, plf \qquad [9.1\, kN/m]$$

Deflection limit of lintel

$$\Delta = \frac{l}{240}$$

Deflection of lintel

$$\Delta = \frac{wl^4}{(0.1)384EI}$$

Calculate the allowable span.

$$\frac{l}{240} = \frac{wl^4}{(0.1)384EI} \Rightarrow l = \sqrt[3]{\frac{(0.1)384EI}{(240)w}}$$

$$= \sqrt[3]{\frac{(0.1)(384)(3{,}122{,}000\, psi)\left(\frac{1}{12}(5.5in)(12in)^3\right)}{(240)(621\, plf)(144 in^2)}} = 16.4\, ft = 16'\!-\!4" \qquad [5.0\, m]$$

B5.4.4.4 Governing Design

A 5.5-inch x 12-inch (140-mm x 305-mm) flat ICF lintel in a non load-bearing wall supporting an ICF gable end wall with one No. 4 bar may span a maximum of 7'-0" (2.1 m) due to shear and does not require stirrups. From Table 5.14 of the *Prescriptive Method* we obtain a maximum clear span of 7 feet (2.1 m). The values in Table 5.14 have been adjusted to simplify the table for all ICF wall types.

TABLE 5.14
MAXIMUM ALLOWABLE CLEAR SPANS FOR
ICF LINTELS IN GABLE END (NON-LOAD-BEARING) WALLS WITHOUT STIRRUPS[1,2,3]
NO. 4 BOTTOM BAR SIZE
(excerpt from the *Prescriptive Method*)

Minimum Lintel Thickness, T (inches)	Minimum Lintel Depth, D (inches)	Maximum Clear Span: Supporting Light-Frame Gable End Wall (feet)	Maximum Clear Span: Supporting ICF Second Story Gable End Wall[4] (feet)
Flat ICF Lintel			
3.5	8	11-1	3-1
	12	15-11	5-1
	16	16-3	6-11
	20	16-3	8-8
	22	16-3	10-5
5.5	8	16-3	4-4
	12	16-3	**7-0**
	16	16-3	9-7
	20	16-3	12-0
	22	16-3	14-3

B6.0 LEDGER BOARD CONNECTION DESIGN EXAMPLES AND ENGINEERING CALCULATIONS

The following engineering calculations are based on the application of several recognized engineering standards and specifications.

B6.1 Ledger Board-Waffle ICF Connection Design

A wood 1.5-inch x 7.25-inch (38-mm x 184-mm) ledger board is attached to a 6-inch (152-mm) waffle-grid wall. Assume a 4-inch- (102-mm-) diameter hole is cut into the form around each bolt and that the bolt length extends to the center of the ICF wall thickness. Assume a 5/8-inch (19-mm) bolt diameter of A36 steel is used with a 1 3/8-inch- (35-mm-) diameter washer. The floor joists are 2 feet (0.61 m) on center and have a clear span of 22 feet (6.7 m). Wood member and connection design is in accordance with *NDS* [B4].

B6.1.1 Calculate Loads

Nominal Service Load

$$V = Dead\ Load + Live\ Load$$

$$V = (0.5)(22\ ft)((40\ psf + 15\ psf)) = 605\ plf \qquad [8.8\ kN/m]$$

Factored Load

$$V_u = 1.4\ Dead\ Load + 1.7\ Live\ Load$$

$$V_u = (0.5)(22\ ft)(1.4(15\ psf) + 1.7(40\ psf)) = 979\ plf \qquad [14.3\ kN/m]$$

B6.1.2 Determine Maximum Bolt Spacing Due to Shear-Friction in Concrete

$$V_n = A_{bolt} F_y \mu \le \begin{Bmatrix} 0.2 f_c' A_{concrete} \\ 800 A_{concrete} \end{Bmatrix}$$

$$V_n = \frac{\pi (1.375\,in)^2}{4}(36{,}000\ psi)(0.6) = 32{,}074\ lb \qquad [143\ kN]$$

$$V_{n,max} = 0.2(2{,}500\ psi)\left(\frac{\pi(4\,in)^2}{4}\right) = 6{,}283\ lb \quad \longleftarrow \quad GOVERNS \qquad [28\ kN]$$

$$V_{n,max} = 800\left(\frac{\pi(4\,in)^2}{4}\right) = 10{,}053\ lb \qquad [44.7\ kN]$$

$$V_u \le \phi V_n \quad OK$$

$$x = \frac{\phi V_n \mu}{V_u}$$

$$x = \frac{(0.85)(6{,}283\ lb)(12\ in/ft)(0.6)}{(979\ plf)} = 39.3\ in \qquad [998\ mm]$$

B6.1.3 Determine Maximum Bolt Spacing Due to Tension in Concrete (Anchorage Capacity)

$$\phi V_c = \phi\,(4)\left(A_v\right)\sqrt{f_c'}$$

$$V_u \le \phi\, V_c \quad OK$$

$$x = \frac{\phi\, V_c}{0.75\left(\dfrac{V_u}{0.6} + 1.7\left(wind\ load\right)\left(wall\ height\right)\right)}$$

$$x = \frac{\phi\, 4\left(\pi\left(2.5\ in\right)^2\right)\sqrt{2{,}500\ psi}\left(12\, inch\,/\,ft\right)}{0.75\left(\dfrac{979\ plf}{0.6} + 1.7\left(40\ psf\right)\left(10\ ft\right)\right)} = 23.1\, in \qquad [587\ mm]$$

B6.1.4 Determine Maximum Bolt Spacing Due to Tension in Bolt Due to Shear-Friction and Wind Suction Pressure

$$T = 0.75\left(\frac{V_u}{0.6} + 1.7\left(wind\ load\right)\left(wall\ height\right)\right)\left(bolt\ spacing\right)$$

$$f_t = \frac{T}{A_{bolt}}$$

$$f_t \le F_t$$

$$x = \frac{F_t\, A_b\left(12\, inches\,/\,ft\right)}{0.75\left(\dfrac{V_u}{0.6} + 1.7\left(wind\ load\right)\left(wall\ height\right)\right)}$$

$$x = \frac{\left(19{,}100\ psi\right)\left(0.306\ in^2\right)\left(12\, inches\,/\,ft\right)}{0.75\left(\dfrac{979\ plf}{0.6} + 1.7\left(40\ psf\right)\left(10\ ft\right)\right)} = 40.5\ in \qquad [1.03\ m]$$

B6.1.5 Determine Maximum Bolt Spacing Due to Bolted Wood Connection to Concrete

$$Z_{actual} \le Z_{allowable}$$

$$Z_{actual} = V\left(bolt\ spacing\right)$$

$$Z_{allowable} = 520\, lb\,/\,bolt$$

$$x = \frac{\left(12\, inches\,/\,ft\right)\left(520\, lb\,/\,bolt\right)}{605\ plf} = 10.3\, in \qquad [262\ mm]$$

B6.1.6 Determine Maximum Bolt Spacing Due to Bending About Strong Axis in Ledger Board

$$f_b = \frac{M}{S_{xx}}$$

$$M = \frac{PL(bolt\ spacing)}{4}$$

$$f_b \le F_b$$

$$x = \frac{4F_b S_{xx}}{V}$$

$$x = \frac{4(850\ psi)\left(13.14\ in^3\right)(12\ in/ft)}{605\ plf\ (2\ ft\ joist\ spacing)} = 443.1\ in$$ [11.3 *m*]

B6.1.7 Determine Maximum Bolt Spacing Due to Bending About Weak Axis in Ledger Board Due to Wind Suction Pressure

$$f_b = \frac{M}{S_{yy}}$$

$$M = \frac{w(bolt\ spacing)^2}{8}$$

$$f_b \le F_b$$

$$x = \sqrt{\frac{8F_b^{'} S_{yy}}{(wind\ load)(wall\ height)}}$$

$$x = (12\ in/ft)\sqrt{\frac{8(1{,}564\ psi)\left(2.72\ in^3\right)}{(40\ psf)(10\ ft)}} = 110.7\ in$$ [2.8 *m*]

B6.1.8 Determine Maximum Bolt Spacing Due to Allowable Bearing at Washer Due to Weak Axis Bending

$$f_{c\perp} = \frac{T}{A_{washer}}$$

$$M = \frac{w(bolt\ spacing)^2}{8}$$

$$f_{c\perp} \le F_{c\perp}$$

$$x = \frac{F_{c\perp}{}'\left(A_{washer}\right)}{(wind\ load)(wall\ height)}$$

$$A_{washer} = \pi\left(\frac{1.375\ in}{2}\right)^2 - \pi\left(\frac{0.625\ in}{2}\right)^2 = 1.18\ in^2$$ [761 mm^2]

$$x = \frac{(506\ psi)\left(1.18\ in^2\right)(12\ in/ft)}{(40\ psf)(10\ ft)} = 17.9\ in$$ [455 *mm*]

Note: Bolts are required to be staggered or placed in pairs in the top and bottom edges of the ledger to minimize cross–grain tension forces on the ledger. Therefore, cross–grain tension failure modes are addressed in this detailing requirement. In high seismic conditions, additional anchorage is also required.

B6.1.9 Minimum Bolt Spacings and Edge Distance as Defined by *NDS* [B4].

Minimum bolt spacing	$3d_b$	$3(0.625\,in) = 1.9\ inches$	$[48\ mm]$
Minimum edge distance	$4d_b$	$4(0.625\,in) = 2.5\ inches$	$[64\ mm]$
Minimum distance between bolts in a row	$3d_b$	$3(0.625\,in) = 1.9\ inches$	$[48\ mm]$
Minimum end distance	$12\ in$		$[305\ mm]$
Minimum distance between rows of bolts	$3d_b$	$3(0.625\,in) = 1.9\ inches$	$[48\ mm]$

B6.1.10 Governing Design

A wood 1.5-inch x 7.25-inch (38-mm x 183-mm) ledger board attached to a 6-inch (152-mm) waffle-grid wall with 5/8-inch-(19-mm) diameter bolts and 1 3/8-inch (35-mm) washers for floor joists that span 22 feet (6.7 m) require a maximum spacing of 10.3 inches (262 mm) on center as governed by shear in the bolted connection of the wood ledger to concrete. From Table 6.1 of the *Prescriptive Method*, we also obtain a maximum bolt spacing of 10 inches (254 mm) on center for a staggered 5/8-inch-(15.9-mm-) diameter bolted connection or 20 inches (508 mm) on center for a double-bolted connection. Bolt patterns are very important to good practice with wood construction.

TABLE 6.1
FLOOR LEDGER-ICF WALL CONNECTION (SIDE-BEARING CONNECTION) REQUIREMENTS[1,2,3]
(excerpt from the *Prescriptive Method*)

Maximum Floor Clear Span4 (feet)	Maximum Anchor Bolt Spacing[5] (inches)			
	Staggered 1/2-Inch-Diameter Anchor Bolts	Staggered 5/8-Inch-Diameter Anchor Bolts	Two 1/2-Inch-Diameter Anchor Bolts[6]	Two 5/8-Inch-Diameter Anchor Bolts[6]
8	18	20	36	40
10	16	18	32	36
12	14	18	28	36
14	12	16	24	32
16	10	14	20	28
18	9	13	18	26
20	8	11	16	22
22	7	**10**	14	**20**
24	7	9	14	18
26	6	9	12	18
28	6	8	12	16
30	5	8	10	16
32	5	7	10	14

For SI: 1 foot = 0.3048 m; 1 inch = 25.4 mm

B6.2 Additional Requirements for Seismic Design Category C, D_1, and D_2

B6.2.1 Out-of-Plane Anchorage Requirements for Seismic Design Category C, D_1, and D_2

Determine the additional anchorage requirements for connecting the ledgers and joists to a 5.5-in (140-mm) flat ICF wall in Seismic Design Category D_1. Anchorage shall not be accomplished by the use of toe-nails or nails subject to withdrawal nor shall such anchorage mechanisms induce tension stresses perpendicular to grain in ledgers or nailers. The required design value of such anchors are listed in Table 6.2.

The out-of-plane seismic load that must be transferred from the wall to floor connection was determined from *International Building Code* Section 1620.2.1 Equation 16-64 [B7]:

$$F_p = 0.80 I_E S_{DS} w_w \qquad \text{where,}$$

F_p = the out-of-plane seismic force
I_E = occupancy importance factor (Table 1604.5 of the *IBC*)
S_{DS} = the short period site design spectral response acceleration coefficient (Table 301.2.2.1.1 of the IRC)
w_w = the weight of the wall

Therefore, the out-of-plane seismic load induced from the 5.5-in (140-mm) flat ICF wall in Seismic Design Category D_1 is:

$$F_p = 0.80 I_E S_{DS} w_w = 0.80(1.0)(0.83)(5.5in)(150pcf)(11ft)(1ft/12in) = 502plf \qquad [734\ N/m]$$

TABLE 6.2
MINIMUM DESIGN VALUES (plf) FOR FLOOR JOIST-TO-WALL ANCHORS REQUIRED IN SEISMIC DESIGN CATEGORIES C, D_1, AND D_2
(excerpt from the *Prescriptive Method*)

WALL TYPE	SEISMIC DESIGN CATEGORY		
	C	D_1	D_2
Flat 3.5	193	320	450
Flat 5.5	303	**502**	708
Flat 7.5	413	685	965
Flat 9.5	523	867	1,223
Waffle 6	246	409	577

B2.2.2 Top Bearing Anchorage Requirements for Seismic Design Category C, D_1, and D_2

Determine the anchorage requirements for connecting the wood sill plates to a flat ICF wall in Seismic Design Category D_2. According to the IBC, the attachment that the anchor is connecting to the structure shall be designed such that the attachment will undergo ductile yielding at a load level corresponding to anchor forces no greater than the design strength of the anchor specified. In short, this provision prevents concrete breakout as a possible failure mode due to the sudden nature of this type of failure. The required design value of such anchors are the values listed in Table 6.2 divided by 2.

Determine the design shear strength of an A307, 3/8-inch (9.5 mm) diameter anchor bolt embedded 7 inches (178 mm) in the concrete.

Steel strength of the anchor in shear:

$$0.75\phi V_s = 0.75\phi\,0.6(A_{se})(f_{ut}) = (0.75)(0.75)(0.6)(0.078in^2)(60{,}000psi) = 1{,}580lb$$

Concrete breakout strength of anchor in shear:

$$0.75\phi V_{cb} = 0.75\phi\left(\frac{A_v}{A_{v_o}}\right)\Psi_6\Psi_7 V_b \quad \text{where,}$$

$$\Psi_6 = 1.0$$
$$\Psi_7 = 1.0$$

$$V_b = 7\left[\frac{l}{d_o}\right]^{0.2}\sqrt{d_o}\sqrt{f'_c}\,c_1^{1.5} = 7\left[\frac{7in}{0.375in}\right]^{0.2}\sqrt{0.375in}\sqrt{3{,}000psi}\left(\frac{5.5in}{2} - 0.375in\right)^{1.5} = 1{,}543lb$$

$$0.75\phi V_{cb} = (0.75)(0.75)(1.0)(1.0)(1.0)(1{,}543lb) = 868lb$$

Determine the 5-percent offset dowel bearing strength using the general dowel equations for calculating lateral connection values. Upon investigating the seven failure modes, failure mode III_s (Side Member Bearing and Dowel Yielding in the Main Member) perpendicular-to-grain governs the general dowel equation yield mode. The following method outlines the yield mode III_s calculation using a Specific Gravity of 0.42 and 0.70 for the wall framing and the concrete, respectively.

$$P = \frac{-B + \sqrt{B^2 - 4AC}}{2A} \quad \text{where,}$$

$$A = \frac{D^{0.5}}{4D(6{,}100)G_s^{1.45}} + \frac{D^{0.5}}{2D(6{,}100)G_m^{1.45}} = \frac{(0.375in)^{0.5}}{4(0.375in)(6{,}100)(0.42)^{1.45}} + \frac{(0.375in)^{0.5}}{2(0.375in)(6{,}100)(0.70)^{1.45}} = 0.000460$$

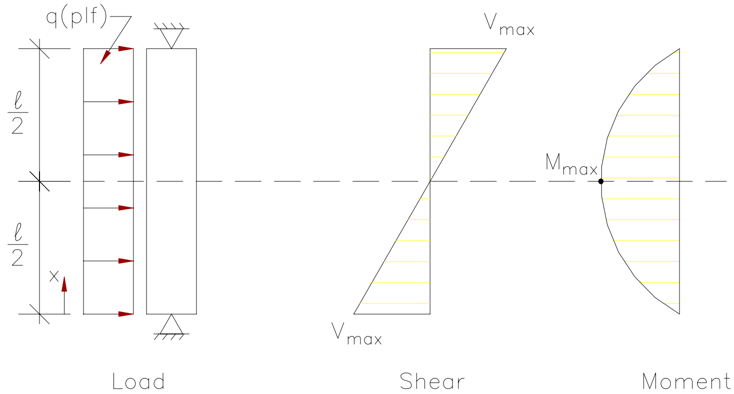
$$B = \frac{l_s}{2} + g = \frac{1.5in}{2} + 0 = 0.75$$

$$C = -\frac{4(6{,}100)(G_s)^{1.45}(D)(l_s)^2}{4(D)^{0.5}} - \frac{(F_b)(D_m)^3}{6} = -\frac{(6{,}100)(0.42)^{1.45}(0.375in)(1.5in)^2}{4(0.375)^{0.5}} - \frac{(45{,}000psi)(0.375in)^3}{6} = -992.7$$

$$P = \frac{-0.75 + \sqrt{0.75^2 - 4(0.000460)(-992.7)}}{2(0.75)} = 865lb$$

Since $865lb < 868lb$, yielding of the attachment governs the design per the requirements in the IBC. Therefore, the spacing required for the 3/8-inch anchor bolt in Seismic Design Category D_2 is equal to $(1{,}223lb/ft)(12in/ft)/(865lb) = 17in$, which is greater than the required 16in spacing.

B7.0 TYPICAL BEAM LOADING CONDITIONS



$$V_{\max} = \frac{ql}{2}$$

$$V_x = q\left(\frac{l}{2} - x\right)$$

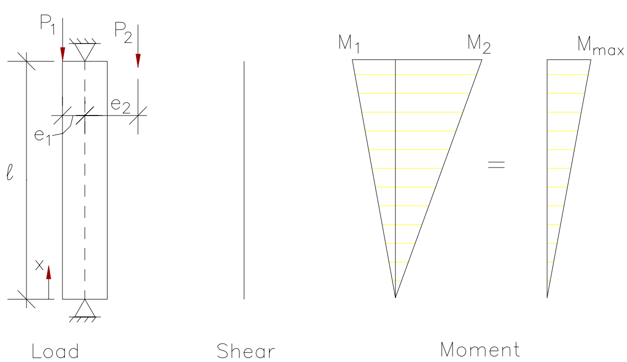
$$M_{\max} = \frac{ql^2}{8}$$

$$M_x = \frac{qx}{2}(l - x)$$

$$\Delta_{\max@center} = \frac{5ql^4}{384EI}$$

$$\Delta_x = \frac{qx}{24EI}\left(l^3 - 2lx^2 + x^3\right)$$

Figure B7.1 Uniform Load, Simple Span



$$V_{\max} = V_x = 0$$

$$M_{\max} = |M_2| - |M_1| \quad where \quad |M_2| > |M_1|$$

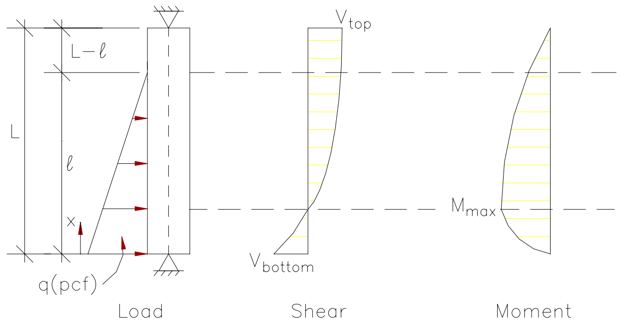
$$M_{\max} = |M_1| - |M_2| \quad where \quad |M_1| > |M_2|$$

$$M_2 = P_2 e_2$$

$$M_1 = P_1 e_1$$

$$M_x = M_{\max}\left(\frac{x}{l}\right)$$

Figure B7.2 Eccentric Point Loads, Simple Span



$$V_{top} = \frac{ql^3}{6L}$$

$$V_{bottom} = \frac{ql^2}{2} - \frac{ql^3}{6L}$$

$$V_x = qlx - \frac{qx^2}{2} - V_{bottom} \quad where \quad x < l$$

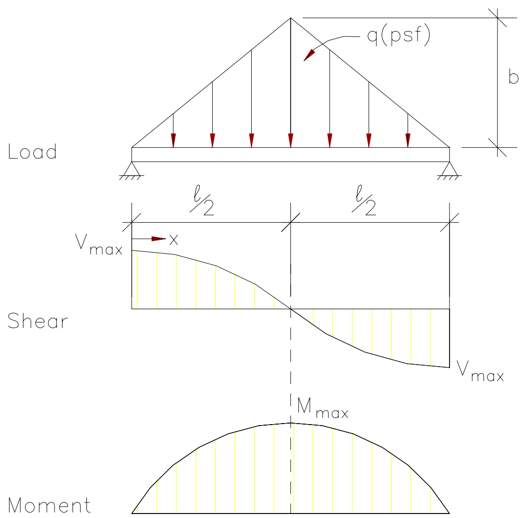
$$V_x = V_{top} \quad where \quad x \geq l$$

$$M_x = -\frac{qlx^2}{2} + \frac{qx^3}{6} + V_{bottom}x \quad where \quad x < l$$

$$M_x = \frac{ql^3}{6L}(L - x) \quad where \quad x \geq l$$

$$x_{@M_{\max}} = \frac{ql - \sqrt{q^2 l^2 - 2qV_{bottom}}}{q}$$

Figure B7.3 Partial Triangular Load, Simple Span



$$V_{\max} = \frac{1}{4}bql$$

$$V_x = \frac{bq}{4l}\left(l^2 - 4x^2\right) \quad where \quad x < \frac{l}{2}$$

$$M_{\max} = \frac{1}{12}bql^2$$

$$M_x = \frac{bqlx}{2}\left(\frac{1}{2} - \frac{2x^2}{3l^2}\right) \quad where \quad x < \frac{l}{2}$$

$$\Delta_{\max\,@\,center} = \frac{bql^4}{120EI}$$

$$\Delta_x = \frac{bqx}{960EI\,l}\left(5l^2 - 4x^2\right)^2 \quad where \quad x < \frac{l}{2}$$

Figure B7.4 Load Uniformly Increasing to Center, Simple Span

B8.0 REFERENCES

[B1] *Minimum Design Loads for Buildings and Other Structures (*ASCE 7-98). American Society of Civil Engineers, New York, New York. 1998.

[B2] *International Residential Code*. International Code Council (ICC), Falls Church, Virginia. 2000.

[B3] *Building Code Requirements for Structural Concrete* (ACI 318-99). American Concrete Institute, Detroit, Michigan. 1999.

[B4] *National Design Specification for Wood Construction*. The American Forest and Paper Association. Washington, DC. 1997.

[B5] *Manual of Steel Construction Allowable Stress Design, Ninth Edition*. American Institute of Steel Construction. Chicago, Illinois. 1989.

[B6] *Design Criteria for Insulating Concrete Form Wall Systems,* (RP116). Prepared for the Portland Cement Association by Construction Technology Laboratories, Inc., Skokie, Illinois. 1996.

[B7] *International Building Code.* International Code Council (ICC). Falls Church, Virginia. 2000.

[B8] *Structural Design of Insulating Concrete Form Walls in Residential Construction.* Portland Cement Association, Skokie, Illinois. 1998.

[B9] *In-Plane Shear Resistance of Insulating Concrete Form Walls.* Prepared for the U.S. Department of Housing and Urban Development, Portland Cement Association, and the National Association of Home Builders by the NAHB Research Center, Inc., Upper Marlboro, Maryland. 2001.

[B10] *Lintel Testing for Reduced Shear Reinforcement in Insulating Concrete Form Systems.* Prepared for the U.S. Department of Housing and Urban Development, Portland Cement Association, and the National Association of Home Builders by the NAHB Research Center, Inc., Upper Marlboro, Maryland. 1998.

[B11] *Testing and Design of Lintels Using Insulating Concrete Forms.* Prepared for the U.S. Department of Housing and Urban Development, Portland Cement Association, and the National Association of Home Builders by the NAHB Research Center, Inc., Upper Marlboro, Maryland. 2000.

APPENDIX C

METRIC CONVERSION FACTORS

The following list provides the conversion relationship between U.S. customary units and the International System (SI) units. A complete guide to the SI system and its use can be found in ASTM E 380, *Metric Practice*.

Length

To convert from	to	Multiply by
inch (in)	meter (m)	2.540 000 E-02
foot (ft)	meter (m)	3.048 000 E-01
yard (yd)	meter (m)	9.144 000 E-01
mile (mi)	meter (m)	1.609 344 E+03

Area

To convert from	to	Multiply by
square foot (ft^2 or sf)	square meter (m^2)	9.290 304 E-02
square inch (in^2)	square meter (m^2)	6.451 600 E-04
square yard (yd^2)	square meter (m^2)	8.361 274 E-01
square mile (mi^2)	square meter (m^2)	2.589 988 E+06

Force per Unit Area (stress or pressure)

To convert from	to	Multiply by
kip per sq. inch (ksi)	Pascal (Pa)	6.894 757 E+06
pound per sq. foot (psf)	Pascal (Pa)	4.788 026 E+01

One Pascal equals 1,000 Newton per square meter.
One kip equals 1,000 pound.

Volume

To convert from	to	Multiply by
cubic inch (in^3)	cubic meter (m^3)	1.638 706 E-05
cubic foot (ft^3)	cubic meter (m^3)	2.831 685 E-02
cubic yard (yd^3)	cubic meter (m^3)	7.645 549 E-01
gallon (gal) Can. liquid	cubic meter (m^3)	4.546 090 E-03
gallon (gal) U.S. liquid	cubic meter (m^3)	3.785 412 E-03
fluid ounce (fl. oz.) U.S. liquid	cubic meter (m^3)	2.957 353 E-05

One U.S. gallon equals 0.8327 Canadian gallon.
One liter equals 0.001 cubic meter.

Force

To convert from	to	Multiply by
kip (1,000 lb)	Newton (N)	4.448 222 E+03
pound (lb)	Newton (N)	4.448 222 E+00
ton (2,000 lb)	Newton (N)	8.896 444 E+03

Force per Unit Length

To convert from	to	Multiply by
kip per linear foot (plf)	Newton per meter (N/m)	1.459 390 E-02
pound per linear foot (plf)	Newton per meter (N/m)	1.459 390 E+01

Mass

To convert from	to	Multiply by
pound (lb), avoirdupois	kilogram (kg)	4.535 924 E-01
ton (2,000 lb)	kilogram (kg)	9.071 847 E+02
slug	kilogram (kg)	1.459 390 E+01

Mass per Unit Length

To convert from	to	Multiply by
kip per linear foot (plf)	kilogram per meter	1.488 164 E-03
pound per linear foot (plf)	kilogram per meter	1.488 164 E+00

Moment

To convert from	to	Multiply by
foot-pound (ft-lb)	Newton-meter (N-m)	1.355 818 E+06

Mass per Unit Volume (Density)

To convert from	to	Multiply by
pound per cubic foot (pcf)	kilogram per cubic meter (kg/m^3)	1.601 846 E+01
pound per cubic yard (lb/yd^3)	kilogram per cubic meter (kg/m^3)	5.932 764 E-01

Velocity

To convert from	to	Multiply by
miles per hour (mph)	kilometer per hour (km/hr)	1.609 344 E+00
miles per hour (mph)	meter per second (m/s)	4.470 400 E-01

Temperature

To convert from	to	Equation
degrees Fahrenheit (^oF)	degrees Celsius (^oC)	$T_c=(T_f-32)/1.8$
degrees Fahrenheit (^oF)	Kelvin (K)	$T_K=(T_f+459.67)/1.8$
Kelvin (K)	degrees Celsius (^oC)	$T_c=(T_K-273.15)$

The prefixes and symbols below are commonly used to form names and symbols of the decimal multiples of the SI units.

Multiplication Factor	Prefix	Symbol	Multiplication Factor	Prefix	Symbol
$1,000,000,000 = 10^9$	giga	G	$0.01 = 10^{-2}$	centi	c
$1,000,000 = 10^6$	mega	M	$0.001 = 10^{-3}$	milli	m
$1,000 = 10^3$	kilo	k	$0.000001 = 10^{-6}$	micro	μ
			$0.000000001 = 10^{-9}$	nano	n

Reinforcement Bar Data

Inch-Pound	Metric
No. 3	No. 10
No. 4	No. 13
No. 5	No. 16
No. 6	No. 19
No. 7	No. 22
No. 8	No. 25
Grade 40	Grade 300
Grade 60	Grade 420