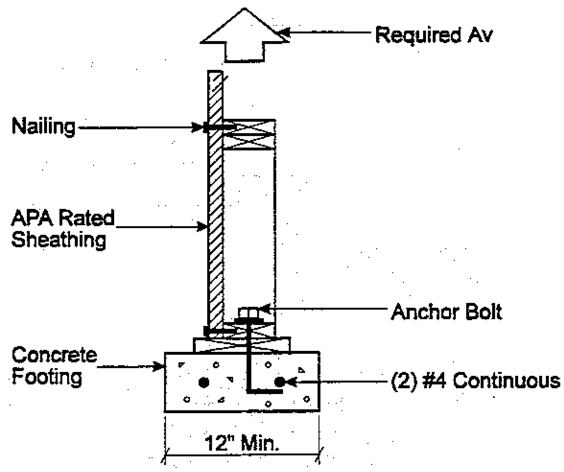
Table C-4B
Vertical Anchor Capacity For Longitudinal Foundation Wall[1, 2]
(In pounds per linear foot of wall)



Treated Wood Wall

Vertical Capacity lbs./ft.	Required Nailing[4, 5] (Edge Spacing, in.)	Min. Plywood Thickness	Required Anchorage[2, 3]	
Standard Washer			Anchor Bolt Diameter	Bolt Spacing[6]
146	6d @ 6" o.c.	3/8"	1/2"	6'-0"max.
164	↓	↓	↓	5'-4"
187	↓	↓	↓	4'-8"
218	8d @ 6" o.c.	↓	↓	4'-0"
262	8d @ 4" o.c.	↓	↓	3'-4"
327	8d @ 4" o.c.	15/32"	↓	2'-8"
437	10d @ 2 1/2" o.c.	↓	↓	2'-0"
***				

*** For required Av greater than 437 lbs./ft., consider using a different foundation material or utilize an engineered design with a higher capacity.

1 Compare with required Av for Type E units.

2 In the case of a treated wood foundation wall, the wood wall and its connections must be designed to transfer the anchor load to a concrete footing. This table does not apply to treated wood foundation walls on gravel bases.

3 Values are based on vertical capacity per foot of wall.

4 Assuming 1 1/2" thick sill plate, 3/4" edge distance for wood or composite nailer plates or 20 diameter end distance for plywood sheathing; APA rated, properly seasoned wood; Group III woods, not permanently loaded, and a 25% length of bearing factor increase.

5 Nailing schedule in this table is intended to secure the superstructure to the foundation only, and not to provide required edge fastening for plywood siding or sheathing.

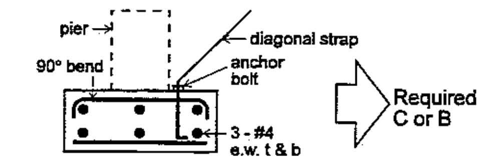
6 Spacing and capacity is based on allowable compression of wood perpendicular to grain for $F_c = 565$ psi and standard washer = 1 3/8" O.D. and 9/16" I.D. washer (for 1/2" ϕ bolt).

Table C-5A
Horizontal Anchor Capacity For Transverse or Longitudinal Shear Walls[1]
(In pounds per foot of wall)

Concrete or Masonry

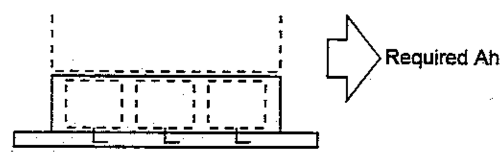
Horizontal Capacity[2] lbs./ft.	Required Anchorage[5]		
	Anchor Bolt[4]	Rebar	Spacing[6]
300	1/2"	#4	72" o.c. max.
600			36" o.c.
675			32" o.c.
900			24" o.c.
1350			16" o.c.
1800			12" o.c.
***			
See Table C-3A For Rebar Details			

*** For required Ah greater than 1800 lbs./ft., consider using an engineered design with a higher capacity.



Pier Footing Used With X-Bracing

Concrete or Masonry



Treated Wood

Table C-5B

Treated Wood

Horizontal Capacity[2] lbs./ft.	Required Nailing[3, 4] (Edge Spacing, in.)	Min. Plywood[4] Nailer Thickness	Required Anchorage	
			Anchor Bolt Diameter	Bolt Spacing[7]
300	8d @ 4" o.c.	7/16"	1/2"	4'-0" max.
360	8d @ 4" o.c.	15/32"		3'-4"
449	10d @ 4" o.c.	15/32"		2'-8"
600	10d @ 3" o.c.	19/32"		2'-0"

1 Compare capacity with required Ah in transverse or longitudinal direction.
2 Values are based on horizontal load per foot of wall. Select Ah for pier spacing of 4 feet for use with this table.
3 Assuming 1 1/2" thick sill plate, 3/4" edge distance for wood or composite nailer plates or 20 diameter end distance for plywood sheathing; APA rated, properly seasoned wood; Group III woods, not permanently loaded.
4 Nailing schedule in this table is intended to secure the superstructure to the foundation only, and not to provide required edge fastening for plywood siding or sheathing.
5 It is assumed that a reinforcing bar of the same diameter as the anchor is adequately embedded in the footing and lapped with the anchor. In the case of a treated wood foundation wall, the wood wall and its connections must be designed to transfer the anchor load to a concrete footing. This table does not apply to treated wood foundation walls on gravel bases.
6 Spacing based on bearing capacity of bolt against concrete/grout.
7 Spacing based on capacity of anchor bolt in bearing against the wood plate. (see also #5.)

APPENDIX D
DERIVATION OF FOUNDATION DESIGN

D-100. CONDITIONS AFFECTING DESIGN. Values for the Foundation Design Load Tables have been derived based on major foundation design factors, foundation design criteria, and design assumptions.

D-100.1 MAJOR FOUNDATION DESIGN FACTORS determine the appropriateness of foundations for manufactured homes:

A. Soil and site conditions.

1. Soil types
2. Bearing capacities
3. Drainage
4. Slopes

B. Load Conditions and Combinations. Various combinations of (1) through (5) with appropriate factors:

1. Dead loads
2. Occupancy live loads
3. Wind loads
4. Snow loads / Minimum roof live loads
5. Seismic loads

C. Foundation Design and Capacity.

1. Footing depth
2. Footing size
3. Reinforcing
4. Materials

D. Connection Compatibility with Manufactured Home. Adequate capacity plus a safety factor is required to transfer forces from the manufactured house to the foundation without failure.

D-100.2 CRITERIA FOR FOUNDATION DESIGN for manufactured homes must meet the following:

A. Assumptions made in foundation system design must be compatible with the design of the housing unit and actual site conditions.

B. Stress Limitations. The design must sustain all loads within stress limitations of connection systems.

C. Acceptable Foundation Design must provide for the Permanent Foundation criteria as specified in Section 100-1.C.

D-100.3 DESIGN ASSUMPTIONS

A. Values Included In Appendix B & C. The foundation tables in Appendices B & C are based on a number of design assumptions:

1. Building width is discussed in terms of minimum chassis beam spacing in Chapter 1: 100-1.A.5 and again in Chapter 6: 600-2.A.1. for comparison of nominal and range of actual width, and then is illustrated in Figure 6-1. It is clear that many actual widths are possible. The following actual widths and projections (dc) were used in the Tables of Appendix B:

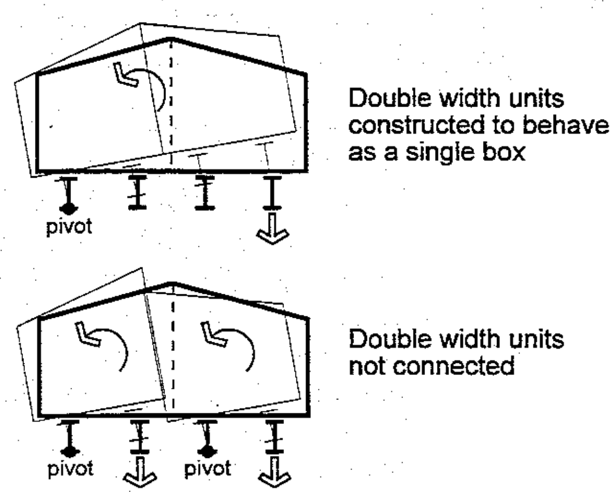
Wt (nominal)	Wt (actual)	dc
12'	11'-8"(11.67')	32.25"(2.69')
14'	13'-8"(13.67')	41"(3.42')
16'	15'-6"(15.5')	45.25"(3.77')

2. The Overturning (Av) and Sliding (Ah) Tables in Appendix B assume h_n=8.0 feet and assume a chassis beam depth of 10" (0.833 ft).

3. The manufactured home is located on a flat, open site with no protection from the wind.

4. Wind force on the manufactured home, instead of seismic force, is the controlling factor for the foundation overturning anchorage design in the transverse direction. Seismic forces or wind force may control sliding anchorage in the transverse or longitudinal direction.

5. Uplift, overturning, and sliding caused by wind or seismic forces acting on the manufactured home are transferred to the foundation by the structural integrity of the manufactured home.



Marriage Wall Connection Options

Figure D - 1

6. The manufactured home unit, single or multi-width, is assumed to be a box with flexible floor and roof diaphragms. End walls and selected interior shear walls were assumed to transfer lateral forces based on tributary area methodology. The unit's shear wall locations must closely coincide with the foundation shear walls or vertical X-bracing planes. A structural engineer shall design the system if deviations from these assumptions exist.

7. Multi-section units are assumed to be connected at the marriage wall to act as a single box for overturning consideration, and do not act separately as illustrated in Figure D-1. This is particularly necessary in high seismic locations.

B. List of Variables. These variables are used throughout Appendix D.

<u>Aa</u> Seismic coefficient representing effective peak acceleration

Ah Required horizontal anchorage (lbs. or lbs./LF)

Av Required vertical anchorage (lbs. or lbs./LF)

<u>Av</u> Seismic coefficient representing effective peak velocity related acceleration

Ce Exposure factor (See ASCE 7-93)

Ct Thermal factor (See ASCE 7-93)

Cp External wall or roof pressure coefficient (See ASCE 7-93)

Cs Roof slope factor (See ASCE 7-93)

<u>Cs</u> Seismic design coefficient (See ASCE 7-93)

dc Distance from perimeter of structure to chassis beam line.

DL Total dead load of structure for each foot of length

Fr Force resisting sliding

Fsl Sliding force (lbs.)

GCpi Internal wall or ceiling pressure coefficient (See ASCE 7-93)

Gh Gust response factor (See ASCE 7-93)

hn Height of the exterior wall acted on by lateral wind pressure

I Importance factor (See ASCE 7-93)

Kz Velocity pressure exposure coefficient (See ASCE 7-93)

LL Live load

Mo Overturning moment of structure

Mr Moment resisting overturning

p Design wind pressure

Pf Design roof snow load (See ASCE 7-93)

Pg Ground snow load (See ASCE 7-93)

Sp or Spacing: Spacing of foundation elements in the longitudinal direction.

V Basic wind speed (See ASCE 7-93)

Wt Width of structure (or 1/2 the total width of a multi-section unit)

D-200. LOAD CONDITIONS INCLUDED IN FOUNDATION DESIGN. The following load conditions have been used as assumptions in design of the foundation systems in this handbook. This information is important for engineers who may be designing connection details or modifying foundations designs. All Design Loads are based on ASCE 7-93, except as noted otherwise.

D-200.1 DEAD LOAD DESIGN FACTORS. Dead loads consist of the material weight of the manufactured home without furnishings or occupants. Dead load includes the weight of the roof, floor, walls, and chassis, and may include permanent attachments such as cabinets and attached appliances.

A. Dead Load Categories. Dead loads were grouped into two categories: heavy and light. The heaviest combinations of dead loads were used for the computation of footing areas, and the determination of inertia forces for the computation of sliding and overturning due to seismic activity. Heavier loads generate the largest inertia forces and produce the largest footings. The lightest combinations of dead loads were used for the computation of horizontal and vertical anchorage due to wind. Lighter loads offer less resistance to overturning and sliding and thus require greater anchorage. The following dead loads in Table D-1 have been included in the calculations for the

Foundation Design Load Tables on the next page.

B. Dead Load Equations for use in computing the required vertical and horizontal anchorage to resist overturning and sliding are listed below by type. The equations are for the total Dead Load per foot of Manufactured Home length. Figure D-2 illustrates the individual component loads and the total dead load situated at the geometric centroid of the unit.

Lightest combination of loads:

SINGLE-SECTION TYPES C, E, & I

$DL = (34.5)2 + (6 + 8.6)Wt + 9 \times 2$
(walls)+(floor+roof)+(chassis beams)

$\boxed{DL = 87 + (14.6)Wt}$

MULTI-SECTION TYPES C, E, & I

$DL = (34.5)2 + (26.25)2 + 2(6 + 8.6)Wt + 9 \times 4$
(ext. walls) + (marriage wall) + (floor + roof) + (chassis beams)

$\boxed{DL = 157.5 + (29.2)Wt}$

Heaviest combination of loads:

SINGLE-SECTION TYPES C, E, & I

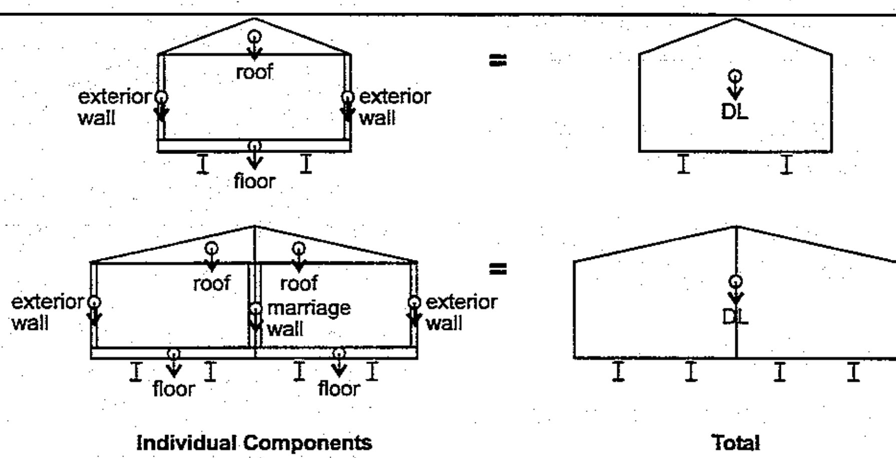
$DL = (44.25)2 + (13 + 9.7)Wt + 9 \times 2$
(walls) + (floor + roof) + (chassis beams)

$\boxed{DL = 106.5 + (22.7)Wt}$

MULTI-SECTION TYPES C, E, & I

$DL = (44.25)2 + (26.25)2 + 2(13 + 9.7)Wt + 9 \times 4$
(ext. walls) + (marriage wall) + (floor + roof) + (chassis beams)

$\boxed{DL = 177 + (45.4)Wt}$



Dead Load Components and Total

Figure D - 2

TABLE D-1
DEAD LOAD ON FOUNDATION

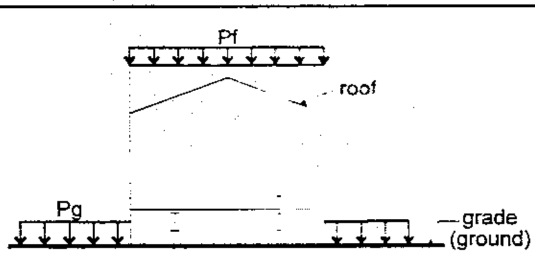
LOCATION	ITEM	HEAVY (psf)	LIGHT (psf)	HEAVY (plf of length)	LIGHT (plf of length)
EXTERIOR WALL	7/16" siding	1.4			
	.019 aluminum		0.1		
	2 x 4 studs @ 16"o.c.	1.5	1.5		
	3 1/2" fiberglass insulation	1.0	1.0		
	1/2" gypsum	2.0	2.0		
	SUM =	**5.9**	**4.6**		
TOTAL	**7'-6" WALL**			**44.25**	**34.5**
FLOOR	carpet & pad	1.0			
	1/16" vinyl		0.7		
	5/8" plywood	1.7	1.7		
	2 x 10 joist @ 16"o.c.	2.6			
	2 x 6 joist @ 16"o.c.		1.4		
	11" fiberglass insulation	2.2			
	5 1/2" fiberglass insulation		1.2		
	mechanical	2.0	1.0		
	misc. partitions	3.5	0.0		
	SUM =	**13.0 ***	**6.0 ***	**$13 \times Wt + 9$**	**$6 \times Wt + 9$**
	* plus 9 plf for each manufactured home beam				
ROOF	asphalt shingles with felt	2.5			
	3/8" plywood	1.1			
	20 ga. steel		2.5		
	2 x 3 truss	1.5	1.5		
	9 1/2" fiberglass insulation	2.6	2.6		
	1/2" gypsum ceiling	2.0	2.0		
	SUM =	**9.7**	**8.6**	**$9.7 \times Wt$**	**$8.6 \times Wt$**
MARRIAGE WALL	2x4 studs @ 16"	1.5	1.5		
	1/2" gypsum (one side)	2.0	2.0		
	SUM =	**3.5**	**3.5**		
TOTAL	**7'-6" WALL**			**26.25**	**26.25**

D-200.2 LIVE LOAD DESIGN FACTORS

A. Description. Design live loads consist of the weight of all moving and variable loads (from use and occupancy) that may act on the manufactured home including loads on floors, operational loads on roofs and ceilings, or snow loads, but do not include wind, earthquake or dead loads. All live loads are assumed to be uniformly distributed and roof live loads are horizontally projected on sloped surfaces. The design live loads specified herein for the floor and attic are the minimums recommended by the ASCE standard. The design live loads specified herein for the roof are the minimum recommended by the *Minimum Property Standard*, HUD Handbook 4910.1, 1994 Edition. The roof live load used for the design of the foundation system should be the greater of the appropriate value indicated in the Data Plate shown here or as obtained from the ASCE 7-93 for snow load.

B. Design Assumptions. The following values for live loads were used in the engineering calculations and are included in the tables. They are provided here as background information only. The field inspector will not need to calculate live loads under normal circumstances. See box of live loads.

D-200.3 SNOW LOAD DESIGN FACTORS



Snow Load Distribution

Figure D - 3

Minimum Uniformly Distributed Live Loads
(used for Foundation Design Load Tables)

Location	Live Load (psf)
Roof (slope 3/12 or less, ≤14°)	20*
Roof (slope over 3/12, >14°)	15*
(Over the entire width of the unit. Compare with snow load value. Use the larger value.)	
Dwelling rooms	40
(Floor design live loads over the entire area of the unit.)	
Attics	10
(uninhabitable, without storage)	

* Due to snow load factors, the 30 psf ground snow load used on the Foundation Design Load Tables is equivalent to a 20 psf roof live load. The 20 psf ground snow load is equivalent to a 15 psf roof live load.

A. Ground Snow Load. The ground snow load values (Pg) to be used in the design of the manufactured home are found in Appendix H. The ground snow load is converted to a roof snow load to account for wind and thermal factors (see Figure D-3). The value (Pg) modified by snow load design factors has been included in the derived values for the Foundation Design Load Tables. The following assumptions were made to find Pf, the horizontally projected uniformly distributed design roof snow load:

B. Design Assumptions.

Basic Snow Load Equation:
$Pf = 0.7 \times Ce \times Ct \times I \times Pg$

Where:

1. Ground snow load (Pg) from the Ground Snow Load maps on pages H-11, H-12 and H-13.

2. Importance factor I = 1.0 (residential buildings)

3. Exposure factor Ce = 1.0 (locations where snow removal cannot be relied on to reduce snow loads)

4. Thermal factor Ct = 1.0 (heated structures)

5. Slope factor Cs = 1.0 (4/12 slope or less)

6. Flat roof factor = 0.7 (contiguous U.S.; Use 0.6 in Alaska.)

Therefore, the Required Effective Footing Area Tables are based on:

$Pf = 0.7 \times Pg$ (Roof snow load)

C. Drifted Snow. At locations where the manufactured home is adjacent to a higher structure, drifted snow loads MUST be calculated in accordance with ASCE 7-93. An average value including the drifted load may be used with the Foundation Design Load Tables.

D-200.4 WIND LOAD DESIGN FACTORS.

A. Model for Analysis. The methodology for resistance of the box to uplift, overturning and sliding utilizes equations for Main Wind-Force Resisting Systems as defined in ASCE 7-93.

B. Basic Wind Speed. The basic wind speed map is found on page H-14. Wind factors have been included in the derived values for the Foundation Design Load Tables of Appendix B.

C. Design Assumptions.

1. To convert mile per hour (MPH) wind speed to a basic wind velocity pressure (q) in pounds per square feet (psf) use the following equation from ASCE 7-93:

$$q = 0.00256 \times Kz \times (V \times I)^2$$

where:

a. Mean roof height is assumed to be less than or equal to 15 feet from grade.

b. Basic Wind Speed (V) is from the isobar map on page H-14 for the unit's geographic location.

c. Velocity Pressure Coefficient (Kz) is based on Exposure C: open terrain with scattered obstructions having heights generally less than 30 feet. This Category includes flat open country and grasslands. For these conditions, including item (a) above, Kz = 0.8.

d. Importance Category I (residential) for inland sites, sets I = 1.0, while for coastal sites (hurricane oceanline) I = 1.05. Linear interpolation can be utilized for sites between the oceanline and 100 miles inland; however, this was not done for the tables of Appendix B. Thus, only the above two values have been included.

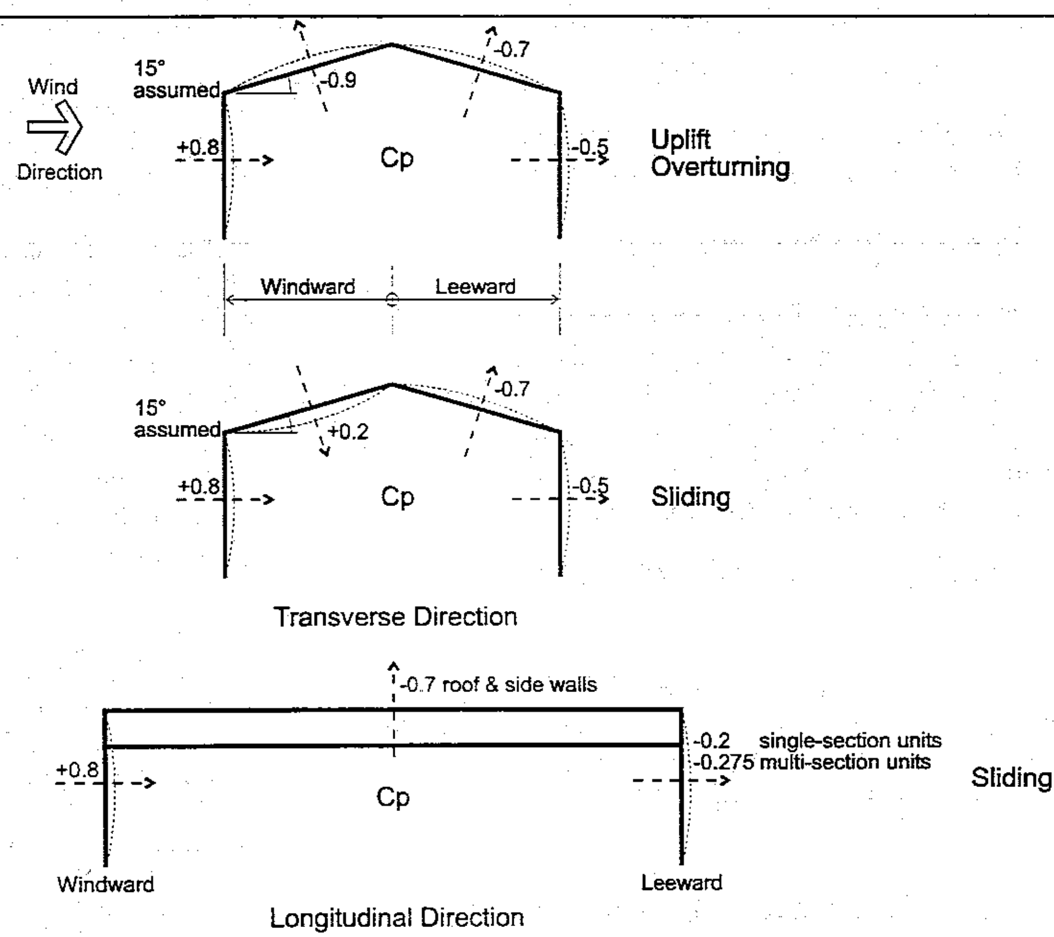
2. Velocity pressure (q) is applied to surfaces, i.e. walls and roof planes, to generate design wind pressures (p) for Main Wind-Force Resisting Systems. Design wind pressures (p) are based on external and internal effects utilizing the following equation from ASCE 7-93:

$$p = q \times Gh \times Cp - q \times (\pm GCpi)$$
(external) - (internal)

where:

a. The Gust Response Factor (Gh) is assumed to be based on Exposure C (see section D-200.4.C.1.c). The Minimum Property Standard (MPS) permits use of Exposure C regardless of whether the site is inland or coastal. Thus, for units of assumed mean height less than or equal to 15 feet, Gh = 1.32.

b. External Roof and Wall Pressure Coefficients (Cp) vary on the windward roof surface based on the structural issue being analyzed. Figure D-4 illustrates the various (Cp) values for the transverse and longitudinal di-

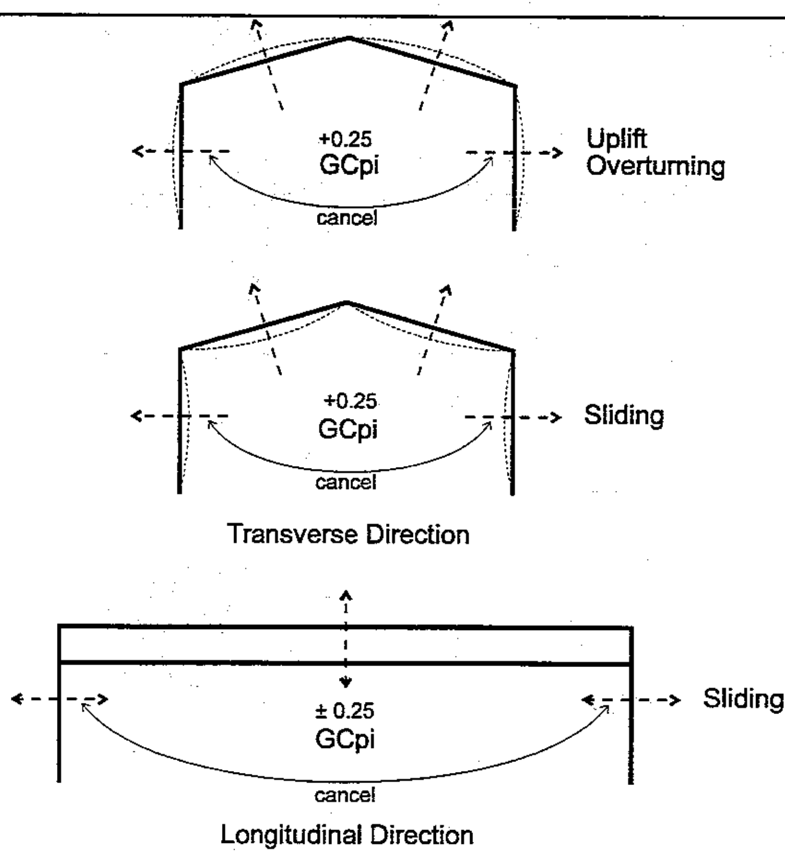


Exterior (Cp) Coefficients

Figure D - 4

rections. A roof slope of 10 to 15 degrees (2 in 12 to 3 in 12) produces 2 possible situations: (+0.2) pressure and (-0.9) suction. The value (-0.9) was selected to produce maximum suction for uplift and overturning while (+0.2) was selected to maximize sliding. Note that (+) means pressure on the external surface, while (-) means suction on the external surface. For the leeward wall in the longitudinal direction the proportions of the unit (L/Wt) are important to establishing the proper exterior (Cp) value. Single-section units, regardless of the combination of width or length, has a ratio L/Wt ≥ 4.0; therefore, Cp = -0.2. For multi-section units An average proportion of unit (28' x 70', or 32' x 80') was assumed. Thus, the L/Wt ratio was 2.5 and by interpolation Cp = -0.275. Single or multi-section units have a Wt/L ratio, which is ≤ 1.0 for all proportions of units. Thus, the leeward value for Cp = -0.5 in the transverse direction.

c. Internal Roof and Wall Pressure Coefficients assume a uniform distribution of openings on all

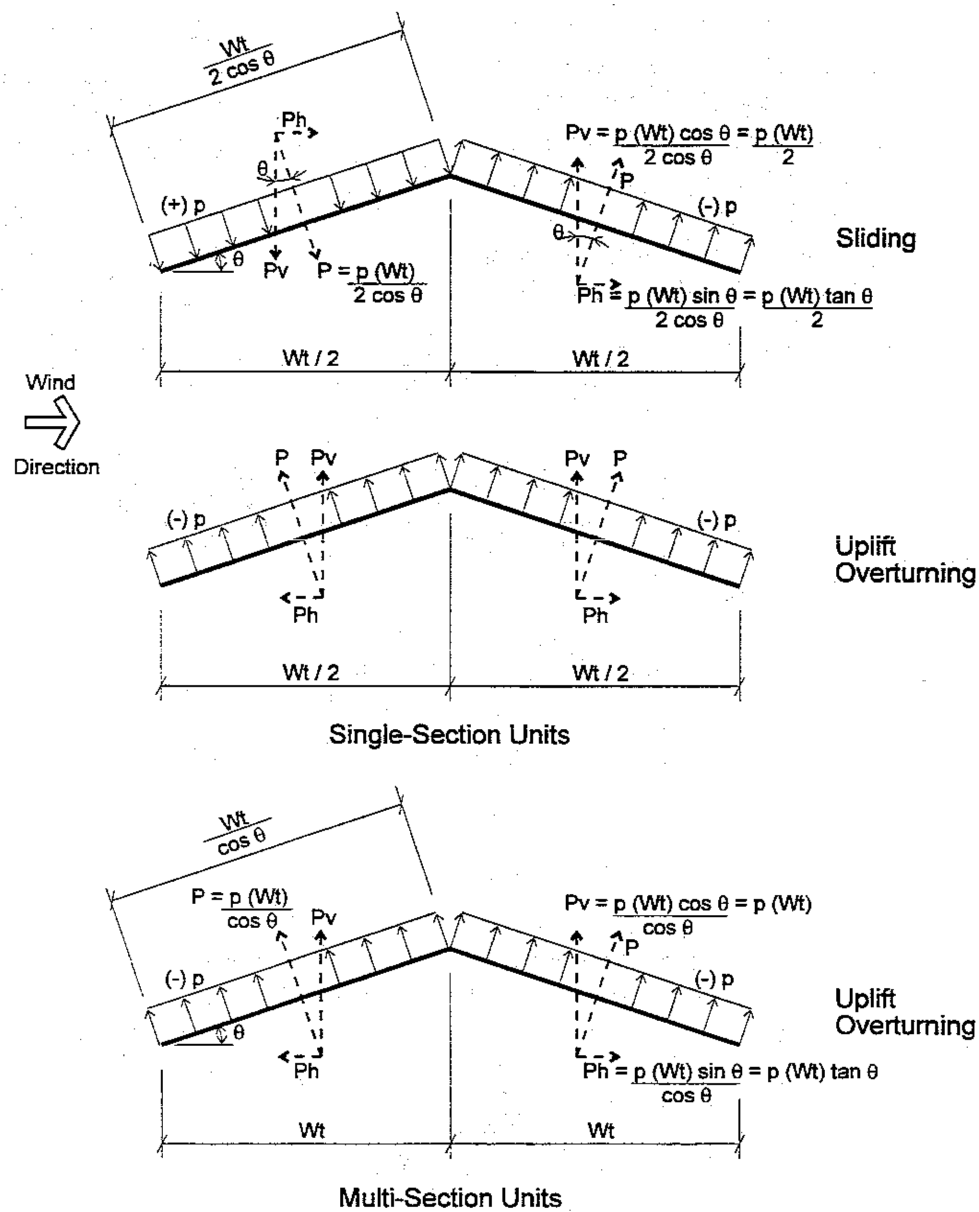


Interior (GCpi) Coefficients

Figure D - 5

surfaces, thus GCpi = ±0.25. Figure D-5 illustrates the pressures and suctions used for various structural considerations. Note that the walls receive offsetting values that cancel any internal effect; therefore, only the roof (GCpi) values are utilized for the calculation of overturning and sliding in the transverse direction. Internal roof Pressures are not utilized in the longitudinal direction.

d. Wind pressures and suctions are typically treated as uniformly



Roof Plane - Wind Components - Transverse Direction

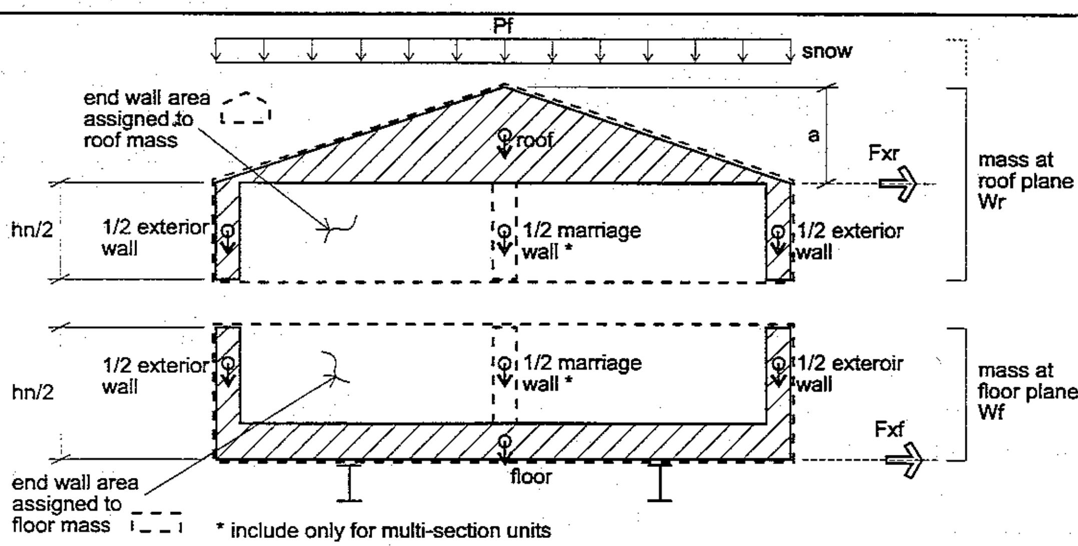
Figure D - 6

distributed and typically applied perpendicular to the orientation of any planar surface. This usually requires the calculation of horizontal and vertical components when wind is applied to sloping surfaces, in this case only roof planes. Figure D-6 illustrates that by the use of trigonometry the resultant force (P) on any sloping surface has components (P_V) and (P_H), which can be arrived at as shown. Note that for the vertical components (P_V) it is possible to merely multiply the pressure (p) by the horizontal length of the slope (Wt/2) for single section units or by (Wt) for multi-section units. This approach simplifies the sample calculations provided in section D-300 for uplift, overturning and sliding in the transverse direction.

D-200.5 SEISMIC LOAD FACTORS.

A. Seismic Versus Wind Forces. It has been stated in Chapters 4 and 6 that seismic forces did not control over wind forces in the computations for consideration of overturning in the transverse direction; however, seismic forces did sometimes control over wind for certain situations of sliding in the transverse and longitudinal direction. This is particularly true in the longitudinal direction because only the end wall elevations are exposed to the wind, producing small applied horizontal forces. Seismic inertia forces are a function of mass that is the same in both directions, which may be larger than the wind forces in particular when the geographic region is also a high snow region.

B. Dead Loads. The model assumes use of the "heavy" dead load values for roof, floor and wall components from Table D-1. It is assumed that the weight of the exterior walls and the weight of the marriage wall (for multi-section units only) are distributed half to the roof plane and half to the floor plane. The



Seismic Dead Load Distribution

Figure D - 7

marriage wall was assumed continuous, without any large openings to maximize the dead load. This distribution of the dead load is illustrated in Figure D-7 to arrive at inertia forces (Fxr) and (Fxf). The weight of the end walls was included in the total mass of the unit and distributed to the roof and floor as shown in Figure D-7 and defined by the equations below:

1. Areas at each end of a Single-Section unit:

$$A_r = \frac{Wt \times a}{2} + \frac{hn}{2} \times Wt$$

$$A_f = Wt \times \frac{hn}{2}$$

2. Areas at each end of a Multi-Section unit:

$$A_r = Wt \times a + 2 \times Wt \times \frac{hn}{2}$$

$$A_f = 2 \times Wt \times \frac{hn}{2}$$

3. These areas are multiplied by the heavy wall weight of 5.9 psf resulting in total roof and floor load additions respectively for Single or Multi-Section units as follows:

$$W_{endroof} = 2 \times 5.9 \times A_r$$

$$W_{endfloor} = 2 \times 5.9 \times A_f$$

The above loads are in pounds and are smeared into the unit's dead load for overturning by using an average length of 60 feet, while for sliding they are smeared into the unit's dead load by dividing by "L". See Section D-200.5.E.7.a for further clarification.

C. Snow Loads. When the flat roof snow load (Pf) is less than 30 psf, the snow load to be attributed to the mass at the roof plane shall be zero. Where siting and snow duration and conditions warrant, and roof snow load is equal to or exceeds 30 psf , the snow load shall be added to the mass of the roof plane. The local authority may permit a reduction in snow load by as much as 80%. See Figure D-7. Note that roof snow load (Pf) has been previously defined as 70% of the ground snow load (Pg) in section D-200.3B.

D. Miscellaneous Loads. No consideration of partial occupancy live load was included in the mass of the floor plane; however, mechanical and partition load was included in the floor plane.

E. Seismic Analysis Method. The Equivalent Lateral Force Procedure (ELF) was assumed for manufactured housing units, as defined by ASCE 7-93. No plan or elevation irregularities were assumed, Thus, the manufactured home superstructure was assumed to be a simple rectangular box with proportions of length to width not exceeding 5 to 1.

1. The Fundamental Period (T): the manufactured home is assumed to have the same period in either direction, transverse or longitudinal, determined from the following equation:

$$T = Ct \times h^{3/4}$$

where:

a. Ct = 0.02 for the category of: all other buildings.

b. the height from bottom of footing to the mean roof height (h) has been assumed as 13.5 feet.

c. Thus: $T = 0.14$ seconds.

2. Site Coefficient (S): the site has been selected for the most significant soil classification, thus $S = 2.0$.

3. The Response Modification Coefficient (R): the structure has been selected as a bearing wall system with light frame walls with shear panels. Thus, $R = 6.5$.

4. Effective peak velocity-related acceleration coefficient ($\underline{A}$v): is selected for the geographic location based on the map H-16 in Appendix H.

5. The Seismic Design Coefficient ($\underline{C}$s) is determined by the following equation:

$$\underline{\underline{Cs}} = \frac{1.2 \times \underline{Av} \times S}{R \times T^{2/3}}$$

Insertion of all the above values in the equation for ($\underline{C}$s) leads to the results tabulated below:

$\underline{A}$v	$\underline{C}$s
0.15	0.204
0.2	0.273
0.3	0.409
0.4	0.546

6. But ($\underline{C}$s) need not exceed the following equation:

$$\underline{\underline{Cs}} = \frac{2.5 \times \underline{\underline{Aa}}}{R}$$

where:

a. Effective peak acceleration coefficient ($\underline{A}$a): selected for the geographic location based on map H-15 in Appendix H.

b. The results are tabulated below:

$\underline{Aa}$	$\underline{Cs}$
0.15	0.058
0.2	0.077
0.3	0.115
0.4	0.154

c. The values for ($\underline{Cs}$) are definitely smaller in item (6.b) above rather than in item (5.a), thus $\underline{Cs}$ is based on the equation in item (6). Thus, for this Manual assuming $\underline{Aa} = \underline{Av}$:

$$\underline{\underline{Cs}} = \frac{2.5 \times \underline{\underline{Aa}}}{R}$$

7. The basic equation for base shear (V_B), using the (ELF) method, is:

$$V_B = \underline{Cs} \times W$$

where:

a. The total weight (W) is the summation of the roof plane mass and the floor plane mass, including snow as applicable, as a function of unit length. It is advantageous to keep the roof and floor loads separated for calculation ease and kept in units of lbs/ft of unit length as follows:

For a Single-Section Unit:

$$w_{roof} = 9.7 \times Wt + 44.25 + \frac{W_{endroof}}{L} + \%P_f \times Wt$$

$$w_{floor} = 13.0 \times Wt + 44.25 + 18 + \frac{W_{endfloor}}{L}$$

For a Multi-Section Unit:

$$w_{roof} = 19.4 \times Wt + 44.25 + 26.25 + \frac{W_{endroof}}{L} +$$
$$2 \times \%P_f \times Wt$$

$$w_{floor} = 26.0 \times Wt + 36 + 44.25 + 26.25 + \frac{W_{endfloor}}{L}$$

Note: For overturning calculations, where (L) does not enter the equations, use L=60 ft as an average length to smear the end wall load. For Sliding (L) is always required and the end wall weight is smearing over the real length (L).

Where for either the Single or Multi-Section unit, the total dead load per foot of length of the unit becomes:

$$W = w_{roof} + w_{floor}$$

b. The seismic coefficient (Cs) is based on equation in item (6.b).

8. The base shear (V_B) is then distributed vertically as inertia forces (F_{xr} and F_{xf}) to the floor and roof levels according to the mass that exists at each level (see Figure D-7), based on the following generic equation:

$$F_x = C_{vx} \times V_B$$

where also generically:

$$C_{vx} = \frac{w_x \times h_x}{\sum_{i=1}^{n}(w_i \times h_i)}$$

a. The weight and height at each respective level is subscripted with an (x) while the sum of the product of each level's weight and height are generically subscripted with an (I). The uppermost level of the building (n) is in this case the roof. For a one story manufactured home, there will only be two levels, w_{roof} and w_{floor} reducing to two expressions substituting Single or Multi-Section unit values as follows:

$$C_{roof} = \frac{w_{roof} \times h_r}{w_{roof} \times h_r + w_{floor} \times h_f}$$

$$C_{floor} = \frac{w_{floor} \times h_f}{w_{roof} \times h_r + w_{floor} \times h_f}$$

Thus, the inertia forces in lbs/ft of unit length at the two respective levels becomes:

$$F_{xr} = C_{roof} \times V_B \qquad \text{and,}$$

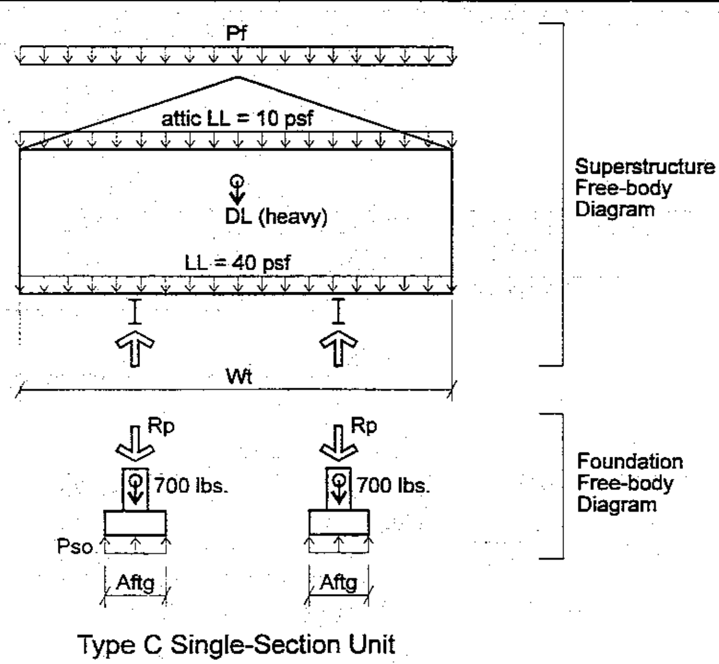
$$F_{xf} = C_{floor} \times V_B$$

b. Sample spreadsheet output for two cases (snow Pg = 0 psf and snow Pg = 100 psf) indicates the range of (F_{xr}) and (F_{xf}) values at the roof and floor levels respectively for a single section unit. These examples include the 12, 14 and 16 nominal width units and are labeled as Tables D-2 and D-3. Note: nominal , rather than actual unit width (Wt) were used in the dead load calculations for conservatism.

9. The forces (F_{xr} and F_{xf}) were applied to the manufactured home unit as illustrated in Figure D-7 and used for transverse and longitudinal overturning and sliding calculations for comparison to the wind forces. The forces that produced the largest required resistance values were used in the Foundation Design Load Tables - Appendix B. Values that are grayed in the Tables of Appendix B are controlled by seismic inertia forces.

D-300. SAMPLE EQUATIONS USED FOR FOUNDATION DESIGN LOAD TABLE VALUES.

D-300.1 REQUIRED EFFECTIVE FOOTING AREA. Refer to Figures D-8(A&B) and D-9(A&B) for the free-body diagrams illustrating the applied gravity loads on the superstructure and on the foundation for a Type **C** and Type **E** or **I** single-section unit, and a Type **C** multi-section unit with consideration of a continuous marriage wall and a marriage wall with a large opening. Note that the "heavy" dead loads are used from Table D-1. For allowable stress design methodology, the load combination from ASCE 7-93 is: DL(heavy) + LL(occupancy) + LL(attic) + SL(or min. roof LL).



Gravity Loads

Figure D -8A

Seismic

Smax=	2	R=	6.5	hn=	11.0 ft
Exposure Group I		Ct=	0.02		
Seismic Performance A to D		Assume no plan or elevation irregularities			
Equivalent Lateral Force Procedure					
Period:	Ta=Ct(hn)^3/4=	0.120802			
	Tmax=Ta*Ca	Ca=	1.5	(1.5 max for Av=.15)	
		Tmax=	0.181203		

Cs max=2.5*Aa/R

Aa	Cs max
0.15	0.057692
0.20	0.076923
0.30	0.115385
0.40	0.153846

Snow Load: Pg= 0 psf Pf= 0 psf

		Wt	
DL	12.0	14.0	16.0
roof	160.65	180.05	199.45
floor	218.25	244.25	270.25
total	378.90	424.30	469.70

Width 12 ft

				Vbase=	21.86	29.15	43.72	58.29
					Fx=Cvx*Vbase			
					Aa			
	w	h	w*h	Cvx	0.15	0.2	0.3	0.4
roof	160.65	11.0	1767.15	0.729654	15.95	21.27	31.90	42.53
floor	218.25	3.0	654.75	0.270346	5.91	7.88	11.82	15.76
sum	378.90		2421.90	1.0	21.86	29.15	43.72	58.29

Width 14 ft

				Vbase=	24.48	32.64	48.96	65.28
					Fx=Cvx*Vbase			
					Aa			
	w	h	w*h	Cvx	0.15	0.2	0.3	0.4
roof	180.05	11.0	1980.55	0.729941	17.87	23.82	35.74	47.65
floor	244.25	3.0	732.75	0.270059	6.61	8.81	13.22	17.63
sum	424.30		2713.30	1.0	24.48	32.64	48.96	65.28

Width 16 ft

				Vbase=	27.10	36.13	54.20	72.26
					Fx=Cvx*Vbase			
					Aa			
	w	h	w*h	Cvx	0.15	0.2	0.3	0.4
roof	199.45	11.0	2193.95	0.730173	19.79	26.38	39.57	52.76
floor	270.25	3.0	810.75	0.269827	7.31	9.75	14.62	19.50
sum	469.70		3004.70	1.0	27.10	36.13	54.20	72.26

Seismic Forces - Ground Snow < 30 psf

Table D - 2

Seismic

Smax= 2 | R= 6.5 | hn= 11.0 ft

Exposure Group I | Ct= 0.02

Seismic Performance A to D | Assume no plan or elevation irregularities

Equivalent Lateral Force Procedure

Period: Ta=Ct(hn)^3/4= 0.120802

Tmax=Ta*Ca | Ca= 1.5 (1.5 max for Av=.15)

Tmax= 0.181203

Cs max=2.5*Aa/R

Aa	Cs max
0.15	0.057692
0.20	0.076923
0.30	0.115385
0.40	0.153846

Snow Load: Pg= 100 psf | Pf= 70 psf

	Wt		
DL	12.0	14.0	16.0
roof	1000.65	1160.05	1319.45
floor	218.25	244.25	270.25
total	1218.90	1404.30	1589.70

Width 12 ft

				Vbase=	70.32	93.76	140.64	187.52
					Fx=Cvx*Vbase			
					Aa			
	w	h	w*h	Cvx	0.15	0.2	0.3	0.4
roof	1000.65	11.0	11007.15	0.943856	66.37	88.50	132.75	176.99
floor	218.25	3.0	654.75	0.056144	3.95	5.26	7.90	10.53
sum	1218.90		11661.90	1.0	70.32	93.76	140.64	187.52

Width 14 ft

				Vbase=	81.02	108.02	162.03	216.05
					Fx=Cvx*Vbase			
					Aa			
	w	h	w*h	Cvx	0.15	0.2	0.3	0.4
roof	1160.05	11.0	12760.55	0.945695	76.62	102.16	153.24	204.31
floor	244.25	3.0	732.75	0.054305	4.40	5.87	8.80	11.73
sum	1404.30		13493.30	1.0	81.02	108.02	162.03	216.05

Width 16 ft

				Vbase=	91.71	122.28	183.43	244.57
					Fx=Cvx*Vbase			
					Aa			
	w	h	w*h	Cvx	0.15	0.2	0.3	0.4
roof	1319.45	11.0	14513.95	0.947095	86.86	115.82	173.72	231.63
floor	270.25	3.0	810.75	0.052905	4.85	6.47	9.70	12.94
sum	1589.70		15324.70	1.0	91.71	122.28	183.43	244.57

Seismic Forces - Ground Snow 100 psf

Table D - 3

A. Gravity Load Considerations for the Type C Single-Section Unit.

1. *General*: The foundation to support the superstructure gravity loads is provided only by the spaced piers under the chassis beams.

2. *Superstructure load to a pier*: As shown in Figure D-8A the snow load, the attic live load and the roof dead load are transferred equally to each exterior wall. The exterior walls in turn transfer the roof loads to the floor framing. The floor live and dead load combine with the roof and wall load to reach the chassis beam, where the foundation piers receive the total concentrated superstructure load (Rp) in proportion to the pier spacing.

$$Rp = \left[(Pf + (40+10)) \times \frac{Wt}{2} + \frac{DL}{2} \right] \times \text{spacing}$$

3. *Typical chassis beam pier foundation weight*: The typical pier assumed for the calculations is based on a pier composed of four 8"x8"x16" concrete masonry units grouted solid with a 2 foot square footing that is 8 inches deep. Thus the assumed pier weight is as follows:

pier: 2.67' × 1.33' × 84 psf = 298.0 lbs.
footing: 150 pcf × 2' × 2' × .67' = 402.0 lbs.
total = 700.0 lbs.

4. *Required chassis beam Pier Footing size*: The footing area (Aftg) must be large enough so that the net allowable soil bearing pressure (Pso) is not exceeded under the full gravity dead, live and snow loads. Note that the pier and footing weight become additional dead load.

$$Aftg = \frac{Rp + 700}{Pso}$$

B. Gravity Load Considerations for the Type E and I Single-Section Unit.

1. *General*: Support of the superstructure gravity loads is shared by the exterior longitudinal walls and the spaced interior piers under the chassis beams, which together comprise the foundation.

2. *Superstructure load to the exterior longitudinal foundation walls*: As shown in Figure D-8B, the snow load, the attic live load and the heavy roof dead load are transferred equally to each exterior wall. The exterior wall weight is added, and both loads transfer directly to the exterior foundation walls. A portion (dc/2) of the floor live and heavy dead load also goes to the exterior foundation walls. The total superstructure gravity load (Rw) transferred to the exterior foundation wall is in units of lbs./ft. of home length. The equation is as follows:

$$Rw = [Pf + (9.7+10)] \times \frac{Wt}{2} + (40+13) \times \frac{dc}{2} + 44.25$$

[snow + (roof DL+attic LL)]
(floor LL+DL) + (wall DL)

3. *Superstructure load to an interior pier*: The remainder of the floor dead and live load is equally divided between the chassis beam lines, and concentrated at the foundation piers based on their spacing. The total superstructure concentrated gravity load to a pier (Rp) is as follows:

$$Rp = \left[(40+13) \times \frac{Wt - dc}{2} + 9 \right] \times \text{spacing}$$

(floor DL+LL) (chassis beam DL)

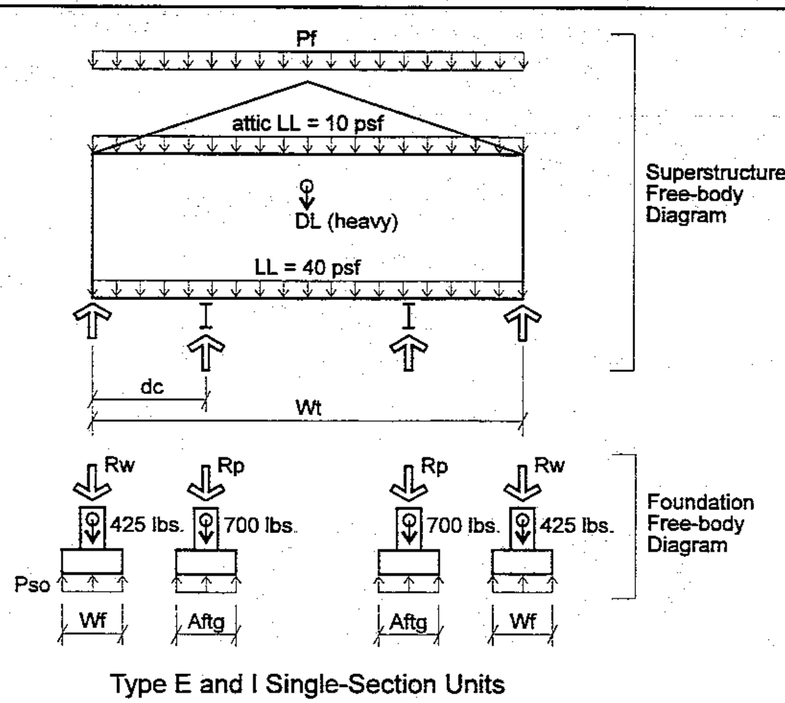
4. *Typical exterior longitudinal foundation wall weight*: The typical exterior longitudinal foundation wall is assumed to be composed of a 6" poured concrete wall, 3'-8" high, and a 6" x 24" continuous concrete footing. Thus, the assumed weight is as follows:

wall: 150 pcf × 3.67' × 0.5' = 275.0 plf
footing: 150 pcf × 2' × 0.5' = 150.0 plf
total = 425.0 plf

5. *Required Exterior Wall Footing Width*: The footing width (Wf) must be large enough so that the net allowable soil bearing pressure (Pso) is not exceeded under the full gravity dead, live and snow loads. Note that the longitudinal foundation wall and footing weight become additional dead load. The required footing width:

$$Wf = \frac{Rw + 425}{Pso}$$

6. *Required Interior Pier Footing Area*: The footing area (Aftg) must



Gravity Loads

Figure D -8B

be large enough so that the allowable soil bearing pressure (Pso) is not exceeded under the full gravity dead and live loads. Note that the pier and footing weight become additional dead load. The required footing area:

$$\text{Aftg} = \frac{\text{Rp} + 700}{\text{Pso}}$$

C. Gravity Load Considerations for the Type C Multi-Section Unit with a Continuous Superstructure Marriage wall.

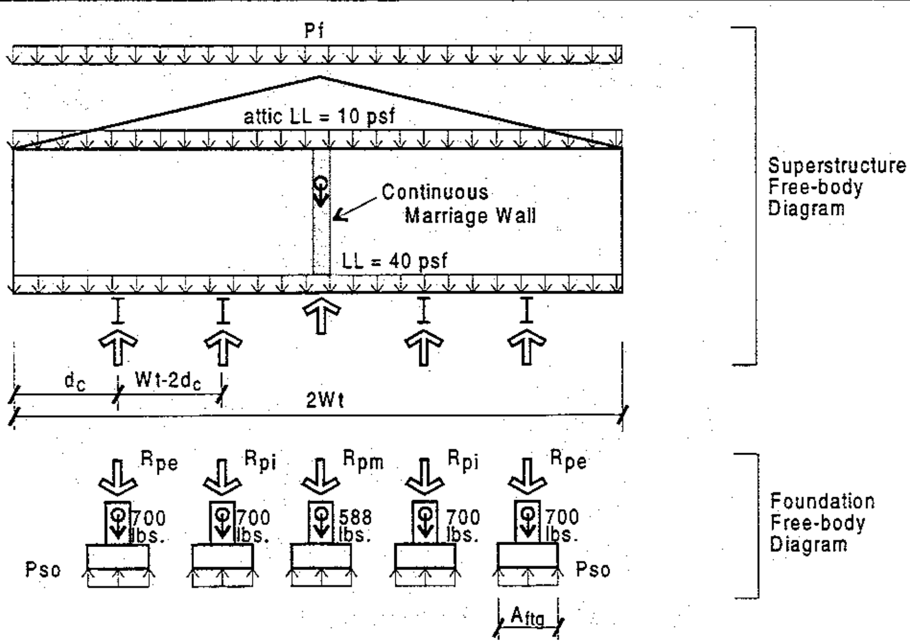
1. *General*: The foundation to support the superstructure gravity loads is provided only by spaced piers under the chassis beams and under the continuous marriage wall.

2. *Superstructure continuous marriage wall load to a pier*: As shown in Figure D-9A the snow load, the attic live load and the roof dead load are transferred between the marriage wall and the exterior walls as bearing walls. The marriage wall in turn transfer the roof loads to the floor framing. A small portion of the floor live and dead load is assumed to combine with the roof loads and marriage wall weight to reach the top of the foundation pier as the total concentrated superstructure load (Rpm) in proportion to the pier spacing.

$$\text{Rpm} = \begin{bmatrix} 52.5 + (\text{Pf} + 19.7) \times \text{Wt} + \\ (40 + 13) \times \text{dc} \end{bmatrix} \times \text{spacing}$$

[marr.wall+(snow+roofDL+attic LL)]
(floor DL+LL)

3. *Superstructure load to an exterior chassis beam pier*: As shown in Figure D-9A the snow load, the attic



Type C - Multi-Section Unit w/Continuous Marriage Wall

Figure D - 9A

live load and the roof dead load are transferred equally between the exterior wall and the marriage wall. The exterior wall in turn transfers the roof loads to the floor framing. The floor live and dead load combine with the roof and wall weight to reach the chassis beam, where the foundation piers receive the total concentrated superstructure load (Rpe) in proportion to the pier spacing.

$$\mathrm{Rpe} = \begin{bmatrix} (\mathrm{Pf} + 19.7 + 40 + 13) \times \mathrm{Wt}/2 + \\ (44.25 + 9) \end{bmatrix} \times \text{spacing}$$

[snow+roofDL+atticLL+floorDL+LL]
(ext.wall DL+chassis bm.)

4. *Superstructure load to an interior chassis beam pier*: As shown in Figure D-9A The floor live and dead load comprise the only load to reach the interior chassis beam, where the foundation piers receive the total concentrated superstructure load (Rpi) in proportion to the pier spacing.

$$\mathrm{Rpi} = \left[(40 + 13) \times \left(\frac{\mathrm{Wt - dc}}{2} \right) + 9 \right] \times \text{spacing}$$

[(floorLL+DL)+chassis bm.]

5. *Typical Continuous Marriage Wall Pier*: The typical continuous marriage wall within the superstructure of the multi-section unit is assumed to have a foundation pier composed of five courses of 8"x8"x16" concrete block (ungrouted), and a concrete footing 2'x2' by 8" deep. The dead load of a typical continuous marriage wall foundation pier is as follows:

pier: 42 psf $\times$ 3.33' $\times$ 1.33' =	186.0 lbs.
footing: 150 pcf $\times$ 2^2 $\times$ .67' =	402.0 lbs.
total =	588.0 lbs.

6. *Required continuous marriage wall pier footing*. The footing area (Aftg) must be large enough so that the net allowable soil bearing pressure (Pso) is not exceeded under the full gravity dead, live and snow loads. Note that the pier and footing weight become additional dead load. The required footing area:

$$\mathrm{Aftg_{mar}} = \frac{\mathrm{Rpm} + 588}{\mathrm{Pso}}$$

7. *Required exterior chassis beam Pier Footing Area*: The footing area (Aftg) must be large enough so that the allowable soil bearing pressure (Pso) is not exceeded under the full gravity dead and live loads. Note that the pier and footing weight become additional dead load. The required footing area:

$$\mathrm{Aftg_{ext}} = \frac{\mathrm{Rpe} + 700}{\mathrm{Pso}}$$

8. *Required interior chassis beam Pier Footing Area*: The footing area (Aftg) must be large enough so that the allowable soil bearing pressure (Pso) is not exceeded under the full gravity dead and live loads. Note that the pier and footing weight become additional dead load. The required footing area:

$$\mathrm{Aftg}_{\mathrm{int}} = \frac{\mathrm{Rpi} + 700}{\mathrm{Pso}}$$

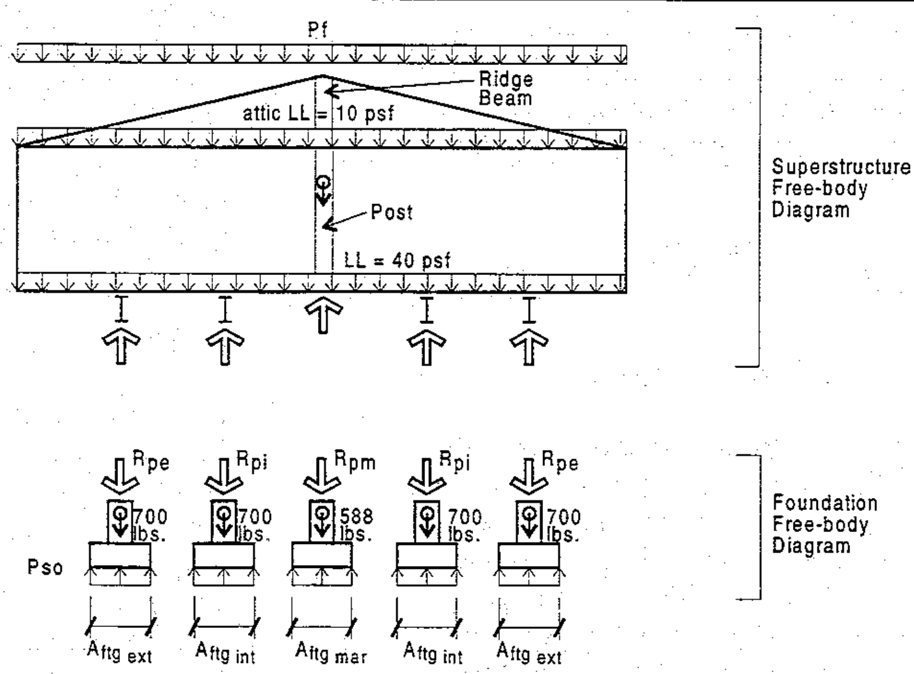
D. Gravity Load Considerations for the Type C Multi-Section Unit with a Superstructure Marriage wall containing one opening or two large adjacent openings.

1. *General*: The foundation to support the superstructure gravity loads, as illustrated in Figure D-9B, is provided only by spaced piers under the chassis beams, piers placed under the posts at the ends of marriage wall openings, and equally spaced piers under the continuous portions of the marriage wall.

2. *Marriage wall openings: limitations and assumptions.* Two marriage wall opening situations were reviewed: (1) a **single opening,** as illustrated in Figure D-9C, is bounded by posts at the ends of the opening with continuous marriage walls extending beyond the opening width in both directions, and (2) **two adjacent marriage wall openings**, as illustrated in Figure D-9D, consisting of three posts with the outer two posts having continuous marriage walls extending beyond the two openings.

Note: A maximum 10 foot pier spacing was assumed under all continuous marriage wall portions.

Note: The center post between the two adjacent openings of the later scheme produces the largest concentrated load to a marriage wall pier. This is the condition used for



Type C - Multi-Section Unit w/Openings in Marriage Wall

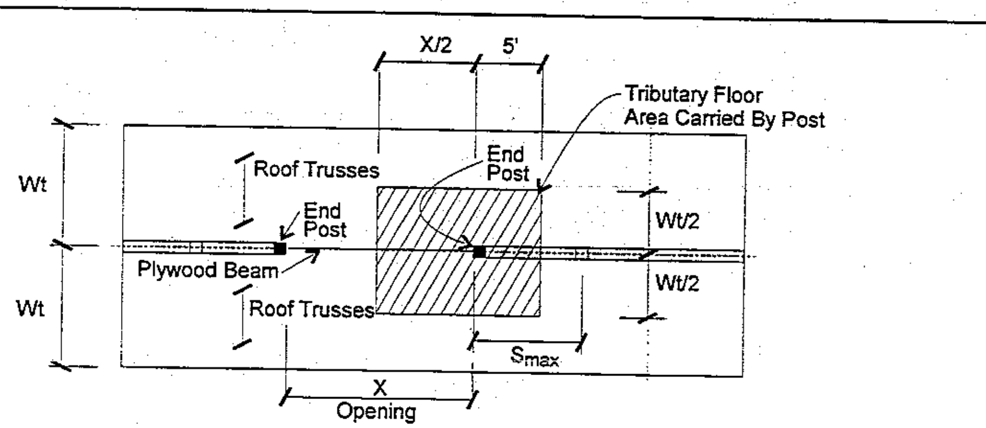
Figure D - 9B

centrated load to a marriage wall pier. This is the condition used for the equations that follow.

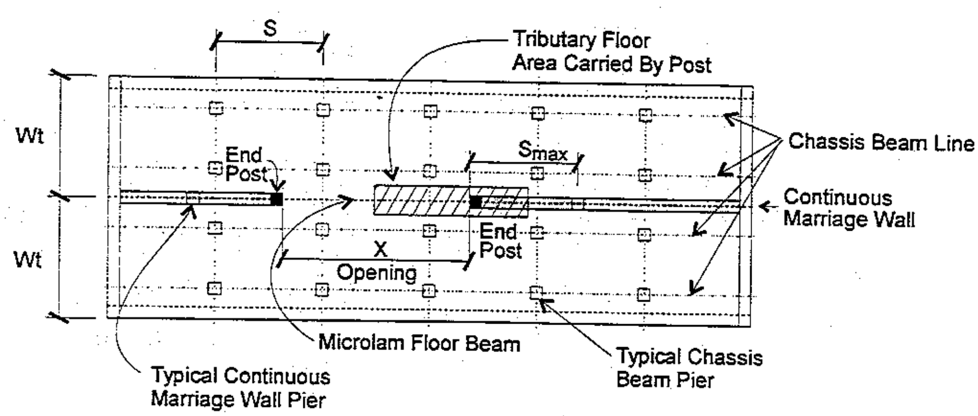
Note: It is conservatively assumed that the footing size required under the center post will be used under all three posts in the Appendix B Part 1 Tables for Multi-section units.

Note: The opening width used for two adjacent openings in the Appendix B Tables, is the average of the two opening widths: $(x+x_1) \div 2$. The marriage wall opening tables use 2 foot increments for single openings, or the average of two adjacent openings, from 10 feet to 20 feet.

3. *Superstructure: Two large adjacent marriage wall openings: load to the pier under center post*: As shown in Figure D-9D the snow load, the attic



Roof Plan



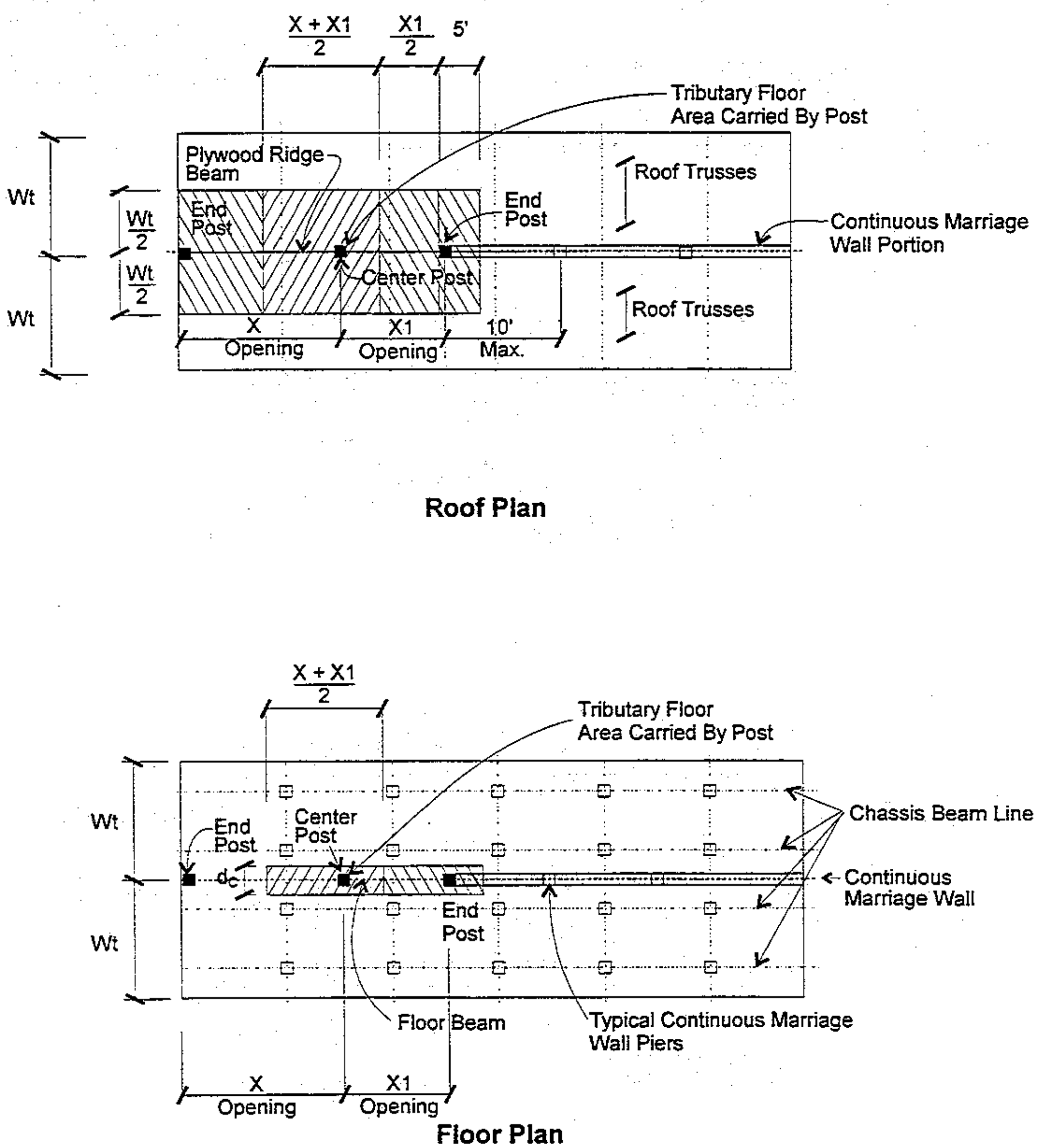
Floor Plan

Marriage Wall w/Large Single Opening

Figure D - 9C

live load and the roof dead load are transferred between the marriage wall ridge beam and the exterior walls as bearing points. The marriage wall ridge beam (assumed to act as two simple span beams) transfers the average of the two opening widths of the roof and attic loads to the center post. The floor live and dead load is assumed to be carried by floor beams, and also is transferred based on the average width of the two openings.

Note: For a single opening x_1=0 and all the formulas still work.



Marriage Wall w/Two Adjacent Openings

Figure D - 9D

The floor beam is assumed to weigh 10 plf and the ridge beam is assumed to be composed of 6 layers of 3/8"plywood at a depth of 3 feet. Thus, the ridge beam weighs 19.8 plf. The post is assumed to be a 4x4 of weight 32 lbs. The total concentrated superstructure load to the pier (Rmax):

$$\mathrm{Rpm} = \begin{bmatrix} (\mathrm{Pf}+19.7)\times \mathrm{Wt} + \\ (40+13)\times \mathrm{dc}+10+19.8 \end{bmatrix} \times \left(\frac{\mathrm{x}+\mathrm{x}_1}{2}\right) + 32$$

[(snow+roofDL+attic LL)]
(floor DL+LL)+floor bm+Ridge bm)
+post DL

4. *Superstructure load to an exterior chassis beam pier*: the superstructure load to an exterior pier is unchanged from that for a Type **C** multi-section unit with a continuous marriage wall. Thus, the total concentrated superstructure load (Rpe) is repeated here:

$$\mathrm{Rpe} = \begin{bmatrix} (\mathrm{Pf}+19.7+40+13)\times \mathrm{Wt}/2 + \\ (44.25\ +9) \end{bmatrix} \times \text{spacing}$$

[snow+roofDL+atticLL+floorDL+LL]
(ext.wall DL+chassis bm.)

5. *Superstructure load to an interior chassis beam pier*: the superstructure load to an interior pier is unchanged from that for a Type **C** multi-section unit with a continuous marriage wall. Thus, the total concentrated superstructure load (Rpi) is repeated here:

$$\mathrm{Rpi} = \left[(40+13)\times\left(\frac{\mathrm{Wt}\text{ - }\mathrm{dc}}{2}\right)+9\right]\times \text{spacing}$$

[(floorLL+DL)+chassis bm.]

6. *Required pier footing area for marriage wall containing large openings.* The footing area (Aftg) must be large enough so that the net allowable soil bearing pressure (Pso) is not exceeded under the full gravity dead, live and snow loads. Note that the pier and footing weight become additional dead load. The required footing area:

$$\mathrm{Aftg}_{mar} = \frac{\mathrm{Rpm}+588}{\mathrm{Pso}}$$

7. *Required pier footing areas for exterior and interior chassis beam lines.* See items 7 and 8 for a Type **C** Multi-section unit with a continuous marriage wall. Equations are the same and are repeated here for ease of use:

$$\mathrm{Aftg}_{ext} = \frac{\mathrm{Rpe}+700}{\mathrm{Pso}}$$

$$\mathrm{Aftg}_{int} = \frac{\mathrm{Rpi}+700}{\mathrm{Pso}}$$

E. Gravity Load Considerations for the Type E and Type I Multi-Section Unit with a Continuous Superstructure Marriage Wall.

1. *General*: As illustrated in Figure D-9E, the foundation to support the superstructure gravity loads is provided by spaced piers under the chassis beams, along the exterior wall and by spaced piers under the continuous marriage wall.

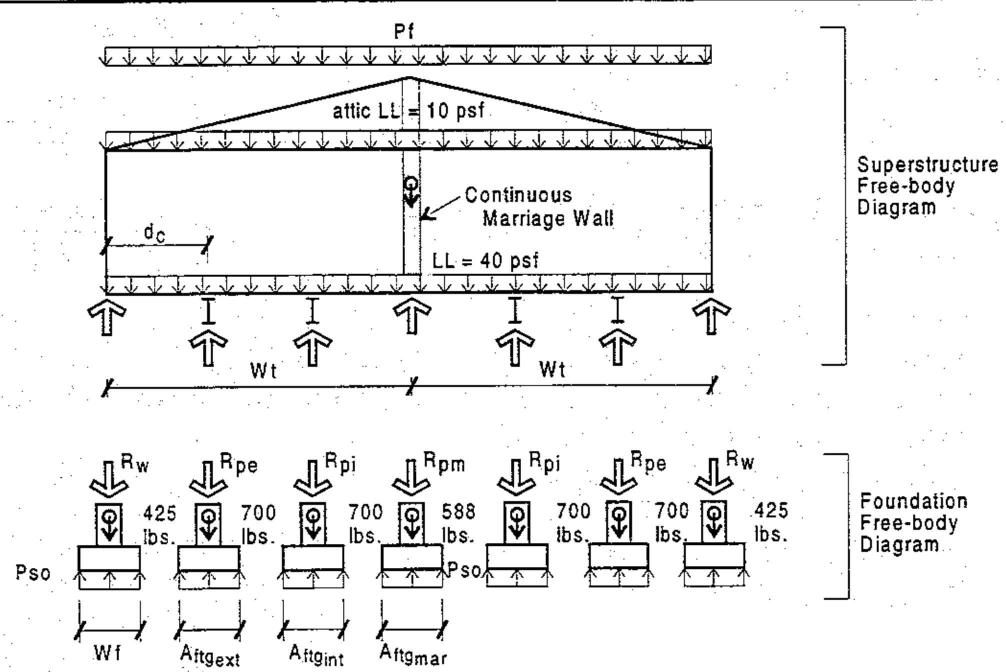
Note: Foundation Concepts **E5** and **E7** do not follow the equation development presented here and are treated separately, later in Section D.300.1.

2. *Superstructure continuous marriage wall load to a pier*: Identical to that shown in Figure D-9A for the Type C multi-section unit; the snow load, the attic live load and the roof dead load are transferred between the marriage wall and the exterior walls as bearing walls. As shown in Figure D-9E the marriage wall in turn transfers the roof loads to the floor framing. A small portion of the floor live and dead load is assumed to combine with the roof loads and marriage wall weight to reach the top of the foundation pier as the total concentrated superstructure load (Rpm) in proportion to the pier spacing.

$$Rpm = \begin{bmatrix} 52.5 + (Pf + 19.7) \times Wt + \\ (40 + 13) \times dc \end{bmatrix} \times \text{spacing}$$

[marr.wall+(snow+roofDL+attic LL)]
(floor DL+LL)

3. *Superstructure load to an exterior and interior chassis beam pier*: As shown in Figure D-9E there are no gravity roof loads or exterior wall load transferred to the piers under the chassis beams. The floor live and dead load comprise the only load to reach the exterior and interior chassis beam, where the exterior and interior foundation piers receive the total concentrated superstructure load (Rpe and Rpi) equally in proportion to the pier spacing.



Type E and I - Multi-Section Units w/Continuous Marriage Wall

Figure D - 9E

$$Rpe = Rpi = \left[\begin{array}{l} (40+13)\times \\ \left(\dfrac{Wt - 2dc}{2} + \dfrac{dc}{2} \right) + 9 \end{array} \right] \times spacing$$

(floorDL+LL) (chassis bm.)

4. *Superstructure load to the exterior foundation wall.* As shown in figure D-9E the snow load, the attic live load and the roof dead load are transferred equally between the exterior wall and the marriage wall. The exterior wall in turn transfers the roof loads to the floor framing. A small portion of the floor live and dead load combine with the roof and wall weight to reach the foundation wall as a lineal uniform load (Rw).

$$Rw = (pf + 19.7) \times \frac{Wt}{2} + (40+13) \times \frac{dc}{2} + 52.5$$

5. *Required continuous marriage wall pier footing.* The footing area (Aftg) must be large enough so that the net allowable soil bearing pressure (Pso) is not exceeded under the full gravity dead, live and snow loads. Note that the pier and footing weight become additional dead load. The required footing area:

$$Aftg_{mar} = \frac{Rpm + 588}{Pso}$$

6. *Required exterior and interior chassis beam Pier Footing Area:* The footing areas (Aftg) must be large enough so that the allowable soil bearing pressure (Pso) is not exceeded under the full gravity dead and live loads. Note that the pier and footing weight become additional dead load. The required footing areas:

$$Aftg_{ext} = Aftg_{int} = \frac{Rpe + 700}{Pso}$$

7. *Required exterior foundation wall Footing Width:* The footing width (Wf) must be large enough so that the allowable soil bearing pressure (Pso) is not exceeded under the full gravity dead and live loads. Note that the wall and footing weight become additional dead load. The required footing width becomes:

$$Wf = \frac{Rw + 425}{Pso}$$

F. Gravity Load Considerations for the Type E and I Multi-Section Units with a Superstructure Marriage wall containing one opening or two large adjacent openings.

1. *Continuous marriage wall.* The equation development presented in Section D.300.1.E for loads and footing sizes at exterior foundation wall and exterior and interior chassis beam line piers is identical to that when a continuous marriage wall exists and will not be repeated here.

2. *One opening or two adjacent openings.* The considerations for the foundation piers under the posts that define one opening, or two adjacent large openings within the length of the continuous marriage wall, is the same as that for the Type C Multi-section unit presented in Section D.300.1D and illustrated in Figure

D-9D. The equation for the maximum reaction under the center post will be repeated here:

$$Rpm = \begin{bmatrix} (Pf + 19.7) \times Wt + \\ (40 + 13) \times dc + 10 + 19.8 \end{bmatrix} \times \left(\frac{x + x_1}{2} \right) + 32$$

[(snow+roofDL+attic LL)]
(floor DL+LL)+floor bm+Ridge bm)
+post DL

Note: For a single opening $x_1 = 0$ and all the equations still work.

3. *Required marriage wall pier footing at center post.* The pier and footing weight become additional dead load. The required footing area under the center post location repeats also:

$$Aftg_{mar} = \frac{Rpm + 588}{Pso}$$

Note: Regardless of Multi-Section Unit Type **C**, **E** or **I** the equations developed for piers under the continuous marriage walls and the equations developed for the pier under the center post, when two large marriage wall openings exist, do not change. The only exception is for the Type **E5**, (**E6** uses **E5** Tables) and **E7** Multi-section units, which are presented further on in this Section D.300.1.

G. Gravity Load Considerations for the Type Cnw Multi-Section Unit with a continuous marriage wall (without any marriage wall piers).

1. *General*: As illustrated in Figure D-9F, the foundation to support the superstructure gravity loads is provided by spaced piers under the exterior and interior chassis beams. **Note**: A marriage wall with large openings is not considered feasible for this foundation concept, since it would require piers under the posts.

2. *Superstructure load to an interior or exterior chassis beam pier*: Similar to that shown in Figure D-9A for the Type **C** multi-section unit; the snow load, the attic live load and the roof dead load are transferred between the marriage wall and the exterior walls as bearing walls. The marriage wall and the exterior wall in turn transfer their dead weight and the roof loads to the floor framing. The floor live and dead load is equally distributed each chassis beam line. These loads from both levels combine to reach the top of the foundation pier as the total concentrated superstructure load (Rp) in proportion to the pier spacing.

Note: The only difference between the exterior pier load and the interior pier load is in the difference of the weight of the exterior wall and marriage wall. Since the exterior wall has a greater weight that the marriage wall, it will be used and thus the load to the exterior and interior chassis beam piers will be assumed equal.

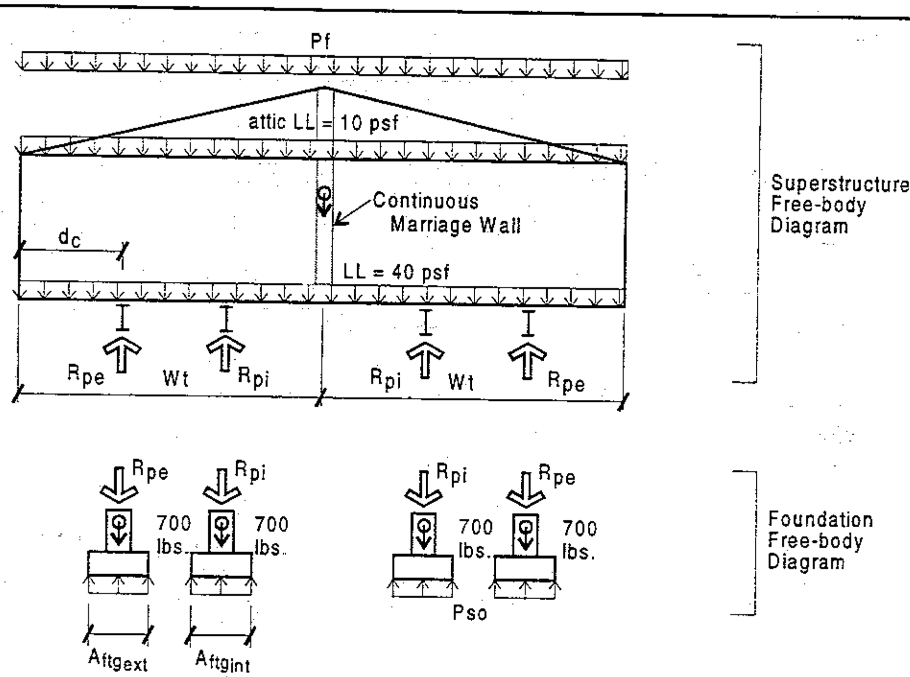
$$Rpe = Rpi = \begin{bmatrix} (pf + 19.7) \times \dfrac{Wt}{2} + \\ (40+13) \times \dfrac{Wt}{2} \\ +(44.25+9) \end{bmatrix} \times spacing$$

3. *Required exterior and interior chassis beam Pier Footing Area:* The footing area (Aftg) must be large enough so that the allowable soil bearing pressure (Pso) is not exceeded under the full gravity dead and live loads. Note that the pier and footing weight become additional dead load. The required exterior and interior footing areas become:

$$Aftg_{ext} = Aftg_{int} = \frac{Rp + 700}{Pso}$$

H. Gravity Load Considerations for the Type E5 Multi-Section Unit with a Continuous Superstructure Marriage Wall.

1. *General:* As illustrated in Figure D-9G, the foundation to support the superstructure gravity loads is provided by spaced transverse steel girders (under the chassis beams) that span between pilasters built into the exterior foundation walls and by spaced piers under the continuous marriage wall. A crawlspace exists below the first floor. The transverse steel girder is assumed to be composed of two simple spans that run from exterior wall to the central marriage wall piers, rather than



Type Cnw - Multi-Section Units w/Continuous Marriage Wall

Figure D - 9F

create a continuous two span girder.

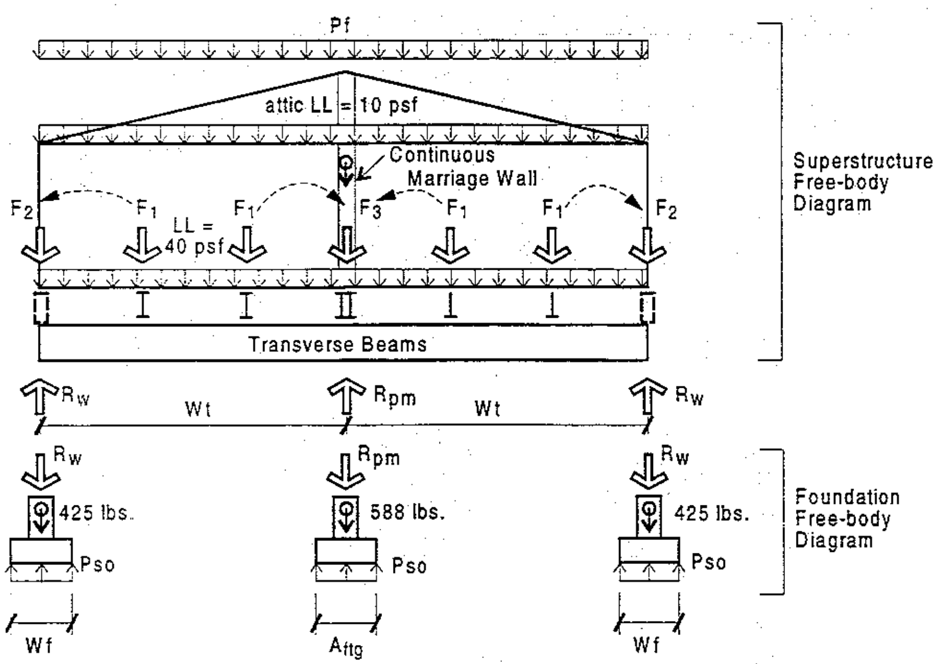
Note: A licensed professional shall be responsible for the design of the transverse steel girders.

2. *Superstructure floor load transferred to the transverse steel girder and then to the exterior foundation wall pilaster.* As shown in Figure D-9G the floor dead and live load transfer to the chassis beam lines and present concentrated loads to the transverse girder. This concentrated load is then assumed to transfer to the end of the transverse girder and bear on the pilaster. Based on the transverse girder spacing, the concentrated load (F1) becomes:

$$F1 = \left[(40+13) \times \left(\frac{\frac{Wt - 2dc}{2} +}{\frac{dc}{2}} \right) + 9 \right] \times \text{spacing}$$

(floorLL+DL)

3. *Superstructure load to the exterior foundation wall*: As shown in Figure D-9G the snow load, the attic live load and the roof dead load are transferred between the marriage wall and the exterior walls as bearing walls. The exterior wall transfers this load down to the top of the foundation wall. A small portion of floor load is assumed to also go to the foundation wall. This is a uniform linear load (F2) as follows:



Type E5 Multi-Section Units w/Continuous Marriage Wall

Figure D - 9G

$$F2 = (Pf + 19.7) \times \frac{Wt}{2} + (40 + 13) \times \frac{dc}{2} + 44.25$$

(snow +roof DL+atticLL) (floorLL+DL) (exterior wallDL)

4. *Superstructure total load to the exterior foundation wall*: As shown in Figure D-9G the pilaster receives load (F1) and this load plus the transverse girder weight of 20 plf spreads at a 45° angle along the wall length based on an assumed wall depth of 2 feet. Therefore, the spread in the wall is 4 feet. This spread load combines with the roof and exterior wall load (F2) to produce a total reaction (Rw) to the footing as follows:

$$Rw = \frac{F1 + \left(20 \times \frac{Wt}{2}\right)}{4} + F2$$

5. *Superstructure load at the marriage wall*: As shown in Figure D-9G the snow load, the attic live load and the roof dead load are transferred between the marriage wall and the exterior walls as bearing walls. The continuous marriage wall transfers this load down to the floor level and to a short steel post as a concentrated load, based on the spacing of the transverse girders. This concentrated load (F3) is as follows:

$$F3 = \begin{bmatrix} (Pf + 19.7) \times Wt + 52.5 + \\ (13 + 40) \times dc \end{bmatrix} \times \text{spacing}$$

(snow +roofDL+LL) (marriage wall weight) (floor DL+LL)

6. *Superstructure total load to a continuous marriage wall pier*: As shown in Figure D-9G two concentrated floor loads (F1) plus the concentrated load (F3) in addition to the transverse girder weight of 20 plf are assumed to be transferred to the continuous marriage wall pier as a total concentrated load (Rpm) as follows:

$$Rpm = 2 \times F1 + F3 \ + 20 \times Wt$$

7. *Required exterior foundation wall Footing Width*: The footing width (wf) must be large enough so that the allowable soil bearing pressure (Pso) is not exceeded under the full gravity dead and live loads. Note that the wall and footing weight become additional dead load. The required footing width becomes:

$$Wf = \frac{Rw + 425}{Pso}$$

Note: The width of the footing between pilasters is assumed to be the same as at the pilaster. It is uneconomical to continually jog footing forms. Plus the spread through the wall will almost encompass the the entire wall between pilasters anyway.

8. *Required continuous marriage wall pier footing*. The footing area (Aftg) must be large enough so that the net allowable soil bearing pressure (Pso) is not exceeded under the

full gravity dead, live and snow loads. Note that the pier and footing weight become additional dead load. The required footing area:

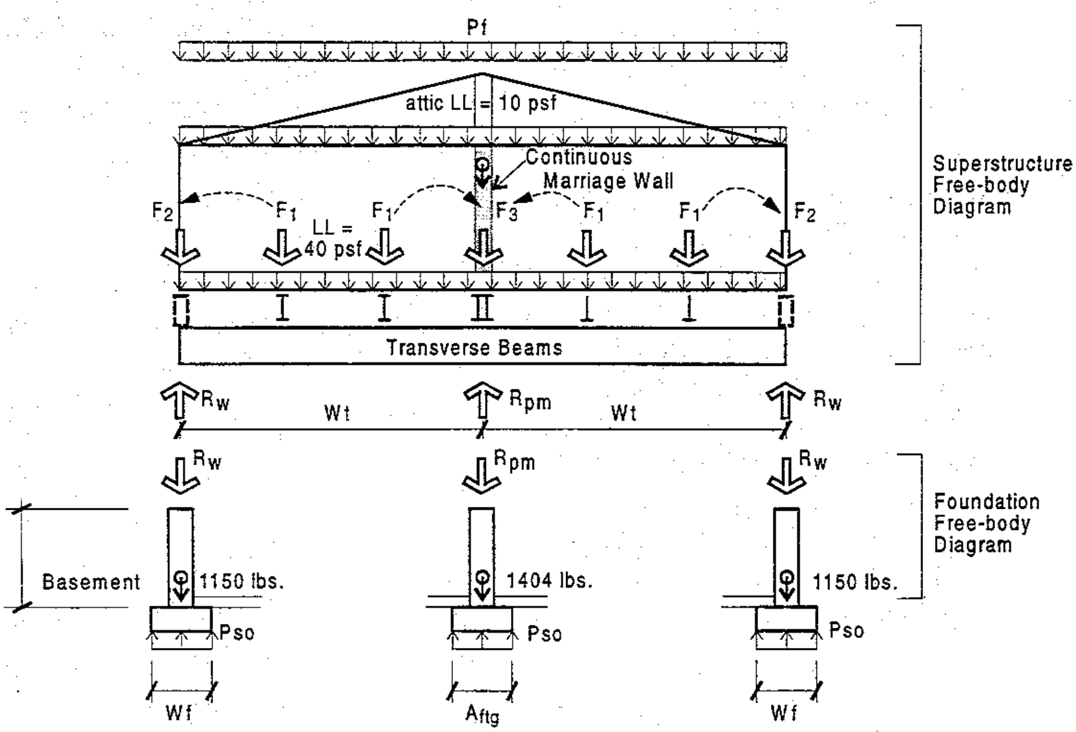
$$Aftg_{mar} = \frac{Rpm + 588}{Pso}$$

I. Gravity Load Considerations for the Type E7 Multi-Section Unit with a Continuous Superstructure Marriage Wall.

1. *General*: As illustrated in Figure D-9H, the load flow of the superstructure gravity loads is identical to that for the Type **E5** multi-section unit, and the equation development is very similar. The only difference is that instead of a crawlspace, a full basement exists below the first floor. Thus, the exterior foundation is a full depth basement wall and footing with space pilasters. Again, the transverse steel girder is assumed to be composed of two simple spans that run from exterior basement wall to the central marriage wall, where steel pipe columns and a spread footing are used instead of piers.

Note: A licensed professional shall be responsible for the design of the basement wall for gravity loads and lateral earth pressures; as well as the transverse steel girders and the steel pipe column.

2. *Exterior foundation basement wall and footing assumptions*. A 6'-8" headroom is assumed under the transverse girders that are assumed



Type E7 Multi-Section Units w/Continuous Marriage Wall

Figure D - 9H

to be 12 inches deep. The chassis beams are assumed to be 10" deep. Thus, the total wall height to the top of basement floor is 8'-6". To maximize the gravity loading the walls are assumed to be 8 " solid concrete, rather than the also acceptable reinforced concrete block. The linear footing proportions are set at 1 foot deep x 2 feet wide. Since the pilaster only exists at the spacing of transverse girders its weight has been ignored. The foundation dead load becomes:

Conc. wall: 0.67'× 8.5'× 150 pcf =	850 plf
footing: 1.0 × 2.0 × 150 pcf =	300 plf
total =	1150 plf

3. *Foundation under the marriage wall*: Steel pipe columns 3.5"ϕ are assumed spaced under the transverse girders with a base plate at the bottom and a cap plate at the top with holes for bolting. The footing is assumed to be 1'deep x 3' x 3'. The column/footing load is therefore:

Column: 7.6 plf ×7 feet tall =	54 lbs.
Footing: 150pcf×1'×3'×3' =	1350 lbs.
total =	1404 lbs.

4. Superstructure floor load transferred to the transverse steel girder and then to the exterior foundation wall pilaster. The load (F1) is identical to that for the Type **E5** Multi-Section unit found in section D.300.1.H.2.

5. *Superstructure load to the exterior foundation wall*: The load (F2) is identical to that for the Type **E5** Multi-Section Unit found in section D.300.1.H.3.

6. *Superstructure load at the marriage wall*: The load (F3) is identical to that for the Type **E5** Multi-Section Unit found in section D.300.1.H.5.

7. *Superstructure total load to the exterior foundation wall*: The pilaster receives load (F1) and the transverse girder weight of 20 plf. This load spreads at a 45° angle along the wall length based on an assumed wall depth of 8'-6" below the superstructure. Therefore, the spread in the wall would be greater than the maximum 10 foot spacing for transverse girders. The maximum Code prescribed spread is thus the spacing (s). This spread load combines with the roof and exterior wall load (F2) to produce a total reaction (Rw) to the footing as follows:

$$\mathrm{Rw} = \frac{\mathrm{F1} + \left(20 \times \frac{\mathrm{Wt}}{2}\right)}{\mathrm{s}} + \mathrm{F2}$$

8. *Superstructure total load to a continuous marriage wall pier*: The total concentrated load to the steel pipe column is identical to that for the Type **E5** Multi-section unit concentrated load to a pier, found in section D.300.1.H.6. The total concentrated load (Rpm) is repeated here as follows:

$$\mathrm{Rpm} = 2 \times \mathrm{F1} + \mathrm{F3} + 20 \times \mathrm{Wt}$$

9. *Required exterior foundation wall Footing Width*: The footing width (wf) must be large enough so that the allowable soil bearing pressure (Pso) is not exceeded under the full gravity dead and live loads. Note that the wall and footing weight become additional dead load. The required footing width becomes:

$$Wf = \frac{Rw + 1150}{Pso}$$

10. *Required continuous marriage wall pipe column footing*. The footing area (Aftg) must be large enough so that the net allowable soil bearing pressure (Pso) is not exceeded under the full gravity dead, live and snow loads. Note that the steel column and footing weight become additional dead load. The required footing area:

$$Aftg_{mar} = \frac{Rpm + 1404}{Pso}$$

11. *Basement longitudinal beams used to space steel pipe columns further apart:* It would be possible to add longitudinal steel beams to support the transverse steel girders in order to avoid a large number of pipe columns. This produces concentrated loads to the longitudinal beams, which could be spaced (b) distance apart, assuming (b)>(s) by a significant amount. The value (n) is the number of transverse beams that occurs within the distance (b). The area of footing would then become:

$$Aftg_{mar} = \frac{b \times \left[\frac{2 \times F3 + F4}{s} + n \times 20 \times Wt\right] + 1404}{Pso}$$

Note: There are no tables in Appendix B to cover this situation. The steel pipe column, the transverse and longitudinal steel beams would require design by an engineer.

J. Gravity Load Considerations for the Type E5, E6 and E7 Multi-Section Units with a Superstructure Marriage Wall containing one opening or two large adjacent openings.

1. *General*: The presence of regularly spaced steel transverse girders in these foundation concepts complicates the equation development to account for randomly placed large openings along the marriage wall line. Any concentrated post load, defining the ends of an opening, that falls between transverse girders would require either another pier or column, that would in many cases be close enough to the grid of transverse girder piers and posts, as to overlap or abut- clearly uneconomical and impractical to construct.

2. *Marriage wall openings: assumptions and limitations*: It has been assumed and is now recommended in this Handbook, that opening widths for these three foundation concepts be a multiple of the transverse girder spacing for the practical reasons stated above. Any other assumptions would require the design of a licensed professional.

The equation development will again follow the logic and assumptions of Section D.300.1.D.2 and will not be repeated here. Thus, two adjacent openings will be considered, with the center post receiving the largest concentrated load. All three post locations will have their foundation sized based on that center post, thus introducing a degree of conservatism.

The equations for the exterior foundation footing width are identical to those of the individual concepts for the Type **E5** (**E6** uses **E5**) and **E7** already developed for a continous marriage wall, and will not be repeated here.

3. *Roof load to a center post between two large marriage wall openings*: The given situation, illustrated in the roof plan of Figure D-9I, shows two adjacent marriage wall openings that follows the assumption of openings being a multiple of the transverse girder spacing; one opening twice the width of the other, hence x =2s and x_1=s The tributary area of gravity loads carried by the center post as the concentrated load (P1) is as follows:

$$P1 = [(Pf + 19.7) \times Wt + 19.8] \times \left(\frac{x + x_1}{2}\right) + 32$$

(snow+roofDL+LL) (ridge bm) (postDL)

4. *Floor load to a center post betweeen two large marriage wall openings*: Refering to the floor plan of Figure D-9I, the tributary area illustrated produces the concentrated gravity load (P2) to the foundation below the post as follows:

$$P2 = [(40 + 13) \times Wt + 10 + 18] \times \left(\frac{2}{3}\right) \times \left(\frac{x + x_1}{2}\right) + 20 \times Wt$$

(floorLL+DL) (floor bm) (two chassis bms) (transverse girder wt)

Note: The 2/3rds factor in the above equation is to account for an average floor load situation as illustrated in Figure D-9I.

5. *Total concentrated load (Rpm) to the foundation at a center post location:* The roof and floor loads combine to produce the total reaction (Rpm) to the foundation pier or column as follows:

$$Rpm = P1 + P2$$

6. *Required, adjacent opening center post location, marriage wall pier footing for Foundation Concept Type* ***E5*** *and* ***E6***. The footing area (Aftg) must be large enough so that the net allowable soil bearing pressure (Pso) is not exceeded under the full gravity dead, live and snow loads. Note that the pier and footing weight become additional dead load. The required footing area:

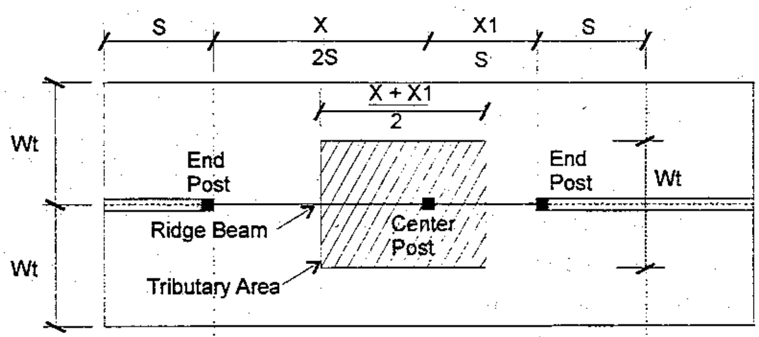
$$Aftg_{mar} = \frac{Rpm + 588}{Pso}$$

7. *Required, adjacent opening center post location, marriage wall pier footing for Foundation Concept*

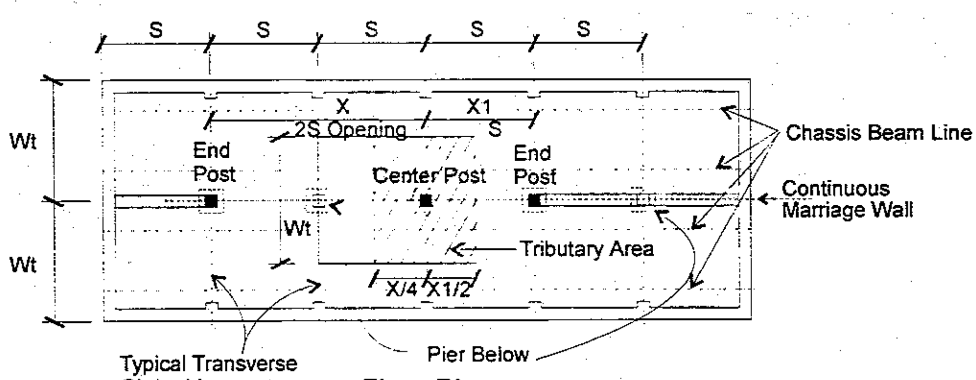
*footing for Foundation Concept Type **E7**.* The footing area (Aftg) must be large enough so that the net allowable soil bearing pressure (Pso) is not exceeded under the full gravity dead, live and snow loads. Note that the pier and footing weight become additional dead load. The required footing area:

$$\text{Aftg}_{\text{mar}} = \frac{\text{Rpm} + 1404}{\text{Pso}}$$

Note: A foundation pier or column exists, centered below the larger marriage wall opening (x = 2×s) at a transverse girder line. The pier or column footing here should be sized only for the floor concentrated load (P2). Substitute (P2) for (Rpm) in the above two equations. This is left to the engineer and is not reflected in the Tables of Appendix B.



Roof Plan



Floor Plan

Type E5, E6 and E7 - Marriage Wall w/Two Adjacent Openings

Figure D - 9I

D-300.2 REQUIRED VERTICAL ANCHORAGE BASED ON WIND IN THE TRANSVERSE DIRECTION. Refer to Figures D-10 to D-12 for the free-body diagrams of the superstructure and foundation, illustrating the overturning forces due to wind and the element dead loads providing resistance. The foundation Types **C**, **C1**, **E** and **I** are included for single-section units. Figure D-4, D-5 and D-6 are also related to the equation development of this section. For allowable stress design methodology, the load combination from ASCE 7-93 is: (Wind) - DL.

A. Wind Load Considerations for the Type C Single-Section Unit.

1. *General:* The superstructure receives external and internal wind pressures or suctions (p) on the two walls and two sloping roof planes in accordance with the equation of section D-200.4.C.2. These wind pressures tend to overturn the superstructure, rotating it about the pivot point at the bottom of the chassis beam as shown on Figure D-10. The vertical anchorage force (Av) necessary to prevent this uplift action is located at the opposite foundation pier. The anchorage connection of superstructure to foundation must be capable of transferring the (Av) force to the pier. The dead load of the pier, footing and soil overburden must be equal to or greater than the (Av) force to keep the superstructure from overturning.

2. *Wind Loads on the Superstructure:* As shown in Figure D-10, the resultant wind force at the top and bottom of the wall are (Pt) and (Pb) respectively. The vertical component of the resultant wind force on the windward and leeward slope are (Pvw) and (Pvl) respectively. They are calculated as follows:

$$Pt = Pb = \left(p_{ww} + |p_{wl}|\right) \times \frac{h_n}{2}$$

$$Pvw = p_{rw} \times \frac{Wt}{2}$$

$$Pvl = p_{rl} \times \frac{Wt}{2}$$

3. *Overturning Moment of the Superstructure:* The resultant wind loads on each surface rotate about the pivot point shown in Figure D-10. The summation of the force times distance values define the equation:

$$Mo = Pt \times (h_n + 0.833) + |Pvw| \times \left(\frac{3 \times Wt}{4} - dc\right) + |Pvl| \times \left(\frac{Wt}{4} - dc\right) + Pb \times (0.833)$$

4. *Resisting moment of the superstructure:* The total dead load provides the only gravity load resistance to overturning. Using the light dead load from section D-200.1.B:

$$Mr = DL \times \left(\frac{Wt}{2} - dc\right)$$

5. *Required Vertical Anchorage Force:* If the overturning moment (Mo) exceeds the resisting moment (Mr), an uplift force exists. The ASCE 7-93 restricts the usable dead load to 2/3rds of the actual dead load. This is the same as inverting

the ratio and making the overturning moment 3/2 times the calculated value. Thus, the final equation for (Av) at a specific pier spacing is:

$$Av = \left[\frac{1.5 \times Mo - Mr}{Wt - 2 \times dc}\right] \times \text{spacing}$$

B. Wind Load Considerations for the Type C1 Single-Section Unit.

1. *General*: The same wind pressures as for Type **C** tend to overturn the superstructure, rotating it about the pivot point at the bottom of the chassis beam as shown on Figure D-10. The vertical anchorage force (Av) necessary to prevent this uplift action is a tie-down strap that wraps over the roof of the unit and down the side walls to anchorage below grade at concrete deadmen. The capacity of the steel straps must be capable of transferring the (Av) force to the deadman. The dead load of the concrete deadman and soil overburden must be equal to or greater than the (Av) force to keep the superstructure from overturning.

2. *Wind Loads on the Superstructure*: As shown in Figure D-10, the resultant wind forces are the same as for the Type **C** single-section unit. See equations in section D-300.2.A.2.

3. *Overturning Moment of the Superstructure*: The resultant wind loads on each surface rotate about the pivot point shown in Figure D-10. The equation is the same as for the Type **C** single-section unit.

4. *Resisting moment of the superstructure*: The resisting moment is the same as for the Type **C** single-section unit.

5. *Required Vertical Anchorage Force*: The final equation for (Av) at a specific vertical tie-down strap or tie spacing is:

$$Av = \left[\frac{1.5 \times Mo - Mr}{Wt - dc}\right] \times \text{spacing}$$

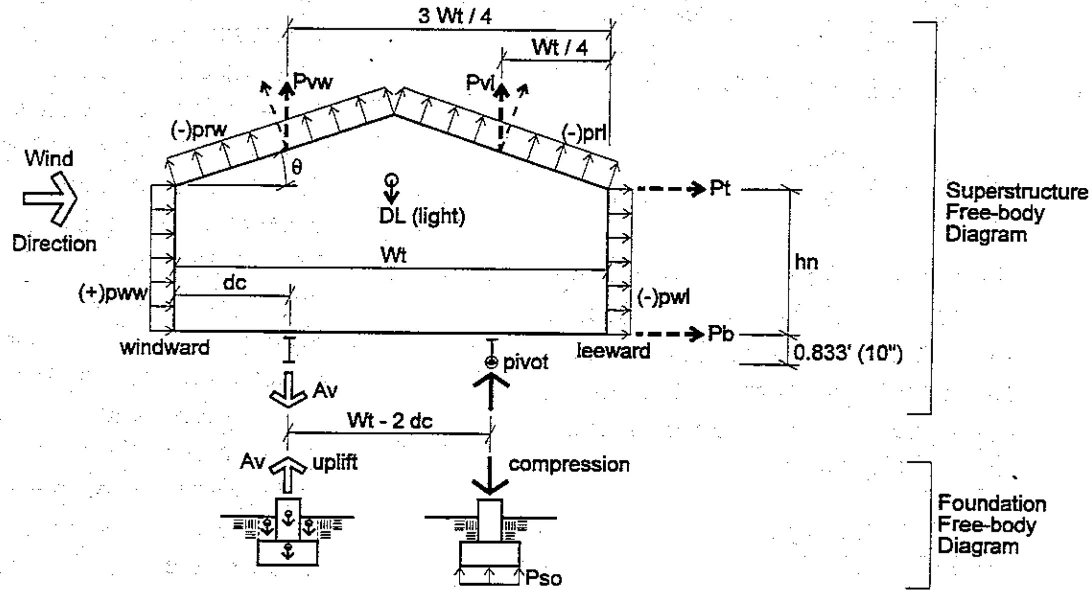
C. Wind Load Considerations for the Type E Single-Section Unit (excluding Types E3 and E4, which follows).

1. *General*: The applied wind loads to the superstructure are the same as for the Type **C** single-section unit. These wind pressures tend to overturn the superstructure, rotating it about the pivot point at the exterior foundation wall as shown in Figure D-11A. The vertical anchorage force (Av) necessary to prevent this uplift action is located at the opposite exterior foundation wall. The anchorage connection of superstructure to foundation must be capable of transferring the (Av) force to the wall. The dead load of the wall, footing and soil overburden must be equal to or greater than the (Av) force to keep the superstructure from overturning.

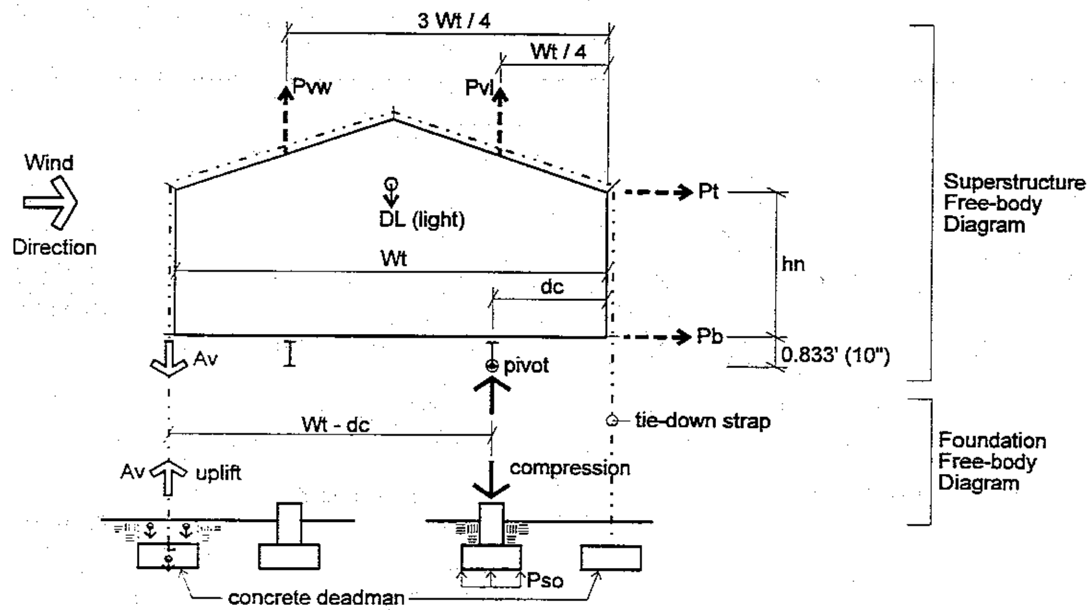
2. *Wind Loads on the Superstructure*: Same as for the Type **C** single-section unit. The equations are shown is section D-300.2.A2.

3. *Overturning Moment of the superstructure*: The resultant wind loads on each surface rotate about the pivot point shown in Figure D-11A.

The summation of the force times distance values define the equation:



Type C Single-Section Unit



Type C1 Single-Section Unit

Wind Related Overturning Loads - Transverse Direction

Figure D - 10

$$Mo = Pt \times h_n + |Pvw| \times \left(\frac{3 \times Wt}{4}\right) + |Pvl| \times \left(\frac{Wt}{4}\right)$$

4. *Resisting moment of the superstructure*: The total dead load provides the only gravity load resistance to overturning. Using the light dead load from section D-200.1.B:

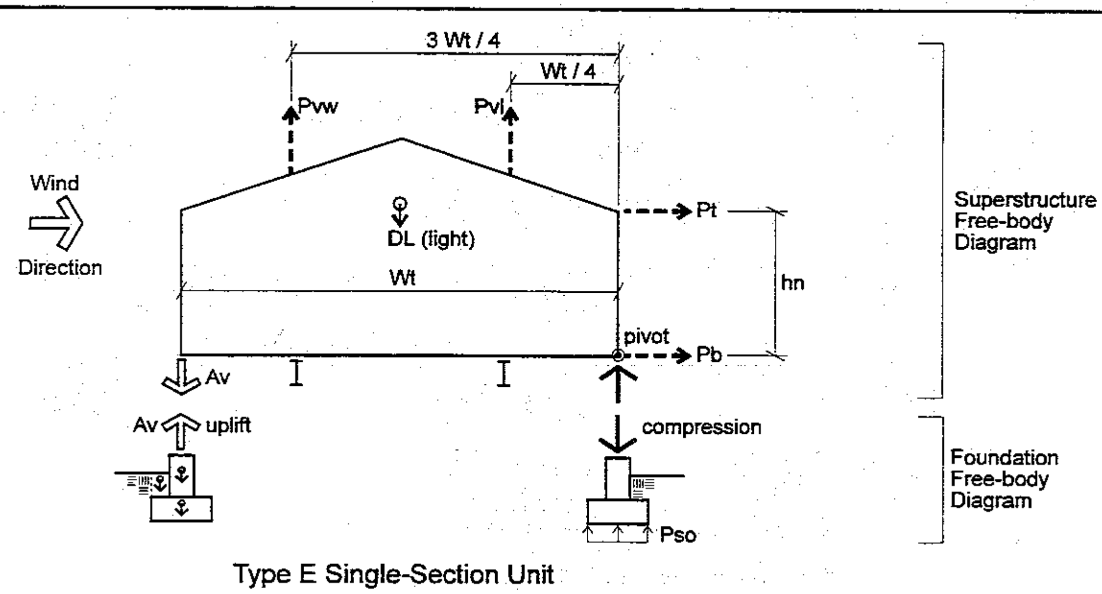
$$Mr = DL \times \left(\frac{Wt}{2}\right)$$

5. *Required Vertical Anchorage Force*: Similar to Section D-300.A.5. Thus, the final equation for anchorage force (Av) to be transferred to the exterior foundation wall becomes:

$$Av = \frac{1.5 \times Mo - Mr}{Wt}$$

C-X. Wind Load Considerations for the Type E3 and E4 Single-Section Unit.

1. *General*: The applied wind loads to the superstructure are the same as for the Type **C** single-section unit as shown in Figure D-10. These wind pressures tend to overturn the superstructure, rotating it about the pivot point at the exterior foundation wall as shown in Figure D-11B. The vertical anchorage force (Av) necessary to prevent this uplift action is located at the two chassis beam piers and the opposite exterior foundation wall. The anchorage connection of superstructure to these piers and foundation wall must be capable of transferring the (Av) force in proportion to their distance from the pivot. The dead load of the exterior wall, footing and soil overburden; plus the dead load of the two piers, footings and soil overburden must all be equal to



Wind Related Overturning Loads - Transverse Direction

Figure D - 11A

or greater than the portion of the (Av) force each must resist to keep the superstructure from overturning.

2. *Wind Loads on the Superstructure*: Same as for the Type **C** single-section unit. The equations are shown is section D-300.2.A2.

3. *Overturning Moment of the superstructure*: The resultant wind loads on each surface rotate about the pivot point shown in Figure D-11B. The summation of the force times distance values define the equation:

$$Mo = Pt \times h_n + |Pvw| \times \left(\frac{3 \times Wt}{4}\right) + |Pvl| \times \left(\frac{Wt}{4}\right)$$

4. *Resisting moment of the superstructure*: The total dead load provides the only gravity load resistance to overturning. Using the light dead load from section D-200.1.B:

$$Mr = DL \times \left(\frac{Wt}{2}\right)$$

5. *Required vertical Anchorage Force*: Assuming the anchorage force at the exterior wall to be (Av), and using triangle proportions, the intermediate vertical anchorage force at the furthest pier from the pivot (Av_1) becomes:

$$Av_1 = \left(\frac{Wt - dc}{Wt}\right) \times Av$$

Note: As illustrated in Figure D-11B the anchorage force at the pier closest to the pivot is very small and is ignored. The anchorage force (Av_1) shall be used at both piers for conservatism.

The resisting moment created by these two anchorage locations is:

$$M_{AV} = Av \times Wt + Av_1 \times (Wt - dc)$$

Substitution of the anchorage force value (Av_1) into the above equation results in the following:

$$M_{AV} = Av \times \left[Wt + \frac{(Wt - dc)^2}{Wt} \right]$$

Since the anchorage moment (M_{AV}) must balance the net overturning moment (1.5 x Mo-Mr), the maximum vertical anchorage force (Av), which is used in the Foundation Design Load Tables of Appendix B, Part 2 for the exterior wall per foot of length, becomes:

$$Av = \frac{(1.5 \times Mo - Mr)}{\left[Wt + \frac{(Wt - dc)^2}{Wt} \right]}$$

Note that the vertical anchorage force (Av_1) used in the Tables for anchorage at both piers under the chassis beams is based on pier spacing (s) and renamed (Av_{1pier}) in the equation becomes:

$$Av_{1pier} = \left(\frac{Wt - dc}{Wt}\right) \times Av \times spacing$$

D. Wind Load Considerations for the Type I Single-Section Unit.

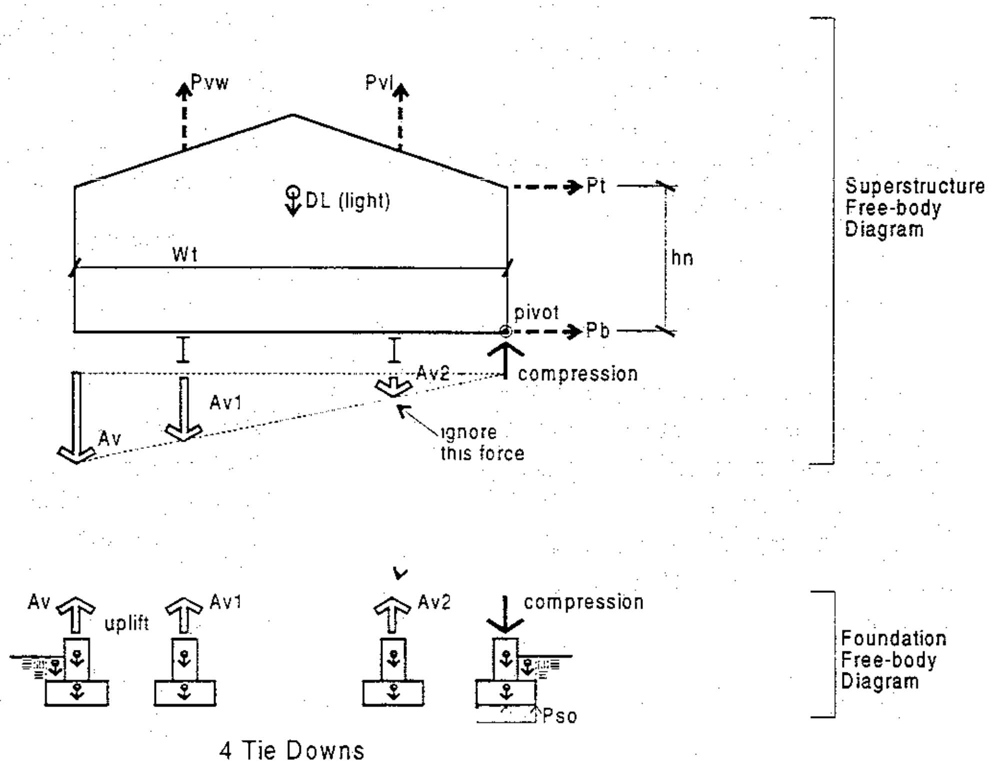
1. *General*: The applied wind loads to the superstructure are the same as for the Type **C** and **E** single-section unit. These wind pressures tend to overturn the superstructure, rotating it about the pivot point at the exterior foundation wall as shown in Figure D-12. The vertical anchorage force (Av) necessary to prevent this uplift action is located at the far side chassis beam at the interior pier spacing. The anchorage connection of superstructure to foundation must be capable of transferring the (Av) force to the pier. The dead load of the wall, footing and soil overburden must be equal to or greater than the (Av) force to keep the superstructure from overturning.

2. *Wind Loads on the Superstructure*: Same as for the Type **C** single-section unit. The equations are shown is section D-300.2.A.2.

3. *Overturning Moment of the superstructure*: Same as for the Type **E** single-section unit.

$$Mo = Pt \times h_n + |Pvw| \times \left(\frac{3 \times Wt}{4}\right) + |Pvl| \times \left(\frac{Wt}{4}\right)$$

4. *Resisting moment of the superstructure*: Same as for the Type **E** sin-



Type E3 and E4 Single-Section Unit

Wind Related Overturning Loads - Transverse Direction

Figure D - 11B

gle-section unit.

$$Mr = DL \times \left(\frac{Wt}{2}\right)$$

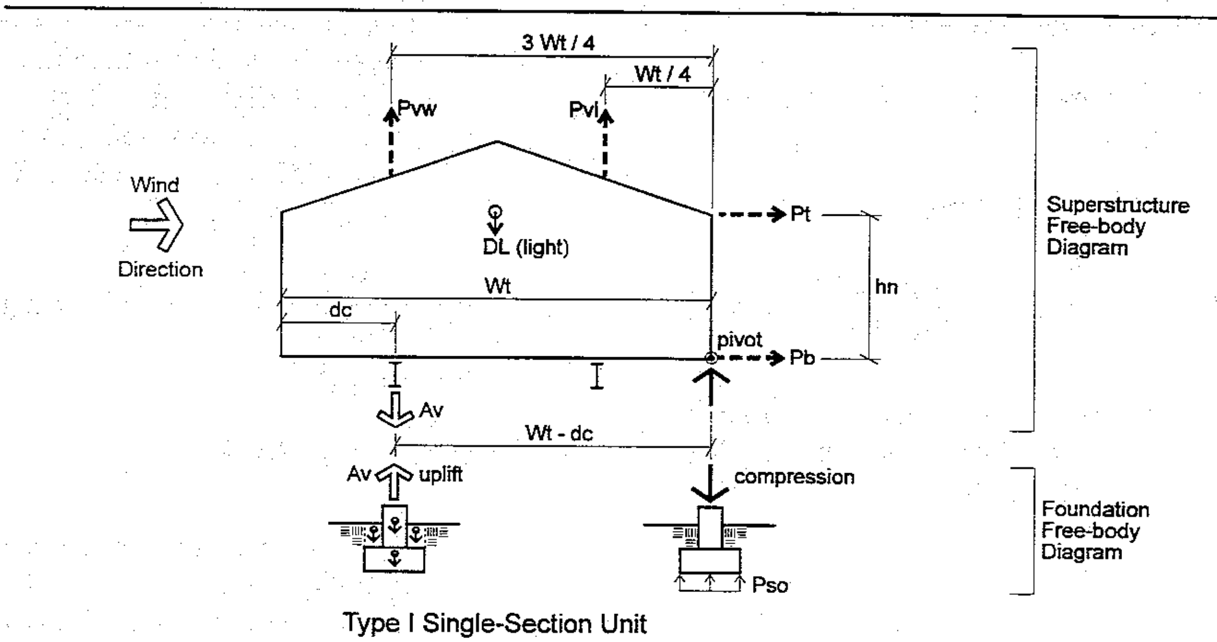
5. *Required Vertical Anchorage Force*: Similar to Section D-300.A.5. Thus, the final equation for anchorage force (Av) to be transferred to the exterior foundation wall becomes:

$$Av = \left[\frac{(1.5 \times Mo - Mr)}{(Wt - dc)}\right] \times spacing$$

E. Wind Load Considerations for a Type C Multi-Section Unit.

1. *General*: The superstructure is assumed to behave as a single box for overturning. It receives wind loads and tends to overturn in a similar manner to the single-section unit as described in Section D-300.2.A.1. The pivot point is under the exterior chassis beam on one side. Anchorage connection of superstructure to foundation is either two tie-downs or four tie-downs as illustrated in Figure D-13 at the other chassis beams.

2. *Wind Loads on the Superstructure*: As shown in Figure D-13, the resultant wind force at the top and bottom of the wall are (Pt) and (Pb) respectively. The vertical component of the resultant wind force on the windward and leeward slope are (Pvw) and (Pvl) respectively. They are calculated as follows:



Wind Related Overturning Loads - Transverse Direction

Figure D - 12

$$Pt = Pb = \left(p_{ww} + |p_{wl}|\right) \times \frac{h_n}{2}$$

$$Pvw = p_{rw} \times Wt$$

$$Pvl = p_{rl} \times Wt$$

3. *Overturning Moment of the Superstructure*: The summation of the force times distance values defines the equation:

$$Mo = Pt \times (h_n + 0.833) + |Pvw| \times \left(\frac{3 \times Wt}{2} - dc\right) + |Pvl| \times \left(\frac{Wt}{2} - dc\right) + Pb \times (0.833)$$

4. *Resisting Moment of the Superstructure*: The total dead load provides the only gravity load resistance to overturning. Using the light dead load for a multi-section unit from section D-200.1.B:

$$Mr = DL \times (Wt - dc)$$

5. *Required Vertical Anchorage Force*:

   a. **Two tie-downs**:

$$Av = \left[\frac{(1.5 \times Mo - Mr)}{2 \times (Wt - dc)}\right] \times spacing$$

   b. **Four tie-downs**: by triangle proportions the intermediate vertical anchorage forces (Av) are:

$$Av_1 = \left[\frac{Wt}{2 \times (Wt - dc)}\right] \times Av$$

$$Av_2 = \left[\frac{Wt - 2 \times dc}{2 \times (Wt - dc)}\right] \times Av$$

The resisting moment created by the three anchorage locations is:

$$M_{AV} = Av_1 \times Wt + Av_2 \times (Wt - 2 \times dc) + Av \times 2 \times (Wt - dc)$$

Substitution of the anchorage force values into the above equation results in the following:

$$M_{AV} = Av \times \left[\frac{(Wt - 2 \times dc)^2}{2 \times (Wt - dc)} + \frac{(Wt)^2}{2 \times (Wt - dc)} + 2 \times (Wt - dc)\right]$$

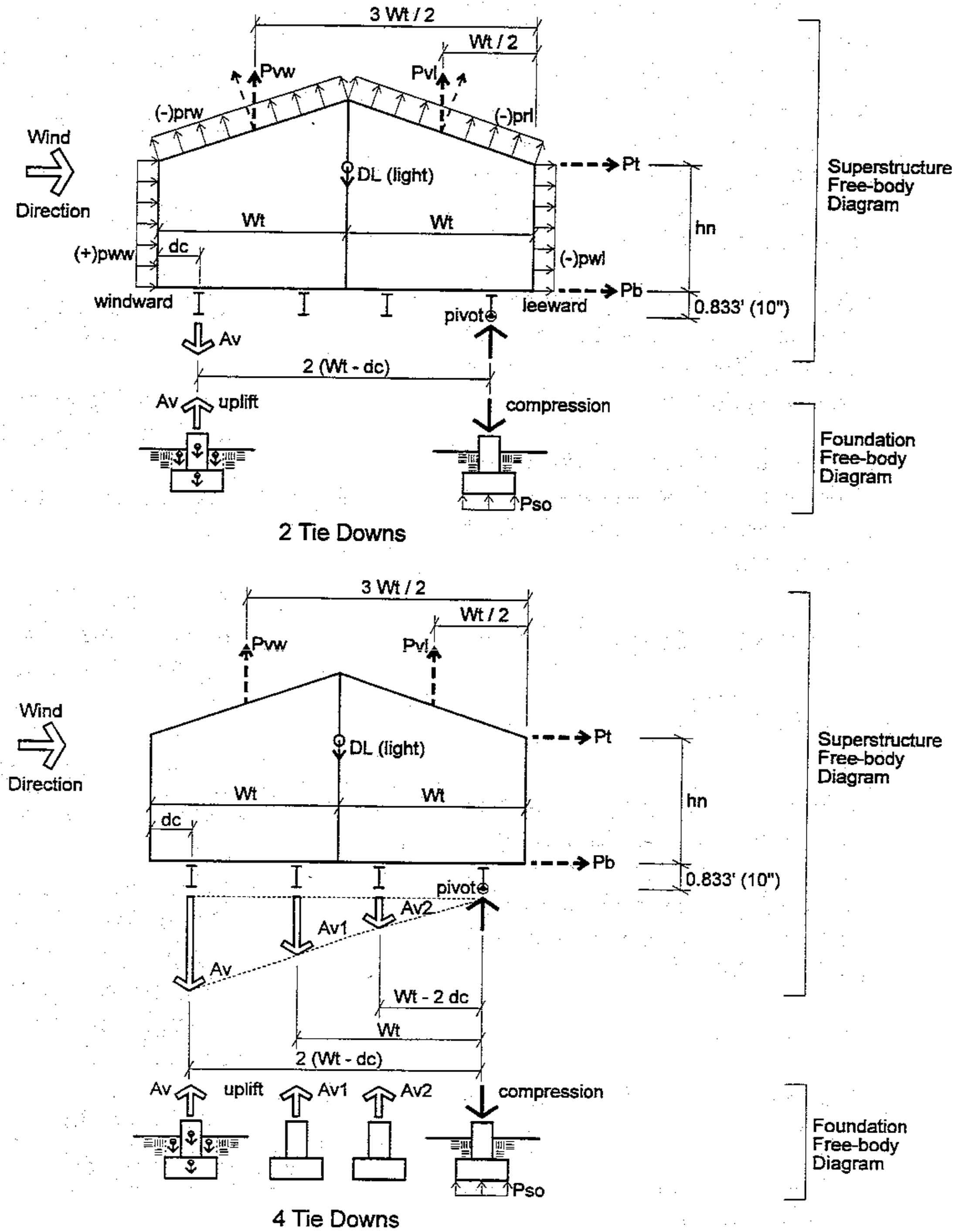
Since the anchorage moment (M_{AV}) must balance the net overturning moment (1.5 x Mo-Mr), the maximum vertical anchorage force (Av) concentrated at the exterior pier, which is used in the Foundation Design Load Tables of Appendix B, Part 2, becomes:

$$Av = \frac{(1.5 \times Mo - Mr)}{\left[\frac{(Wt - 2 \times dc)^2}{2 \times (Wt - dc)} + \frac{(Wt)^2}{2 \times (Wt - dc)} + 2 \times (Wt - dc)\right]} \times spacing$$

Note that the smaller vertical anchorage forces (Av_1) and (Av_2) are **not** used in the Tables.

F. Wind Load Considerations for a Type E Multi-Section Unit.

1. *General*: The pivot point is located at the exterior foundation wall on



Wind Related Overturning Loads: Type C - Multi-Section Unit - Transverse Direction

Figure D - 13

one side. Anchorage connection of superstructure to foundation is accomplished at the opposite exterior wall and at specific pier locations resulting in either two tie-downs or four tie-downs as illustrated in Figure D-14. Foundation Concept Type E3 has six tie-downs. The illustration would be similar to that for four tie-downs; however, the calculations are included here.

2. *Wind Loads on the Superstructure*: Wind loads on the walls and roof planes are the same as for the Type **C** multi-section unit.

3. *Overturning Moment of the Superstructure*: The summation of the force times distance values defines the equation:

$$Mo = Pt \times h_n + |Pvw| \times \left(\frac{3 \times Wt}{2}\right) + |Pvl| \times \left(\frac{Wt}{2}\right)$$

4. *Resisting Moment of the Superstructure*: The total dead load provides the only gravity load resistance to overturning. Using the light dead load for a multi-section unit from section D-200.1.B:

$$Mr = DL \times Wt$$

5. *Required Vertical Anchorage Force*:

   a. **Two tie-downs**: at the exterior wall in lbs/ft:

$$Av = \left[\frac{(1.5 \times Mo - Mr)}{2 \times Wt}\right]$$

   b. **Four tie-downs:** by triangle proportions the intermediate vertical anchorage forces (Av), also in lbs/ft, are:

$$Av_1 = \left[\frac{Wt + dc}{2 \times Wt}\right] \times Av$$

$$Av_2 = \left[\frac{Wt - dc}{2 \times Wt}\right] \times Av$$

The resisting moment created by the three anchorage locations is:

$$M_{AV} = Av_1 \times (Wt + dc) + Av_2 \times (Wt - dc) + Av \times 2 \times Wt$$

Substitution of the anchorage force values into the above equation results in the following:

$$M_{AV} = Av \times \left[\frac{(Wt + dc)^2}{2 \times Wt} + \frac{(Wt - dc)^2}{2 \times Wt} + 2 \times Wt\right]$$

Since the anchorage moment (M_{AV}) must balance the net overturning moment ($1.5 \times Mo - Mr$), the maximum vertical anchorage force (Av) at the exterior wall in lbs/ft, becomes:

$$Av = \frac{(1.5 \times Mo - Mr)}{\left[\frac{(Wt + dc)^2}{2 \times Wt} + \frac{(Wt - dc)^2}{2 \times Wt} + 2 \times Wt\right]}$$

And the next largest anchorage force (in lbs.) (Av_1) at the first interior pier becomes:

$$Av_1 = \left[\frac{Wt + dc}{2 \times Wt}\right] \times Av \times spacing$$

b. **Six tie-downs:** by triangle proportions the intermediate vertical anchorage forces (Av), in lbs/ft of unit length, are:

$$Av_1 = \left[\frac{2 \times Wt - dc}{2 \times Wt}\right] \times Av$$

$$Av_2 = \left[\frac{Wt + dc}{2 \times Wt}\right] \times Av$$

$$Av_3 = \left[\frac{Wt - dc}{2 \times Wt}\right] \times Av$$

The resisting moment created by the four anchorage locations is:

$$M_{AV} = Av_1 \times (2 \times Wt + dc) + Av_2 \times (Wt + dc) + Av_3 \times (Wt - dc) + Av \times 2 \times Wt$$

Substitution of the anchorage force values into the above results in the following:

$$M_{AV} = Av \times \left[\frac{(2 \times Wt - dc)^2}{2 \times Wt} + \frac{(Wt + dc)^2}{2 \times Wt} + \frac{(Wt - dc)^2}{2 \times Wt} + 2 \times Wt\right]$$

Since the anchorage moment (M_{AV}) must balance the net overturning moment ($1.5 \times$ Mo-Mr), the maximum vertical anchorage force (Av) at the exterior wall in lbs/ft, becomes:

$$Av = \frac{(1.5 \times Mo - Mr)}{\left[\frac{(2 \times Wt - dc)^2}{2 \times Wt} + \frac{(Wt + dc)^2}{2 \times Wt} + \frac{(Wt - dc)^2}{2 \times Wt} + 2 \times Wt\right]}$$

And the next largest anchorage force (in lbs.) (Av_1) at the first interior pier becomes:

$$Av_1 = \left[\frac{2 \times Wt - dc}{2 \times Wt}\right] \times Av \times spacing$$

The smaller values of Av are not printed in the tables for fabrication economy.

G. Wind Load Considerations for a Type I Multi-Section Unit.

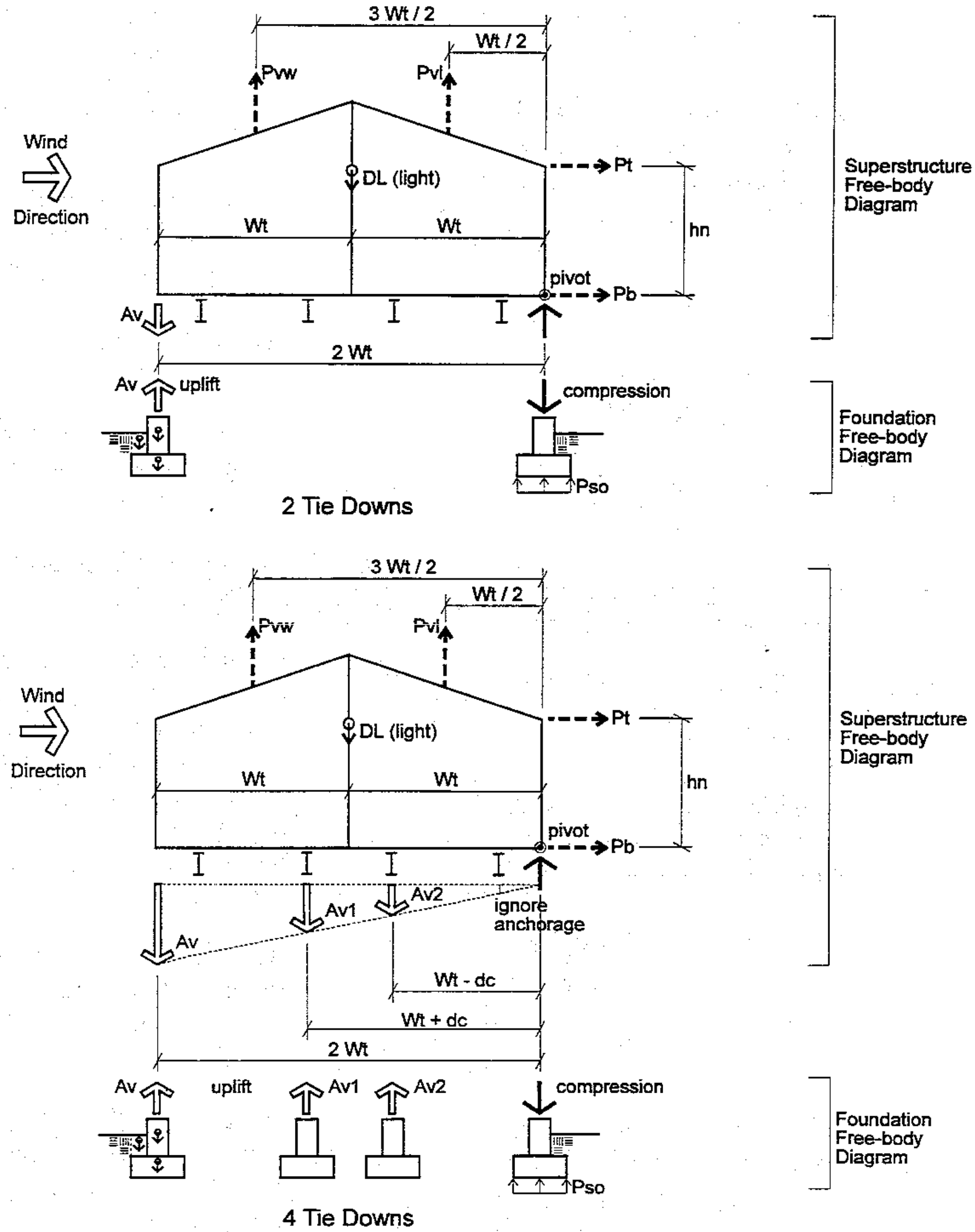
1. *General*: The pivot point is located at the exterior foundation wall on one side. Anchorage connection of superstructure to foundation is accomplished at specific pier locations resulting in either two tie-downs or four tie-downs as illustrated in Figure D-15.

2. *Wind Loads on the Superstructure*: Wind loads on the walls and roof planes are the same as for the Type **C** or **E** unit.

3. *Overturning Moment of the Superstructure*: The summation of the force times distance values defines the equation:

$$Mo = Pt \times h_n + |Pvw| \times \left(\frac{3 \times Wt}{2}\right) + |Pvl| \times \left(\frac{Wt}{2}\right)$$

4. *Resisting Moment of the Superstructure*: The total dead load provides the only gravity load resistance to



Wind Related Overturning Loads: Type E - Multi-Section Unit - Transverse Direction

Figure D - 14

overturning. Using the light dead load for a multi-section unit from section D-200.1.B:

$$Mr = DL \times (Wt)$$

5. *Required Vertical Anchorage Force*:

   a. **Two tie-downs:** Concentrated load in lbs. at the exterior pier becomes:

$$Av = \left[\frac{(1.5 \times Mo - Mr)}{(2 \times Wt - dc)}\right] \times spacing$$

   b. **Four tie-downs:** by triangle proportions the intermediate vertical anchorage forces (Av) are similar to the Type **E** multi-section unit.

   The resisting moment created by the three anchorage locations is:

$$M_{AV} = Av_1 \times (Wt + dc) + Av_2 \times (Wt - dc) + Av \times (2 \times Wt - dc)$$

   Substitution of the anchorage force values into the above equation results in the following:

$$M_{AV} = Av \times \left[\frac{(Wt+dc)^2}{(2 \times Wt - dc)} + \frac{(Wt - dc)^2}{(2 \times Wt - dc)} + (2 \times Wt - dc)\right]$$

   Since the anchorage moment (M_{AV}) must balance the net overturning moment ($1.5 \times Mo$ - Mr), the maximum vertical anchorage force (Av) at the exterior pier, used in the Foundation Design Load Tables of Appendix B, Part 2, becomes:

$$Av = \frac{(1.5 \times Mo - Mr)}{\left[\frac{(Wt+dc)^2}{(2 \times Wt - dc)} + \frac{(Wt - dc)^2}{(2 \times Wt - dc)} + (2 \times Wt - dc)\right]} \times spacing$$

   And the next largest anchorage force (Av_1) at the first interior pier becomes:

$$Av_1 = \left[\frac{Wt + dc}{(2 \times Wt - dc)}\right] \times Av$$

   This (Av_1) force equation is **not** used in the Foundation Design Load Tables of Appendix B. It is shown here for engineers who wish to reduce the design (Av) force at interior pier locations.

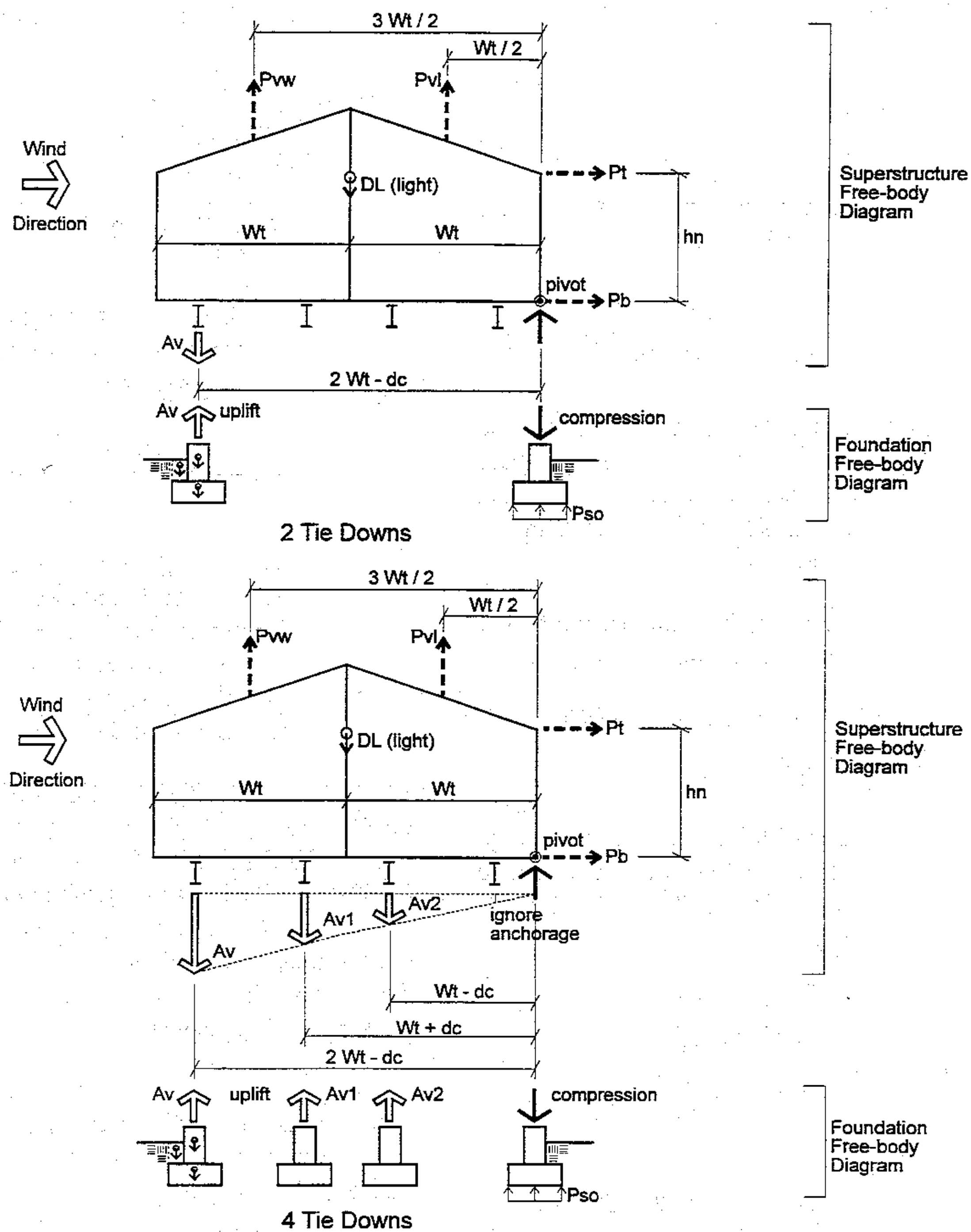
D-300.3 REQUIRED VERTICAL ANCHORAGE BASED ON SEISMIC IN THE TRANSVERSE DIRECTION. Refer to Figure D-16 to D-18 for the free-body diagrams of the superstructure and foundation for single-section units, illustrating the overturning forces due to seismic activity and the element dead loads providing resistance.

A. General. The seismic provisions of ASCE 7-93 are a limit state methodology that must be modified to an allowable stress methodology for comparison to wind. This is accomplished in the load combination as follows:

The basic load combination from ASCE 7-93:

E - DL

The seismic equation from ASCE



Wind Related Overturning Loads: Type I - Multi-Section Unit - Transverse Direction

Figure D - 15

7-93:

$$E = \pm Q_E \pm 0.5 \times \underline{A}v \times DL$$

where Q_E is the effect of horizontal seismic. Substitution of E into the basic equation:

$$Q_E - D \times (1 - 0.5 \times \underline{A}v)$$

Thus, for Seismic the net overturning equation generalizes to:

$$1.5 \times Mo - (1 - 0.5 \times \underline{A}v) \times Mr$$

which includes the same 1.5 factor of safety as used for wind.

B. Seismic Force Consideration for the Type C, C1, E or I Single-Section Units.

1. *Seismic Inertia Forces on the Superstructure*: Determination of the horizontal forces was explained in section D-200.5. The "heavy" component dead loads were used to arrive at the inertia forces F_{xr} and F_{xf}.

2. *Overturning Moment of the Superstructure*: The moment components are force times distance from the pivot.

   a. For Type **C** and **C1** single-section units:

$$Mo = F_{xr} \times (h_n + 0.833) + F_{xf} \times 0.833$$

   b. For Type **E** or **I** single-section units:

$$Mo = F_{xr} \times h_n$$

3. *Resisting moment of the Superstructure*: The dead load and snow load, where applicable, constitute the gravity load resisting overturning. The "light" unit dead load was used for overturning resistance to be conservative, even though the "heavy" dead loads for single-section units were used for the calculation of the floor and roof inertia forces. Generally the equations become:

   a. For Type **C** and **C1** single-section units:

$$Mr = \left(DL + \%P_f \times Wt\right) \times \left(\frac{Wt}{2} - dc\right)$$

   b. For Type **E** and **I** single-section units:

$$Mr = \left(DL + \%P_f \times Wt\right) \times \left(\frac{Wt}{2}\right)$$

4. *Required Vertical Anchorage Force*: Using the general equation described in section D-300.3.A and using the appropriate Mo and Mr equarions for each unit Type, the equations become:

   a. For Type **C** single-section units:

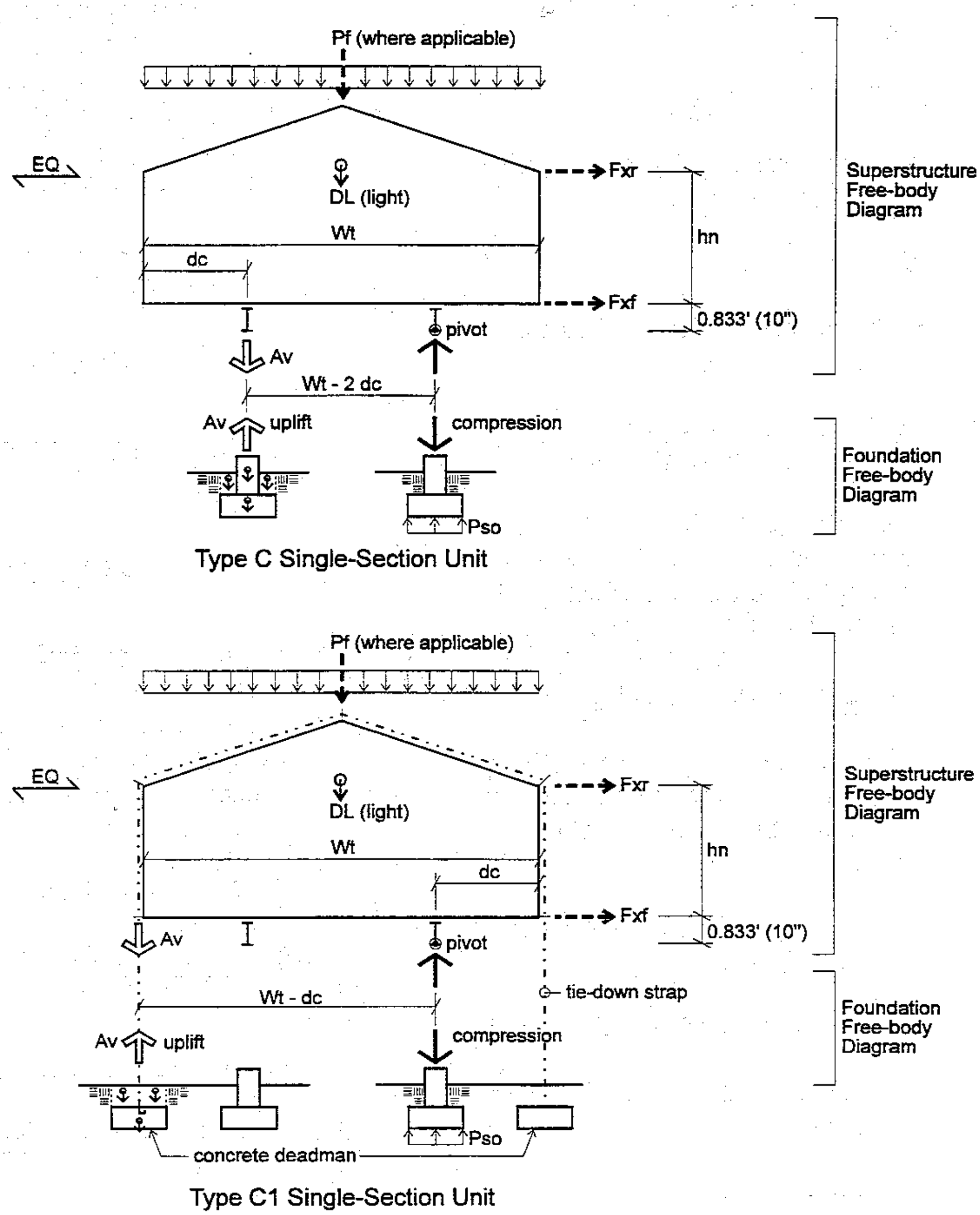
$$Av = \left[\frac{1.5 \times Mo - (1 - 0.5 \times Av) \times Mr}{Wt - 2 \times dc}\right] \times \text{spacing}$$

   b. For Type **C1** single-section unit:

$$Av = \left[\frac{1.5 \times Mo - (1 - 0.5 \times Av) \times Mr}{Wt - dc}\right] \times \text{spacing}$$

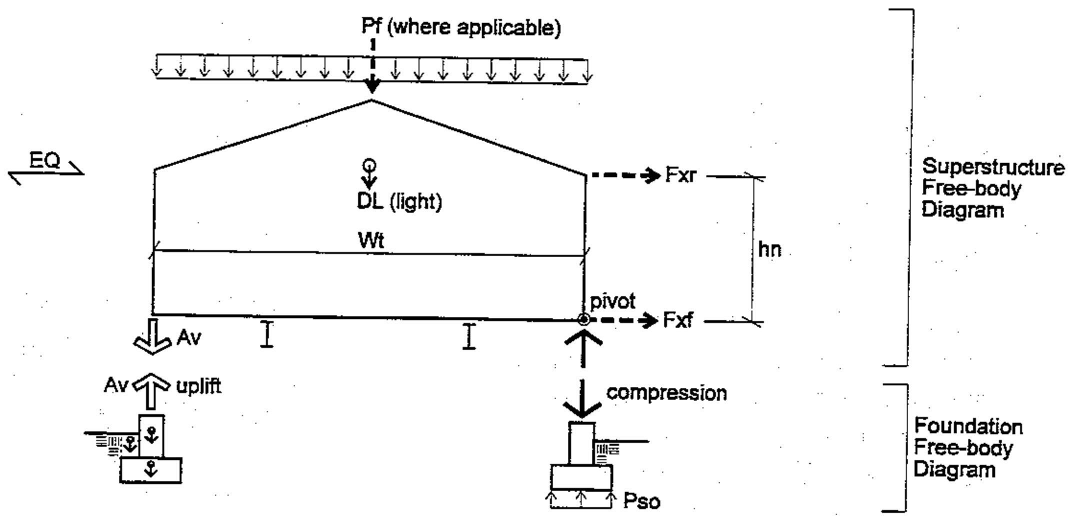
c. For Type **E** single-section units:

$$Av = \left[\frac{1.5 \times Mo - (1 - 0.5 \times Av) \times Mr}{Wt}\right]$$



Seismic Related Overturning Loads - Transverse Direction

Figure D - 16



Type E Single-Section Unit

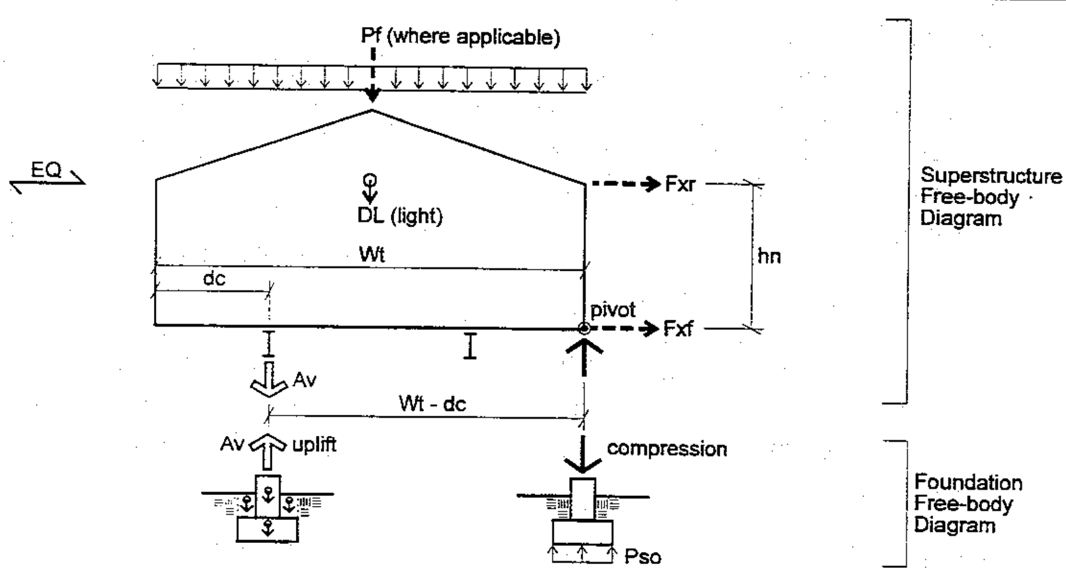
Seismic Related Overturning Loads - Transverse Direction

Figure D - 17

d. For Type **I** single-section units:

$$Av = \left[\frac{1.5 \times Mo - (1 - 0.5 \times Av) \times Mr}{Wt - dc} \right] \times spacing$$

5. *Comparison: Wind vs. Seismic*: The values for the vertical anchorage force (Av), based on overturning in the transverse direction, will be the larger value of wind or seismic.



Type I Single-Section Unit

Seismic Related Overturning Loads - Transverse Direction

Figure D - 18

This is reflected in the Foundation Design Load Tables of Appendix B, Part 2. **Note**: should any (Av) value become negative, there is no uplift.

C. Seismic Force Considerations for the Type C, E and I Multi-Section Units.

1. *General*: The moment equilibrium equations for anchorage resistance are similar to those for the multi-section units subjected to wind load as shown in Figures D-13 to D-15. The applied roof and floor inertia forces (F_{xr} and F_{xf} respectively) are based on heavy dead loads for multi-section units and positioned where shown in Figures D-16 to D-18. Calculation of the horizontal roof and floor forces was explained in section D-200.5.

2. *Overturning Moment of the Superstructure*:

   a. For Type **C** multi-section units: Use the same equation found in section D-300.3.B.2.a., except calculate F_{xr} and F_{xf} from the expressions for Multi-Section units.

   b. For Type **E** or **I** multi-section units: Use the same equation found in section D-300.3.B.2.b., except calculate F_{xr} and F_{xf} from the expressions for Multi-Section units.

3. *Resisting Moment of the Superstructure*:

   a. For Type **C** multi-section units:

$$M_r = (DL + \%P_f \times 2 \times Wt) \times (Wt - dc)$$

   b. For Type **E** or **I** multi-section units:

$$M_r = (DL + \%P_f \times 2 \times Wt) \times Wt$$

4. *Required Vertical Anchorage Force*: Using the general equation described in section D-300.3.A the equations become:

   a. For Type **C** multi-section units: The concentrated force at the exterior pier for **two tie-downs**:

$$Av = \left[\frac{1.5 \times Mo - (1 - 0.5 \times Av) \times Mr}{2 \times (Wt - dc)}\right] \times \text{spacing}$$

   with **four tie-downs**: The maximum concentrated (Av) force that is used in the Foundation Design Load Tables of Appendix B, Part 2 is:

$$Av = \frac{(1.5 \times Mo - (1 - 0.5 \times Av) \times Mr)}{\left[\frac{(Wt - 2 \times dc)^2}{2 \times (Wt - dc)} + \frac{(Wt)^2}{2 \times (Wt - dc)} + 2 \times (Wt - dc)\right]} \times \text{spacing}$$

   Note: that the smaller vertical anchorage forces (Av_1) and (Av_2) derived in Section D-300.2.E.5 are not used in the tables. **Note**: negative values of (Av) produce no uplift.

   b. For Type **E** multi-section units, anchored at the exterior walls, the (Av) value is in units of lbs/ft. With **two tie-downs**:

$$Av = \left[\frac{1.5 \times Mo - (1 - 0.5 \times Av) \times Mr}{2 \times Wt}\right]$$

For **four tie-downs:** The maximum vertical anchorage force (Av) that is used in the Foundation Design Load Tables of Appendix B, Part 2 at the exterior wall in units of lbs/ft is:

$$Av = \frac{(1.5 \times Mo - (1 - 0.5 \times Av) \times Mr)}{\left[\frac{(Wt + dc)^2}{2 \times Wt} + \frac{(Wt - dc)^2}{2 \times Wt} + 2 \times Wt\right]}$$

The next largest anchorage force (Av_1 in lbs.) at the first interior pier as used in the Appendix B, Part 2 Tables becomes:

$$Av_1 = \left[\frac{Wt + dc}{2 \times Wt}\right] \times Av \times spacing$$

For **six tie-downs:** This condition exists only for the Type **E3** Foundation Concept. The maximum vertical anchorage force (Av) that is used in the Foundation Design Load Tables of Appendix B. Part 2 at the exterior wall in units of lbs/ft is:

$$Av = \frac{(1.5 \times Mo - (1 - 0.5 \times Av) \times Mr)}{\left[\frac{(2 \times Wt - dc)^2}{2 \times Wt} + \frac{(Wt + dc)^2}{2 \times Wt} + \frac{(Wt - dc)^2}{2 \times Wt} + 2 \times Wt\right]}$$

The next largest anchorage force (Av_1 in lbs.) at the first interior pier as used in the Appendix B, Part 2 Tables becomes:

$$Av_1 = \left[\frac{2 \times Wt - dc}{2 \times Wt}\right] \times Av \times spacing$$

c. For Type **I** multi-section units: The concentrated force at the exterior pier for **two tie-downs**:

$$Av = \left[\frac{1.5 \times Mo - (1 - 0.5 \times Av) \times Mr}{2 \times (Wt - dc)}\right] \times spacing$$

with **four tie-downs:** The maximum vertical anchorage force (Av in Lbs.) at the exterior pier, which is used in the Foundation Design Load Tables, Part 2, is:

$$Av = \frac{(1.5 \times Mo - (1 - 0.5 \times Av) \times Mr)}{\left[\frac{(Wt + dc)^2}{(2 \times Wt - dc)} + \frac{(Wt - dc)^2}{(2 \times Wt - dc)} + (2 \times Wt - dc)\right]} \times spacing$$

And the next largest anchorage force (Av_1) at the first interior pier is:

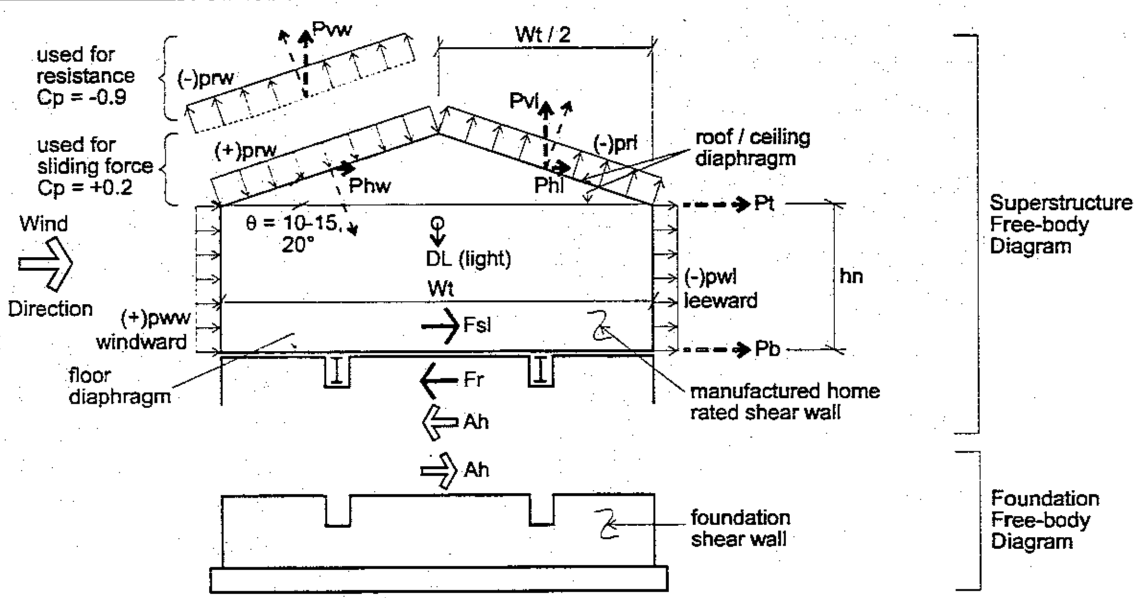
$$Av_1 = \left[\frac{Wt + dc}{(2 \times Wt - dc)}\right] \times Av$$

This (Av_1) force equation is **not** used in the Foundation Design Load Tables of Appendix B. It is shown here for engineers who wish to reduce the design (Av) force at interior pier locations.

D-300.4 REQUIRED HORIZONTAL ANCHORAGE BASED ON WIND IN THE TRANSVERSE DIRECTION. Refer to Figures D-19 and D-21 for the free-body diagrams of the superstructure and foundation for single section and multi-section units, illustrating the horizontal forces due to sliding and element dead loads providing resistance. Horizontal sliding is not influenced by the foundation Type C, E or I; thus the same analysis applies to all of the foundation types. Figure D-4, D-5 and D-6 are also related to the equation development of this section. A roof slope of 10-15° (20° also) (approx. 3 in 12 slope) was used so as to utilize the maximum exterior pressure coefficient on the windward slope (Cp = +0.2) to produce the largest horizontal windward force component, and thus the largest sliding force. An external windward slope uses a Cp=-0.9 to produce the smallest resistance force. These were conservative assumptions for the Tables. Note that internal pressures on the walls cancel; therefore, only internal pressures of +GCpi on the roof planes are considered (see Figure D-5). For allowable stress design methodology, the load combination from ASCE 7-93 is: (Wind - DL). Figure D-20 illustrates that a tributary width approach is used to calculate the forces to each foundation horizontal load resisting plane.

A. Wind Load Considerations for the Type C, E and I Single-Section Unit.

1. *General*: As shown in Figure D-19 the external wind pressure on the windward wall and the external suction on the leeward wall are transferred into the roof (plus ceiling) and floor diaphragms. The roof (plus ceiling) diaphragm transfers the force into superstructure shear walls perpendicular to the unit length, and then in turn to the floor diaphragm, assuming all connections are properly designed to resist the horizontal wind forces. From the floor diaphragm the horizontal



Type C, E or I Single-Section Units

Wind Related Sliding - Transverse Direction

Figure D - 19

force is transferred into the foundation shear wall or vertical X-bracing plane. Reference Figure 6-4. It is assumed that the location of superstructure shear walls coincides with the foundation shear wall locations.

2. *Wind Loads on the Superstructure*: As shown in Figure D-19, the resultant wind force at the top and bottom of the wall are (Pt) and (Pb) respectively. The vertical component of the resultant wind force on the windward and leeward slope are (Pvw) and (Pvl) respectively. The horizontal components of the roof wind loads both contribute to sliding, and are calculated as follows:

$$Pt = Pb = \left(p_{ww} + \left|p_{wl}\right|\right) \times \frac{h_n}{2}$$

$$Pvw = p_{rw} \times \frac{Wt}{2}$$

For calculation of (Fr) use (Cp) = -0.9 in the above equation.

$$Pvl = p_{rl} \times \frac{Wt}{2}$$

$$P_{HW} = p_{rw} \times \left(\frac{Wt}{2}\right) \times \tan 20°$$

For calculation of (F_{SL}) use (Cp) = +0.2 in the above equation.

$$P_{HL} = \left|p_{rl}\right| \times \left(\frac{Wt}{2}\right) \times \tan 20°$$

3. *Sliding Force on the Superstructure*: The sliding force is a function of the number of foundation shear walls (transverse foundation walls) that are used. Note that all four sliding force horizontal components point in the same direction and thus are additive.

   a. For **two** end shear (transverse) walls: the end wall sliding force distribution is illustrated in Figure D-20 and calculated as follows:

$$F_{SL_{end}} = (Pt + Pb) \times \left(\frac{L}{2}\right) + \left(P_{HW} + P_{HL}\right) \times \left(\frac{L}{2}\right)$$

   b. For **four** shear (transverse) walls: the interior and end wall sliding force distribution is illustrated in Figure D-20 and calculated as follows:

$$F_{SL_{int}} = (Pt + Pb) \times \left(\frac{L}{3}\right) + \left(P_{HW} + P_{HL}\right) \times \left(\frac{L}{3}\right)$$

$$F_{SL_{end}} = (Pt + Pb) \times \left(\frac{L}{6}\right) + \left(P_{HW} + P_{HL}\right) \times \left(\frac{L}{6}\right)$$

   c. For **six** shear (transverse) walls: the interior and end wall sliding force distribution is illustrated in Figure D-20 and calculated as follows:

$$F_{SL_{int}} = (Pt + Pb) \times \left(\frac{L}{5}\right) + \left(P_{HW} + P_{HL}\right) \times \left(\frac{L}{5}\right)$$

$$F_{SL_{end}} = (Pt + Pb) \times \left(\frac{L}{10}\right) + \left(P_{HW} + P_{HL}\right) \times \left(\frac{L}{10}\right)$$

4. *Resisting Force supplied by the Superstructure*: At the shear walls the sliding force (F_{SL}) is resisted by the friction from the dead load of the

structure, reduced by the differential uplift pressure on the roof planes. Note that the "light" unit dead load was assumed for the calculations. The coefficient of static friction is assumed to be 0.4 for wood against concrete or masonry.

a. For **two** end shear (transverse) walls: the frictional resistance is a function of dead load as illustrated in Figure D-20 and calculated as follows:

$$Fr_{end} = \left(DL - |P_{VL}| - |P_{VW}|\right) \times 0.4 \times \left[\frac{\text{spacing}}{2}\right]$$

Spacing has been conservatively set to 4 feet, regardless of actual pier spacing. If (Fr_{end}) is negative, set $Fr_{end} = 0$.

b. For **four** shear (transverse) walls: the frictional resistance is distributed to an end and interior shear wall location as illustrated in Figure D-20 and calculated as follows:

$$Fr_{int} = \left(DL - |P_{VL}| - |P_{VW}|\right) \times 0.4 \times \text{spacing}$$

$$Fr_{end} = \left(DL - |P_{VL}| - |P_{VW}|\right) \times 0.4 \times \left[\frac{\text{spacing}}{2}\right]$$

Spacing has been conservatively set to 4 feet, regardless of actual pier spacing. If (Fr_{int} or Fr_{end}) is negative, set Fr_{int} or $Fr_{end} = 0$ as appropriate.

c. For **six** shear (transverse) walls: the frictional resistance is distributed to an interior and end wall the same as for four shear walls as illustrated in Figure D-20.

5. *Required Horizontal Anchorage Force*: If the horizontal sliding force exceeds the horizontal sliding resistance, then sliding occurs. This net sliding force (Ah) must be resisted by connections between the superstructure and the foundation shear walls or vertical X-bracing planes with an appropriate factor of safety, generally assumed to be 1.5 as for overturning. Refer to section D-300.2.A.5 for a full description. The equation requires substitution of the above (F_{SL}) and (Fr_{int} and Fr_{end}) values for the selected 2, 4 or 6 shear walls. For the interior shear wall locations:

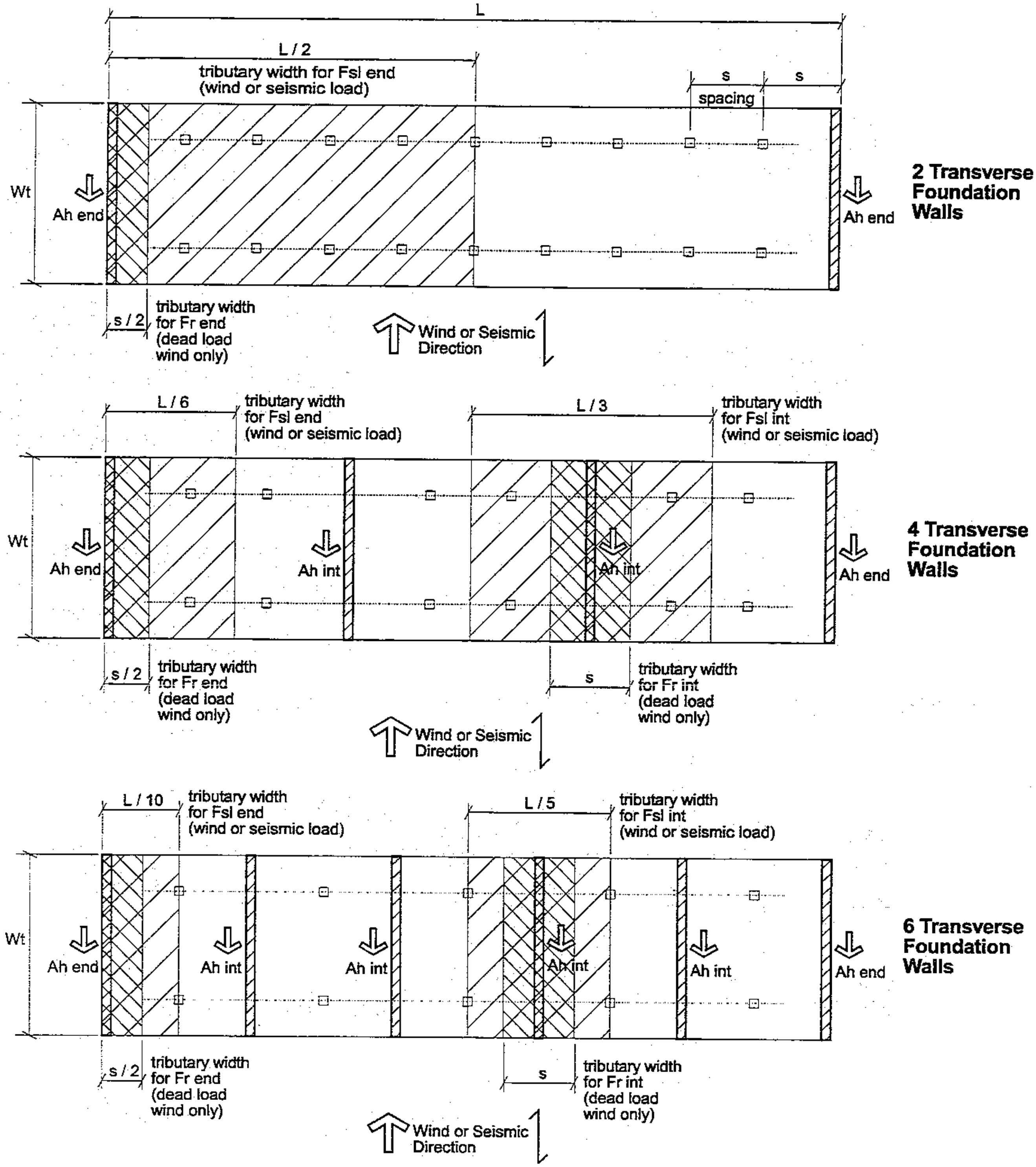
$$Ah_{int} = \frac{1.5 \times F_{SL_{int}} - Fr_{int}}{Wt}$$

and for the end shear wall locations:

$$Ah_{end} = \frac{1.5 \times F_{SL_{end}} - Fr_{end}}{Wt}$$

B. Wind Load Considerations for a Type C, E or I Multi-Section Unit.

1. *General*: Comparing Figures D-19 and D-21, it is clear that the behavior of a multi-section unit is identical to the single-section unit in regards to sliding. The behavior described in section D-300.4.A. can be applied here, except that the multi-section unit is twice as wide (2 × Wt).



Foundation Shear Wall Planes - Sliding - Transverse Direction

Figure D - 20

2. *Wind Loads on the Superstructure*: As shown in Figure D-21, the same wind force components are required, except that the roof forces are twice as large for the multi-section unit as follows:

$$Pt = Pb = \left(p_{ww} + |p_{wl}|\right) \times \frac{h_n}{2}$$

$$Pvw = p_{rw} \times Wt$$

For calculation of (Fr) use (Cp) = -0.9 in the above equation.

$$Pvl = p_{rl} \times Wt$$

$$P_{HW} = p_{rw} \times Wt \times \tan 20^\circ$$

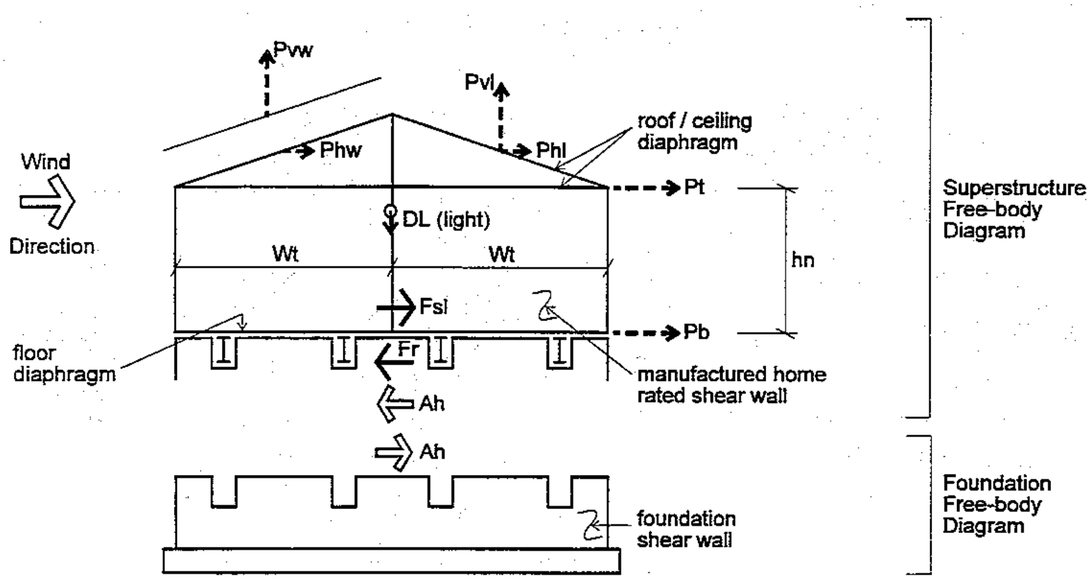
For calculation of (F_{SL}) use (Cp) = +0.2 in the above equation.

$$P_{HL} = p_{rl} \times Wt \times \tan 20^\circ$$

3. *Sliding Force on the Superstructure*: The sliding force equations for single-section units from section D-300.4.A.3 are applicable, substituting the force values from section D-300.4B.2 for multi-section units.

4. *Resisting Force supplied by the Superstructure*: The resisting force equations for single-section units from section D-300.4.A.4 with the same notes are applicable, substituting the "light" dead load for a multi-section unit and the wind force values from section D-300.4.B.2.

5. *Required Horizontal Anchorage Force*: Similar equations are utilized as for the single-section units except for unit width (2 x Wt). The



Type C, E or I Multi-Section Units

Wind Related Sliding - Transverse Direction

Figure D - 21

equation requires substitution of the above (F_{SL}) and (Fr_{int} and Fr_{end}) values for the selected 2, 4 or 6 shear walls. For the interior shear wall location:

$$Ah_{int} = \frac{1.5 \times F_{SL_{int}} - Fr_{int}}{2 \times Wt}$$

and for the end shear wall location:

$$Ah_{end} = \frac{1.5 \times F_{SL_{end}} - Fr_{end}}{2 \times Wt}$$

C. Horizontal Anchorage with X-Bracing. The calculation of (Ah) is necessary to proceed to analyze X-bracing alternatives. Refer to Figure 6-10 and section 602-5.G for illustration and explanation of two horizontal anchorage options:

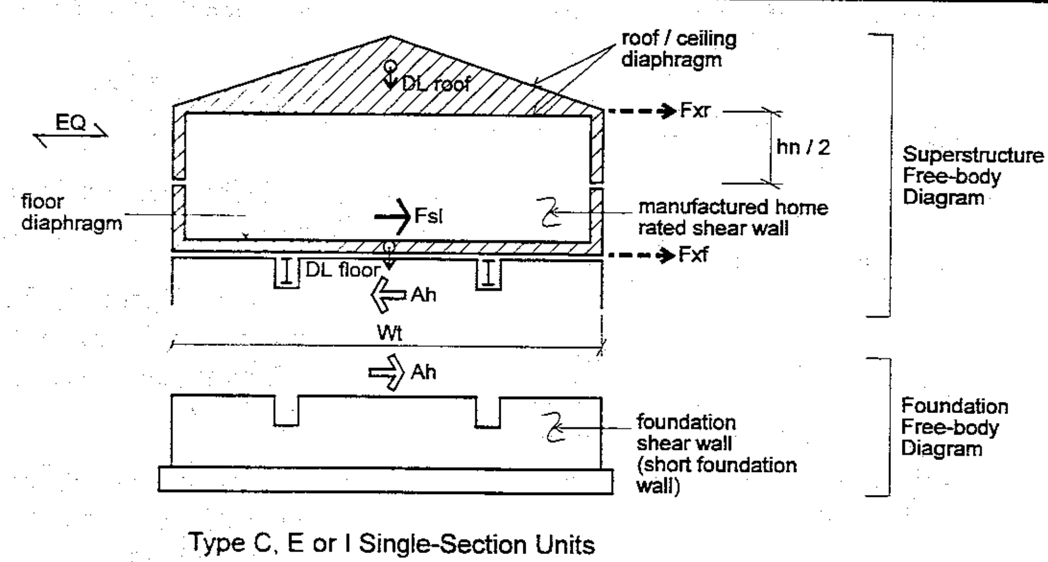
1. To use steel straps to complete the transverse foundation walls, or

2. To use steel straps instead of transverse foundation walls.

D-300.5 REQUIRED HORIZONTAL ANCHORAGE BASED ON SEISMIC IN THE TRANSVERSE DIRECTION. Refer to Figures D-22 and D-23 for the free-body diagrams of the superstructure and foundation for single section and multi-section units, illustrating the horizontal forces due to seismic induced sliding. No gravity load frictional resistance is considered due to the dynamic vertical component of acceleration. Horizontal sliding is not influenced by the foundation Type **C**, **E** or **I**; thus the same analysis applies to all of the foundation types. Figure D-7 is related to the equation development for the calculation of horizontal inertia floor and roof forces.

A. Seismic Force Considerations for the Type C, E and I Single-Section Units.

1. *General*: Figure D-22 shows all the applied and resisting forces involved in the horizontal equilibrium equations.



Seismic Related Sliding - Transverse Direction

Figure D - 22

2. *Seismic Inertia Forces on the Superstructure*: Determination of the horizontal forces was explained in section D-200.5. The "heavy" component dead loads were used to arrive at the inertia forces for single-section units.

3. *Sliding Force on the Superstructure*: The sliding force is a function of the number of foundation shear walls (transverse foundation walls) that are used.

   a. For **two** end shear (transverse) walls: the end wall sliding seismic force distribution is illustrated in Figure D-20 and calculated as follows:

   $$F_{SL_{end}} = (F_{xr} + F_{xf}) \times \left(\frac{L}{2}\right)$$

   b. For **four** shear (transverse) walls: the interior and end wall sliding seismic force distribution is illustrated in Figure D-20 and calculated as follows:

   $$F_{SL\,int} = (F_{xr} + F_{xf}) \times \left(\frac{L}{3}\right)$$

   $$F_{SL\,end} = (F_{xr} + F_{xf}) \times \left(\frac{L}{6}\right)$$

   c. For **six** shear (transverse) walls: the interior and end wall sliding seismic force distribution is illustrated in Figure D-20 and calculated as follows:

   $$F_{SL_{int}} = (F_{xr} + F_{xf}) \times \left(\frac{L}{5}\right)$$

   $$F_{SL_{end}} = (F_{xr} + F_{xf}) \times \left(\frac{L}{10}\right)$$

4. *Resisting Force supplied by the Superstructure*: The unreliability of friction to provide horizontal resistance to sliding during a seismic event requires :

   Fr = 0 for interior and end wall resistance.

5. *Required Horizontal Anchorage Force*: The equations require substitution of the above (F_{SL}) values for the selected 2, 4, or 6 shear walls. The horizontal sliding force for interior shear wall locations is:

   $$Ah_{int} = \frac{1.5 \times F_{SL_{int}}}{Wt}$$

   The horizontal sliding force for the end shear wall locations is:

   $$Ah_{end} = \frac{1.5 \times F_{SL_{end}}}{Wt}$$

B. Seismic Force Considerations for the Type C, E and I Multi-Section Units.

1. *General*: Figure D-23 shows all the applied and resisting forces involved in the horizontal equilibrium equations.

2. *Seismic Inertia Forces on the Superstructure*: Determination of the horizontal forces was explained in section D-200.5. The "heavy" com-

ponent dead loads for multi-section units were used to arrive at the inertia forces.

3. *Sliding Force on the Superstructure*: The sliding force is a function of the number of foundation shear walls (transverse foundation walls) that are used. Reference Figure D-20 as a similar illustration, changing the unit width from (Wt) to (2 × Wt). The equations for horizontal sliding are the same as for single-section units, except that the magnitude of the inertia forces is for multi-section units as described in section D.200.5.B and D.200.5.E.7.a. and D-200.5.E.8. The sliding (F_{SL}) equations then duplicate as shown in section D-300.5.A.3 with the larger F_{xr} and F_{xf} values used in the equations.

4. *Resisting Force supplied by the Superstructure*: The unreliability of friction to provide horizontal resistance to sliding during a seismic event requires :

   Fr = 0 for interior and end wall resistance.

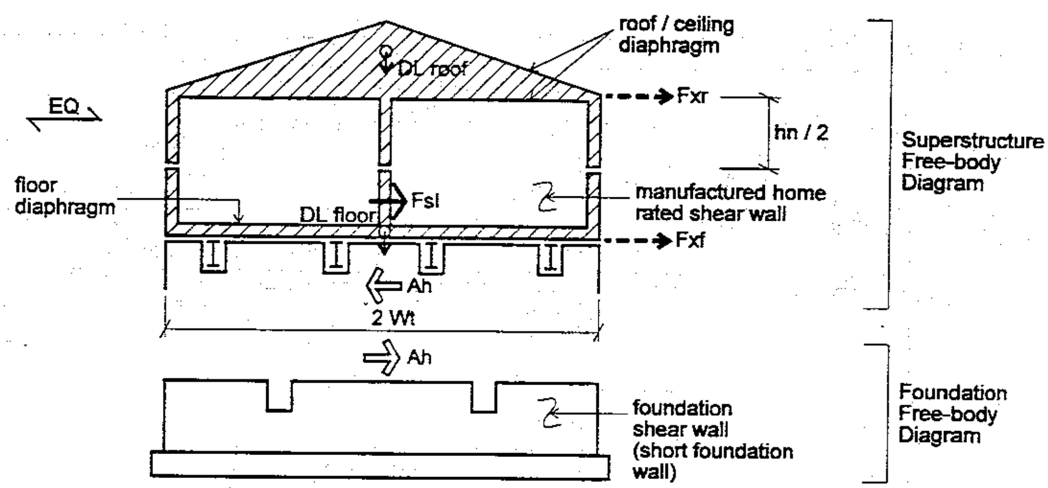
5. *Required Horizontal Anchorage Force*: The equations require substitution of the above (F_{SL}) Multi-Section unit values for the selected 2, 4 or 6 shear walls. The horizontal sliding force for the interior shear wall locations is:

$$Ah_{int} = \frac{1.5 \times F_{SL_{int}}}{2 \times Wt}$$

and the horizontal sliding force for the end shear wall location is:

$$Ah_{end} = \frac{1.5 \times F_{SL_{end}}}{2 \times Wt}$$

C. Horizontal Anchorage with



Type C, E or I Multi-Section Units

Seismic Related Sliding - Transverse Direction

Figure D - 23

X-Bracing. The calculation of (Ah) is necessary to proceed to analyze X-bracing alternatives. Refer to Figure 6-10 and section 602-5.G for illustration and explanation of two horizontal anchorage options:

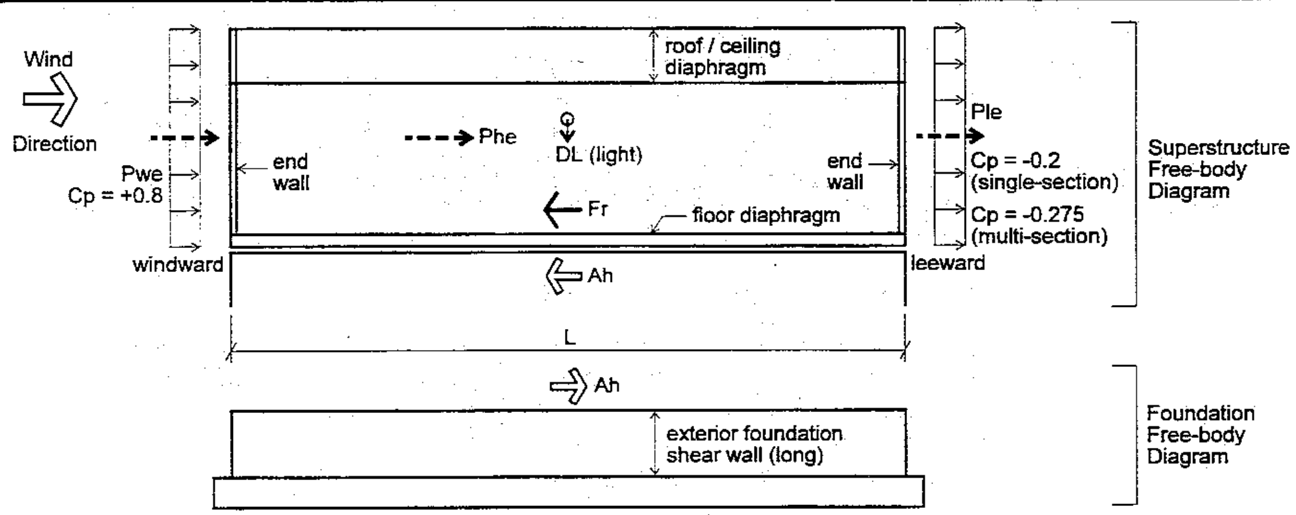
1. To use steel straps or rods to complete the transverse foundation walls, or

2. To use steel straps or rods instead of transverse foundation walls.

D-300.6 REQUIRED HORIZONTAL ANCHORAGE BASED ON WIND IN THE LONGITUDINAL DIRECTION. Refer to Figure D-24 for the free-body diagram of the superstructure and foundation for single section and multi-section, illustrating the horizontal forces due to longitudinal sliding from wind loading. The longitudinal sliding force (Ah) is not influenced by the foundation Type **C**, **E** or **I**. The same free-body diagram is used for the analysis; however, the detailing does differ based on foundation Type **C**, or **E** and **I**. The Type **E** or **I** foundation, where structural exterior longitudinal foundation walls are used, is illustrated in Figure D-25. A Type **C** unit, where non-structural exterior longitudinal walls are typically used, incorporates vertical X-bracing planes along the chassis beam lines for longitudinal sliding resistance as illustrated in Figure D-26. Figure D-4 is also related to the equation development of this section. A roof slope of 20 degrees (approx. 4 in 12 slope) was used so as to maximize the end wall area to produce the largest horizontal windward and leeward forces. Note that internal pressures GCpi on the end walls cancel (see Figure D-5). For allowable stress design methodology, the load combination from ASCE 7-93 is: (Wind - DL). Figures D-25 and D-26 also illustrate that a tributary width approach is used to calculate the (Ah) force transferred to each foundation horizontal load resisting plane.

A. Wind Load Considerations for the Type C, E and I Single-Section Units.

1. *General:* As shown in Figure D-24



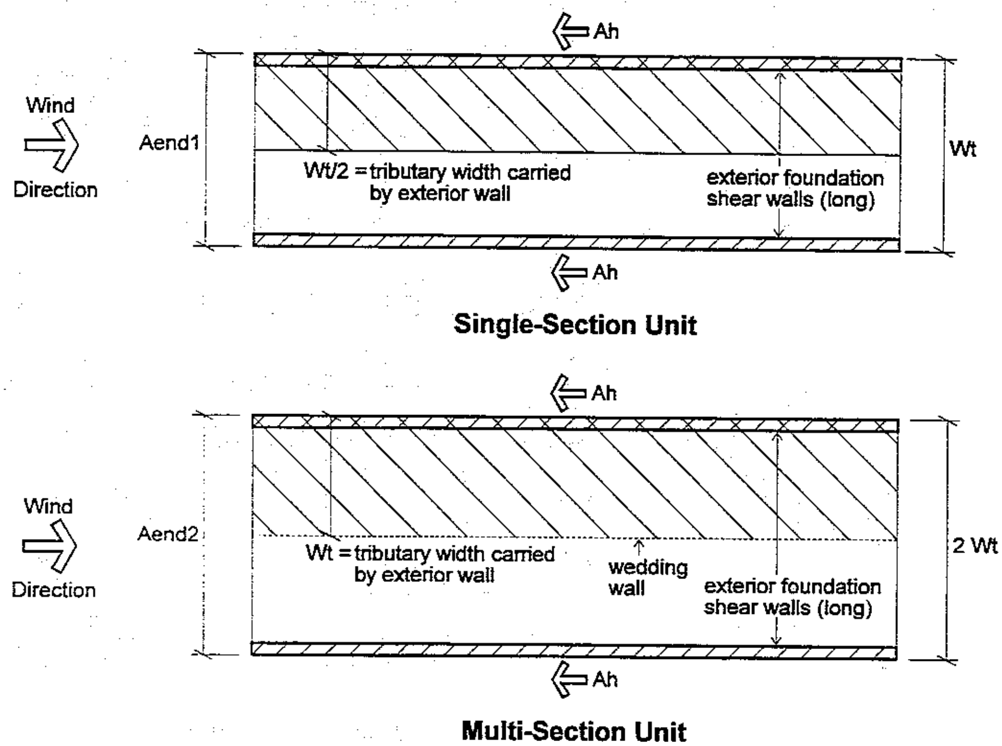
Single or Multi-Section Unit

Wind Related Sliding - Longitudinal Direction

Figure D - 24

the external wind pressure on the windward wall and the external suction on the leeward wall are transferred into the roof (plus ceiling) and floor diaphragms. The roof (plus ceiling) diaphragm transfers the force into the exterior superstructure shear walls parallel to the unit length, and then in turn to the floor diaphragm, assuming all connections are properly designed to resist the horizontal wind forces. From the floor diaphragm the horizontal force is transferred into the exterior (longitudinal) foundation shear walls for Type **E** or **I** units, or is transferred from the exterior walls to the vertical X-bracing planes under chassis beam lines for Type **C** units as shown in Figures D-25 and D-26 respectively. Also, reference Figure 6-6 for further illustration of both longitudinal resistance systems. **Note:** it is assumed that the exterior superstructure shear walls can transfer their force through the floor diaphragm and send the total sliding force over to the chassis beam lines for the Type **C** foundation.

2. *Wind Loads on the Superstructure*: The resultant wind forces occur on the end elevations of the single-section unit. The windward pressure (p_{WE}) and the leeward suction (p_{LE}) include exterior effects only (internal effects cancel) as shown in Figures D-4 and D-5. The areas over which these pressures act are



Type E or I - Foundation Shear Wall Plans - Wind Related Sliding - Longitudinal Direction

Figure D - 25

illustrated in Figure D-27 for single-section units, and is calculated as follows:

$$A_{end_1} = Wt \times h_n + \left(\frac{Wt}{2}\right)^2 \times \tan 20°$$

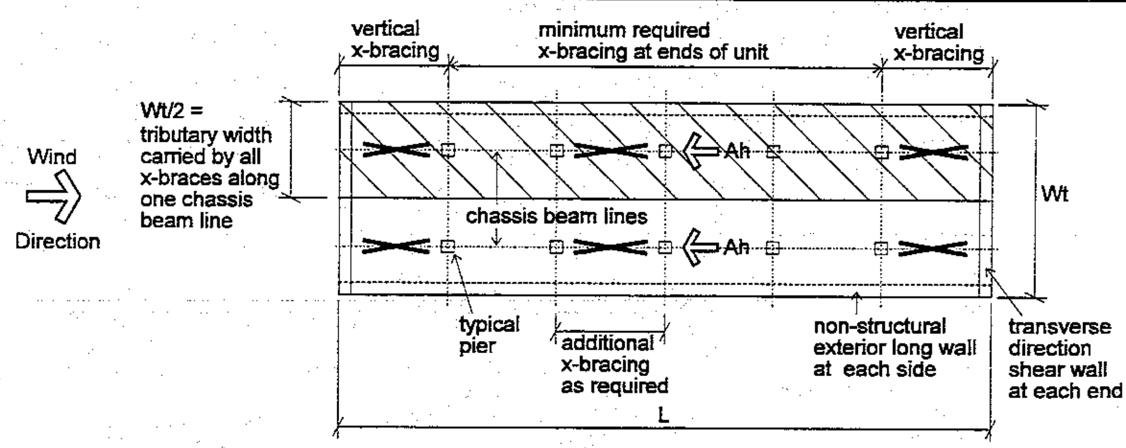
The combined longitudinal resultant force for the windward and leeward end walls of a single-section unit is:

$$P_{HE} = \left(p_{WE} + |p_{LE}|\right) \times A_{end_1}$$

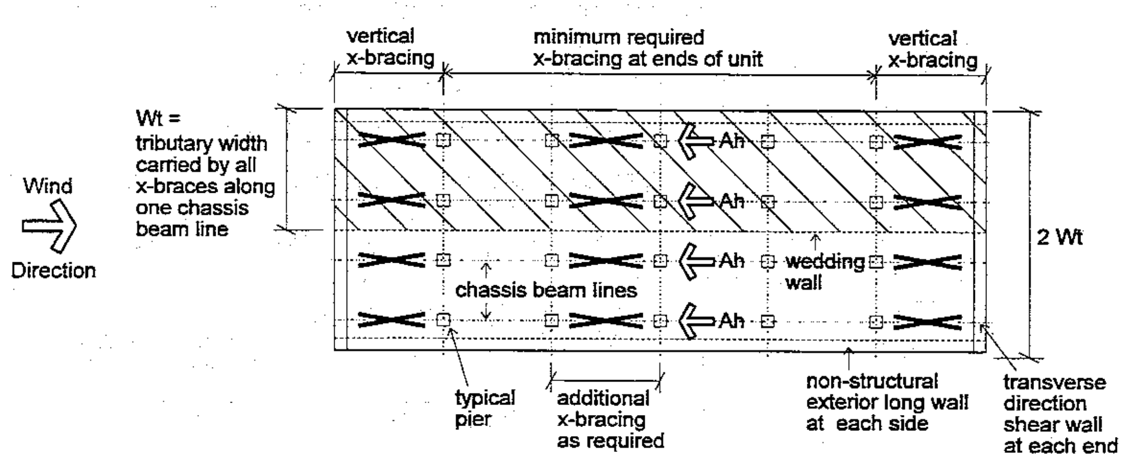
For selection of external pressure coefficients (Cp) on the leeward side, use Cp = -0.2 for single-section units

3. *Sliding Force on the Superstructure*: The sliding force is distributed to the two longitudinal exterior superstructure walls and then to the floor diaphragm as follows:

$$F_{SL} = \frac{P_{HE}}{2 \times L}$$



Single-Section Unit



Multi-Section Unit

Type C - Foundation Vertical X-Bracing - Wind Related Sliding - Longitudinal Direction

Figure D - 26

4. *Resisting Force supplied by the Superstructure*: Superstructure gravity dead loads are distributed differently to the Type **C** and Type **E** or **I** foundations as described in section D-300.1.A to C. Also, the roof planes are subjected to external and interior combined suctions as shown in Figures D-4 and D-5 that would offset much of the dead load in most cases. **Note**: the "light" unit dead load was assumed. For these reasons, and for simplicity in the analysis, no frictional resistance was assumed to exist. This is a conservative approach. It should be pointed out that for wind speeds of 80 and 90 MPH incorporating the Cp=-0.7 on both roof sloping planes would have resulted in no sliding, meaning values of F_{SL} that are negative. This was ignored for additional conservatism.

5. *Required Horizontal Anchorage Force*: The longitudinal sliding force (A_h), without any assumed frictional resistance, is the same magnitude as the sliding force on the superstructure (F_{SL}). This sliding force (Ah) must be resisted by connections between the superstructure and the longitudinal foundation shear walls for Type **E** or **I** Foundations, and it must be resisted by vertical X-bracing planes for Type **C** foundations. The appropriate factor of safety is assumed to be 1.5 (as for overturning). Refer to section D-300.2.A.5 for a full description. The longitudinal sliding force per foot of length of unit is:

$$\mathrm{Ah} = 1.5 \times F_{SL}$$

B. Wind Load Considerations for the Type C, E and I Multi-Section Units.

1. *General*: The analysis process is the same as for single-section units, except that the end elevation area is greater than the single section unit. Figures D-24 to D-27 illustrate the multi-section unit information required.

2. *Wind Loads on the Superstructure*: The resultant wind forces occur on the end elevations of the multi-section unit. The windward pressure (p_{WE}) and the leeward suction (p_{LE}) include exterior effects only (internal effects cancel) as shown in Figures D-4 and D-5. The area over which these pressures act is illustrated in Figure D-27 and is calculated as follows:

$$A_{end_2} = (2 \times \mathrm{Wt}) \times h_n + (\mathrm{Wt})^2 \times \tan 20°$$

The combined longitudinal resultant force for a multi-section unit is:

$$P_{HE} = (p_{WE} + |p_{LE}|) \times A_{end_2}$$

and Cp = -0.275 for multi-section units in the calculation of (p_{LE}) as required above.

3. *Sliding Force on the Superstructure*: The sliding force is distributed to the two longitudinal exterior superstructure walls and then to the floor diaphragm as follows:

$$F_{SL} = \frac{P_{HE}}{2 \times L}$$

4. *Resisting Force supplied by the Superstructure*: Same discussion applies as for single-section units.

5. *Required Horizontal Anchorage Force*: The same discussion applies as for single-section units. The longitudinal sliding force, distributed to each exterior longitudinal wall, per foot of length of unit is:

   $$Ah = 1.5 \times F_{SL}$$

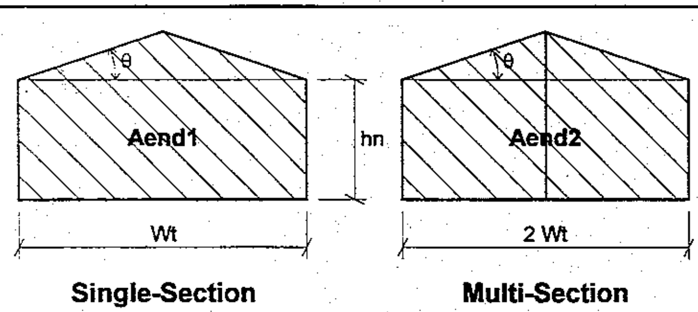
   Note if 4 lines of vertical X-bracing are to carry the sliding force (F_{SL}) as depicted in Figure D-26 then:

   $$Ah = \frac{1.5 \times F_{SL}}{2}$$

C. Horizontal Anchorage with X-Bracing. The calculation of (Ah) is necessary to proceed to analyze X-bracing. Refer to Figure 6-11 and section 603-6.F for illustration and explanation of the horizontal anchorage with X-bracing in the longitudinal direction:

Note: X-bracing is typically used for Type **C** units. Only Perimeter longitudinal foundation walls would typically be required for Type **E** or **I** units.

D-300.7 REQUIRED HORIZONTAL ANCHORAGE BASED ON SEISMIC IN THE LONGITUDINAL DIRECTION. Refer to Figure D-28 for the free-body diagram of the superstructure and foundation for single section and multi-section units, illustrating the horizontal forces due to longitudinal sliding from seismic forces. The longitudinal sliding force (Ah) is not influenced by the foundation Type **C, E** or **I.** The same free-body diagram is used for the analysis; however, the detailing does differ based on foundation Type **C**, or **E** and **I.** The Type **E** or **I** foundation, where structural exterior longitudinal foundation walls are used, is similar to that illustrated for wind in Figure D-25. A Type **C** unit, where non-structural exterior longitudinal walls are typically used, incorporates vertical X-bracing planes along the chassis beam lines for longitudinal sliding resistance is similar to that illustrated for wind in Figure D-26. Figure D-7 illustrates the seismic terms and is related to the equation development found in section D-200.5.B. and E.7. and E.8. for the calculation of horizontal inertia floor and roof forces. These forces are the same magnitude in the transverse and longitudinal directions. For allowable stress design methodology, the load combination from ASCE 7-93 is: (Seismic)-DL.



End Elevation Areas - Wind - Longitudinal Direction

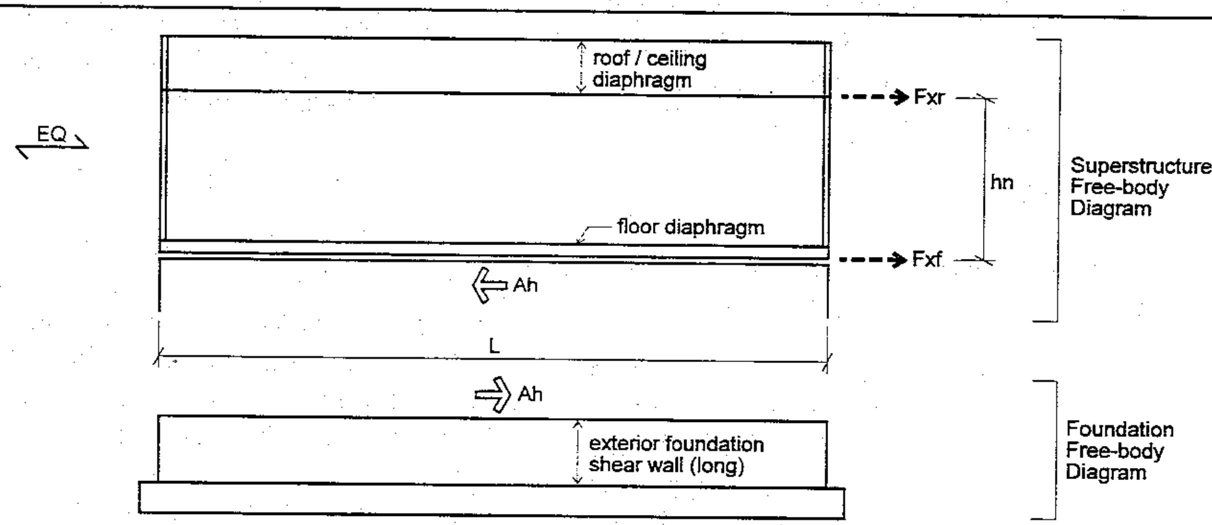
Figure D - 27

A. Seismic Force Considerations for the Type C, E and I Single-Section Units.

1. *General*: Figure D-28 shows all the applied and resisting forces involved in the horizontal equilibrium equations. The seismic inertia floor and roof forces are transferred into the roof (plus ceiling) and floor diaphragms. The roof (plus ceiling) diaphragm transfers the force into the exterior superstructure shear walls parallel to the unit length, and then in turn to the floor diaphragm, assuming all connections are properly designed to resist the horizontal inertia forces. From the floor diaphragm the horizontal force is transferred into the exterior (longitudinal) foundation shear walls for Type **E** or **I** units, or is transferred from the exterior walls to the vertical X-bracing planes under chassis beam lines for Type **C** units. Figures D-25 and D-26, drawn for wind loads, can be similarly applied. Also, reference Figure 6-6 for further illustration of both longitudinal resistance systems. It is assumed that the exterior superstructure shear walls can transfer their force through the floor diaphragm and send the total sliding force over to the chassis beam lines for the Type **C** foundation.

2. *Seismic Loads on the Superstructure*: Calculation of the seismic inertia forces is the same as that determined for the transverse direction seismic related sliding found in section D-200.5.B and E.7. and E.8.

3. *Sliding Force on the Superstructure*: The sliding force is distributed to the two longitudinal exterior superstructure walls and then to the floor diaphragm in lbs/ft of length as follows:



Single or Multi-Section Unit

Seismic Related Sliding - Longitudinal Direction

Figure D - 28

$$F_{SL} = \frac{(F_{xr} + F_{xf})}{2}$$

4. *Resisting Force supplied by the Superstructure*: The unreliability of friction to provide horizontal resistance to sliding during a seismic event requires :

   Fr = 0: for all foundation types

5. *Required Horizontal Anchorage Force*: The longitudinal sliding force (A_h), without any assumed frictional resistance, is the same magnitude as the sliding force on the superstructure (F_{SL}). This sliding force (Ah) must be resisted by connections between the superstructure and the longitudinal foundation shear walls for Type **E** or **I** Foundations, and it must be resisted by vertical X-bracing planes for Type **C** foundations. The appropriate factor of safety is assumed to be 1.5 (as for overturning). Refer to section D-300.2.A.5 for a full description. The longitudinal sliding force per foot of length of unit is:

   $$Ah = 1.5 \times F_{SL}$$

B. Seismic Load Considerations for the Type C, E and I Multi-Section Units.

1. *General*: The analysis process is the same as for single-section units, except that the dead load for the multi-section unit is greater than the single section unit, and the inertia forces will be greater. Figures D-24 to D-26, although illustrating wind loads, are similar for the multi-section unit information required for seismic forces.

2. *Seismic Loads on the Superstructure*: Calculation of the seismic inertia forces is the same as that determined for the transverse direction seismic related sliding.

3. *Sliding Force on the Superstructure*: The sliding force is distributed to the two longitudinal exterior superstructure walls and then to the floor diaphragm in lbs/ft of unit length as follows:

   $$F_{SL} = \frac{(F_{xr} + F_{xf})}{2}$$

4. *Resisting Force supplied by the Superstructure*: Same discussion applies as for single-section units. Thus, (Fr) = 0.

5. *Required Horizontal Anchorage Force*: The same equation applies as for single-section units. The longitudinal sliding force per foot of length of unit is:

   $$Ah = 1.5 \times F_{SL}$$

C. Horizontal Anchorage with X-Bracing. The calculation of (Ah) is necessary to proceed to analyze X-bracing. Refer to Figure 6-11 and section 603-6.F for illustration and explanation of the horizontal anchorage with X-bracing in the longitudinal direction:

Note: X-bracing is typically used for Type **C** units. Only Perimeter longitudinal foundation walls would typically be required for Type **E** or **I** units.

APPENDIX E
OWNER'S SITE ACCEPTABILITY WORKSHEET

Owner's Name: ______________________________

Address: ______________________________

____

____

Telephone: ______________________________

Site Location: ______________________________

Legal Description: ______________________________

____

____

____

Have you provided a copy of a map pinpointing the site? yes no

Have you submitted a foundation plan? yes no
(See #10 of Manufacture's Worksheet)

Preliminary Site Information

Before approval of the site can begin, the applicant must provide preliminary site information to the field office. Refer to Chapter 2, "Site Acceptability Criteria" for clarification.

1. Provide survey results showing existing grade elevation. (201-1) __________ ft.
2. Is the building in a flood-prone area? (201-2) yes no
   If the answer to 2 is Yes, answer 3, 4, & 5.
   If the answer to 2 is No, answer 6, below.

3. What is the Base Flood Elevation? ________ ft.

   What is the Flood Protection Elevation? ________ ft.

4. Has approval for drainage, grading and berming been approved for flood-prone sites? yes no

5. Have permits been provided? yes no
   (Permits must be obtained for any alteration of the building site in a flood protection area.)

6. Provide geotechnical report in areas of known high water table. (201-4) yes no

7. Provide geotechnical report if adverse site conditions are found or suspected. (203) yes no

8. Provide site-drainage plan complying with CABO R301.3 or local requirements. (301) yes no

9. Provide fill specifications if site is to be prepared with earth fill. (303-2) yes no

10. If a geotechnical report is required, what is the net allowable soil bearing pressure? (202) ________ psf.

11. If no adverse soil conditions are known or suspected, and if the home is individually sited, assume a soil bearing pressure of 1,000 psf. and use this value when a determination of soil bearing pressure is called for. 1,000 psf.

APPENDIX E
MANUFACTURER'S WORKSHEET

Manufacturer's
Company Name: ______________________________

Address: ______________________________

____

____

Telephone: ______________________________

Determination of Building Structure and Size

The manufacturer shall provide the following information:

1. Type of unit — Single-Section / Multi-Section
2. Method, location and types of support:
   Refer to Figures 6-7 and 6-8 and Section 601-4
   Is the home a **C, E, or I**? __________
3. Length of unit L __________ ft.
4. Actual width of unit Wt __________ ft.
5. Height of exterior wall ** __________ ft.
6. Height of roof peak ** __________ ft.
7. Roof slope ** __________
8. Self weight of total unit (W) including mechanical equipment ** __________ lbs.
9. Distance between chassis members __________ ft.
10. One foundation design concept (See Appendix A)
    (C1-C4; E1-E8; or I) __________

11. Recommended pier spacing **

   a. Exterior ____________ ft.

   b. Interior ____________ ft.

   c. Continuous Marriage Wall ____________ ft.

      Length of largest isolated marriage wall opening or average of largest two adjacent openings ____________ ft.

   d. Tie-down Strap (C1 concept only) ____________ (Number) ____________ ft. (Spacing)

12. One installation method recommendations (include documentation showing connection details pertinent to geographic area for seismic or wind). ** yes no

13. Interior shear wall locations (include documentation showing locations). ** yes no

14. Design wind speed used in designing connection details for horizontal anchorage (Ah) and vertical anchorage (Av) in the transverse direction. ** ____________ mph.

15. Seismic acceleration values used in designing connection details for horizontal anchorage (Ah) in the transverse and longitudinal directions. ** Av ____________ Aa ____________

16. Shear wall connection details with rated capacity for wind and seismic are provided. ** † yes no

   a. Connection locations at foundation end and interior walls shown? ** yes no

   b. Rated connection capacity for uplift and overturning ** ____________ lbs./ft. (or lbs./tie-down)

   c. Rated connection capacity for sliding in transverse direction ** ____________ lbs./ft. (or lbs./diag. strap)

   d. Rated connection capacity for sliding in longitudinal direction ** ____________ lbs./ft.

   e. Vertical X-bracing tension strap capacity ** ____________ lbs./diag. strap

f. Engineering calculation by licensed structural engineer? ** yes no

** **Optional values:** It is optional for the manufacturer to provide these values. If the manufacturer does not provide the values, it is the responsibility of the owner to supply values, based on engineering analysis by a licensed structural engineer.

† Item 16 is provided in California.

APPENDIX F
DESIGN WORKSHEET

Owner's Name: ____________________

Address: ____________________

Builder's Name: ____________________

Site Location: ____________________

PART 1: SITE CONDITIONS
(Accompanies Chapter 2)

1. Has the Manufacturer's Worksheet been provided? yes no

Existing Grade Elevation (201-1)

2. Does the site require a survey? yes no
   (Answer yes if: 1) elev. to be altered by grade or fill; 2) site near flood zone; 3) subdivision. Answer no if individually-sited with no alteration of building site.)

3. If yes to above, what is the surveyed existing grade elevation? __________ ft.

Flood Protection Elevation (201-2)

4. Is the building site in a flood zone? yes no
   (If yes to 4, then answer 5, 6, 7 & 8. If no, skip to 9.)

5. What is the Base Flood Elevation or the Flood Protection Elevation (use highest value)? __________ ft.

6. Is the site to be graded, filled, or bermed? yes no
   (If no, skip to 9.)

7. If yes to 6, have all permits been provided? yes no

8. If no to 6, then are the buildings to be built on elevated foundations? yes no
   (If yes, this handbook cannot be used. Refer to FEMA Manual.)

Frost Penetration Depth (201-3)

9. What is the maximum frost penetration depth? (see Appendix H, page H-4) ____________ in.

10a. Does foundation plan show base of footing extending below frost penetration depth? (If yes proceed; if no, applicant should revise plans.) yes no

10b. Does foundation plan show base of footing extending below top-soil layer (min. 12") to undisturbed soil? yes no

Ground Water Table Elevation (201-4)

11. For subdivisions, does a Geotechnical Engineer recommend drainage of subsurface water? (If no, skip to 13.) yes no

12. Has groundwater drainage plan been provided? yes no

Soil Conditions (202, 203)

13. If any of the following adverse site conditions are discovered, specific recommendations by a Geotechnical Engineer will be required (applies to subdivisions and individually-sited homes.)

Organic soil (8" topsoil layer)	yes	no
Expansive (shrink-swell) soil	yes	no
Sloping site	yes	no
Subsidence	yes	no

(Applicant may be referred to Geotechnical Engineer if any of the above are yes. If no, to all of above, move to next step.)

14. Is area in a known termite infestation area? yes no

Region classification? ____________
(See Appendix H, Termite Infestation Map, page H-10) (If no, skip to 16.)

15. Has applicant complied with CABO R-308 or local ordinance for construction procedures and treatment? (If yes, continue; if no, refer applicant to CABO requirements.) yes no

PART 2: SITE PREPARATION
(Accompanies Chapter 3)

16. Acceptable surface drainage plan provided? (301) yes no
    (If no, one must be provided for subdivision)

17. Grading plan provided? (302) yes no

18. Fill specifications conforming to those cited in HUD Land Planning Data Sheet (79g)? (303) yes no
    (If fill is used, below the home's foundation, a report by Geotech. Eng. should be submitted to provide fill specifications.)

19. Finish grade elevation? (304) ___________ *
    (Check answers to Part 1: #4 & #5. The finish grade elevation must be higher than #5 if in flood zone.)

PART 3: DESIGN LOADS
(Accompanies Chapter 4)

Information from Manufacturer's Worksheet

20. Has all the information been provided on the Manufacturer's Worksheet? (Appendix E) yes no

21. What is the building self weight (W)? ___________ lbs.
    (Mfg. Wksht. #8)

22. What is the building length (L)? ___________ ft.
    (Mfg. Wksht. #3)

23. What is the distributed weight per foot of unit length? (w=W/L) ___________ lbs./ft.
    (402-1.B, C)

24. What is the building type? Single-Section Multi-Section
    (Mfg. WkSht. #2)

    C, E, or I

    Foundation design concept? ___________ *
    (C1, C2, C3, C4, E1, E3, E4, E5, E6, E7, E8, I)

Dead Load (402-1)

25. What is the light dead load value from Table 4-1? (402-1.A.1) ____________ * (lbs./ft.)

26. What is the heavy dead load value from Table 4-1? (402-1.A.2) ____________ * (lbs./ft.)

27. Does the answer from Question #23 fall within the values in #25 and #26? (402-1.D) yes no
(If the answer is yes, continue. If no, the foundation is not within the limits of this document and must be redesigned by a structural engineer.)

Snow Load (402-2) / Minimum Roof Live Load (402-2.C)

28a. What is average annual ground snowfall (Pg)? (See Ground Snow Load map, pages H-11, H-12 and H-13.) ____________ * (lbs./sq.ft.)

28b. What is 0.7 multiplied by Pg? ____________ psf.

29a. What is the roof slope? (Mfg. Wksht. #7) ____________

29b. What is the minimum roof live load for the roof slope? (D-200.2.B) ____________ psf.

30. Record the larger magnitude of item 28b or item 29b. Use this magnitude for roof load where required. ____________ psf.

Wind Load (402-3)

31a. What is the basic wind speed (V)? (See Wind Speed map, page H-14.) ____________ mph.

31b. If V is less than 80 mph, record *MPS* min. 80 mph for wind design. (402-3.A) ____________ mph.

32. Is the site inland or coastal? (402-3.B) Inland Coastal
(If inland, skip to question #38.)

33. If a coastal area, has the manufacturer provided connection details? (402-3.D) (Mfg. Wksht. #12) yes no

34. If yes to #33, what design wind speed has the manufacturer used in designing connection details? (Mfg. Wksht. #14) ________ mph. *

35. Are the connection locations shown? (Mfg. Wksht. #16a) yes no

36. Are connection details provided for foundation shear walls? (For an answer of yes, all questions under Mfg. Wksht #16 must be answered satisfactorily.) yes no

37. Is the value for Question 34 equal to or greater than the number given in Question 31? (If yes, proceed. If no, return design to manufacturer for clarification.) yes no

Seismic Load

38a. What are the seismic acceleration values? (See Seismic maps, pages H-15 and H-16)

Aa ________ *

Av ________ *

38b. Is Av < 0.15? (if no, proceed. If yes, seismic need not be considered, skip questions 39 to 41.) yes no

39. Seismic performance category. (See H-300 for Special Requirements of Foundation Design.) ________

40. What is the applicant's proposed design concept? (Design Wksht. #24) ________ *

41. Do the Foundation Design Concept Tables approve the foundation system for use in seismic areas of Question #38 above? (See Appendix A) (If yes, proceed. If no, return to applicant for foundation design choice more suited to high seismic areas.) yes no

PART 4-FINAL DESIGN PROCEDURE

(Accompanies Chapter 6)

42. What is the actual building width? (Mfg. Wksht. #4) ________ ft.

43. The nominal building width to be used in the Foundation Design Tables, (Aftg, Av & Ah) is Wt: __________ ft.
(600-2.A and Figure 6-1)

44. Where are the foundation supports located? Check drawings submitted by the owner and Foundation Design Concepts in Appendix A. Circle the support locations shown on the Manufacturer's foundation concept plan.

Chassis Beams
Exterior Walls
Marriage Wall

45. Do these locations match the Foundation Concept shown in Appendix A? Do the locations match Question #24 on the Design Worksheet? yes no
(If yes, proceed. If no, return to Owner for clarification.)

46. Is Vertical Anchorage present? yes no
(601-2.B, 601-3.B & 601-4.B (Figures 6-7 & 6-8); Mfg. Wksht. #12 & #16)

APPENDIX A

47. What is the basic system type? __________ *
(From Part 3: #24; Mfg. Wksht. #2)

48. What is the spacing between piers?
(Mfg. Wksht. #11)
(602-2)

Exterior: 4' 5' 6' 7' 8'

Interior: 4' 5' 6' 7' 8'

Continuous Marriage Wall: 4' 5' 6' 7' 8'

Largest or Average Marriage Wall Opening: __________ ft.

Tie Down (C1) __________ ft.

APPENDIX B

Required Footing Size

49. The required Exterior Wall Footing, for the foundation type, is found in the Required Effective Footing Area table in App. B, Part 1. (Use maximum value from item #30.) __________ *

The Required Exterior Square Footing size is: Type C __________ sq.ft.

Type E or I __________ ft.
(width)

50. The Required Interior Footing area is: __________ sq.ft.
(Also exterior piers for foundation type E)

51a. The Required Continuous Marriage Wall Footing area is: __________ sq.ft.

51b. The Required Footing area under posts at the ends of marriage wall opening(s) is: __________ sq.ft.

Vertical Anchorage Requirements in the Transverse Direction (602-4)

52a. Using the Foundation Design Load Tables (Appendix B, Part 2), determine the Required Vertical Anchorage. Exterior Av __________ *
(lbs./pier spacing;
lbs./ft for E type;
lbs./tie-down spacing)

52b. Number of vertical tie-down locations for multi-section units: 2 or 4 or 6

52c. For units with additional vertical anchorage at the interior piers, determine the Required Vertical Anchorage. Interior Av __________ *
(lbs./int pier spacing)

53. What is the manufacturer-supplied value? (#16b, Mfg. WkSht.) Exterior __________ *

Interior __________ *

54. Is this value (#53) greater than the value given in #52a? (If yes, continue. If no, return to owner for clarification.) yes no

Horizontal Anchorage Requirements In The Transverse Direction (602-5)

	trial 1	trial 2	trial 3	
55a. What number of transverse foundation walls was selected? (602-5.E) (If vertical X-bracing planes are used, complete items #55a, #56 and #57 for 2 transverse walls, and then skip to item #59.)	2	4	6	
55b. Are diagonal ties used to complete the top of the transverse short wall for horizontal anchorage? (602-5.G.1)	yes no	yes no	yes no	
Estimate height (h) for appropriate illustration in Figure 6-10.				ft.

		trial 1	trial 2	trial 3	
56. Using the tables, find the Required Horizontal Anchorage (Ah). (Appendix B; Part 3)	End Wall Ah				lbs./ft.
	Int Wall Ah				lbs./ft.
57a. What is the manufacturer's-supplied rated capacity for sliding? (#16c, Mfg. WkSht.)					lbs./ft.
57b. If answer to item #55b is yes, record manufacturer or product supplier rated strap tension capacity					lbs./strap
58a. Is value #57a greater than item #56? If yes, continue. If no, return to section 602-4.C and to question #55a and select a larger number of transverse foundation walls. If the maximum number selected (6) does not work, return to owner (who may wish to contact the manufacturer for clarification).		yes no	yes no	yes no	
58b. If answer to #55b is yes, required tension in diagonal (T_t). (Complete procedure in Section 602.5.G.1.)					lbs.
58c. Is value #57b greater than #58b? If yes, continue to item #62. If no, return to owner for product with greater capacity.		yes no	yes no	yes no	

59. If using vertical X-bracing planes in lieu of transverse short walls (and the formulas in section 602-5.G.2). determine anchorage values and sizes for diagonal members. (If shear walls are selected in item #55, skip to item #62.)

	trial 1	trial 2	trial 3	
a. Vertical X-bracing spacing proposed.				ft. *
b. Number of vertical X-bracing locations proposed. (Item #13, Mfg. WkSht. for trial 1.)				*

	trial 1	trial 2	trial 3	
c. Required horizontal anchorage (C) value, based on formula. (602-5.G.2.c)				lbs./ x-brace set
d. Estimated height (h) in Figure 6-10.				ft.
e. Tension (T_t) required. (602-5.G.2.d)				lbs./diag.
60. What is the manufacturer-supplied rated strap tension capacity? (#16, Mfg. WkSht.) (or capacity defined by literature supplied by product supplier)				lbs. *
61a. Is value #57 greater than value #59c? If yes, continue. If no, return to Section 602-5.G and to question #59 and select a greater number of X-brace locations as a next trial. Repeat until answer is yes, then continue.	yes no	yes no	yes no	
61b. Is value #60 greater than value #59e? If yes, continue. If no, return to section 602-5.G and to question #59 and select a greater number of X-bracing locations. If the maximum number selected does not work, return to owner (who may wish to contact the manufacturer for clarification or product supplier for clarification).	yes no	yes no	yes no	

Horizontal Anchorage Requirements In The Longitudinal Direction (602-6)

62a. Using the tables, find the required horizontal anchorage (Ah) in the longitudinal direction. (Appendix B, Part 4) (602.6.E) Exterior Wall Ah __________ lbs./ft.

62b. If using vertical X-bracing planes (and the formulas in section 602-6.F) determine anchorage value for X-bracing planes. (If using exterior long walls, skip to item #63.)

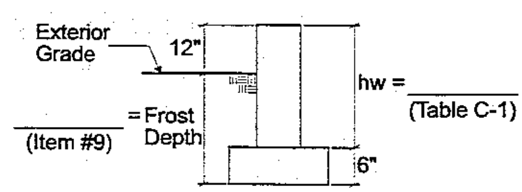
	trial 1	trial 2	trial 3
1. Number of chassis beam lines used for vertical X-bracing planes.	2 or 4	2 or 4	2 or 4

	trial 1	trial 2	trial 3	
Number of X-bracing planes proposed under each chassis beam along the length of the unit.				
2. Horizontal anchorage (B) required force, based on formula.				lbs.
3. Assumed height (h-b) based on Figure 6-11.				ft.
4. Tension (T_L) based on formula. (602-6.F.(3)).				lbs.
63. What is the manufacturer-supplied value for horizontal anchorage? (#16d, Mfg. WkSht.)				lbs./ft.
64a. For shear walls: is value #63 greater than #62a? If yes, skip to item #67. If no, contact owner for clarification.	yes no	yes no	yes no	
64b. For X-bracing: is value #63 greater than value #62b.2? If yes, return to item #62b.3. If no, increase number of vertical X-bracing planes and repeat items 62b.1 and 62b.2 until answer is yes. For multi-section units consider 4 lines of vertical X-bracing under all chassis beams.	yes no	yes no	yes no	
65. What is the manufacturer-supplied rated strap tension? (#16e, Mfg. WkSht. or product supplier)				lbs.
66. Is value #65 greater than #62b.4? If yes, continue. If no, contact owner to obtain straps with greater capacity, or return to item #62b.1 and increase the number of vertical X-bracing planes until answer is yes.	yes no	yes no	yes no	

APPENDIX C

Withdrawal Resistance Verification (603-2.B)

67. Using Appendix C, Table C-1 or C-2, verify that the foundation system will resist withdrawal. Answer question #67a for type E. Answer question #67b for types C, I, or type E with interior pier anchorage.



a. ***Withdrawal Resistance for long foundation wall.*** (Type E)
Circle the type of material that is to be used.

Reinforced Concrete
Masonry-Fully Grouted
Masonry-Grouted @ 48" o.c.
All-Weather Wood / Footing

1) Using Table C-1, which capacity is greater than required Av? (603-2.B.(1)) (#52a) __________ lbs./ft.

2) Using Table C-1, what is the height of the wall + footing for required withdrawal resistance? (hw + 6") __________ in.

3) What is the height of the wall + footing for frost protection? (frost depth (#9) + 12") __________ in.

4) What is the greatest height #67a.2 or #67a.3? __________ in.

   Circle the height which controls.

   Withdrawal
   Frost Depth

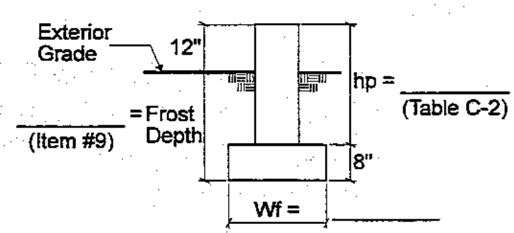
5) Record the bottom of footing depth from grade. (Item #67a.4 - 12") __________ in.

6) Using Table C-1, what is the required width of the wall footing for withdrawal? __________ in.

7) Is item #67a.6 greater than or equal to item #49? yes no
   If yes, continue. If no, change footing width to item #49.

8) Record design exterior wall footing width. __________ in.

b. ***Withdrawal Resistance for Piers.*** (Types C, C1 (concrete dead-man), I or type E with interior pier anchorage - multi-section units.)



Circle pier type:

Reinforced Concrete
Reinforced Masonry - fully grouted
Reinforced Concrete Dead-man

	Exterior	Interior (when used)	
1) Using Table C-2, which capacity is greater than required Av? (#52a and #52c) (603-2.B.(2))	________	________	lbs./pier *
2) Using Table C-2, what is the height of the pier + footing for required withdrawal resistance? (hp + 8")	________	________	in. *
3) What is the required height of pier + footing for frost protection? (frost depth (#9) + 12")	________	________	in.
4) What is the greatest height #67b.2 or #67b.3?	________	________	in.
Circle the height which controls.	Withdrawal Frost Depth	Withdrawal Frost Depth	
5) Record the bottom of footing depth from grade. (Item #67b.4 - 12")	________	________	in.
6) Using Table C-2, what is the required width of the square footing if withdrawal resistance controls or if frost depth controls?	________	________	in. *

c. ***Frost depth for marriage walls.*** What is the required depth of footing below grade for frost protection? (frost depth (#9)) (no withdrawal resistance) ________ in.

Vertical Anchorage and Reinforcement for Longitudinal Foundation Walls and Piers (603-2.D)

68. Using Appendix C, Table C-3, C-4A or C-4B, verify that the foundation anchors will resist uplift. Answer question #68a for type E. Answer question #68b for types C, I, or type E with interior pier anchorage.

    a. ***Vertical Anchor Capacity for longitudinal foundation wall*** (type E). (603-2.D.2)

        1) Using Table C-4A (concrete & masonry), which capacity is greater than the required Av? (#52a, Design Wksht.) If treated wood wall, skip to item #68a.3. ________ lbs./lineal ft. of wall

Circle correct washer choice for the capacity selected — Standard Washer / Oversized Washer

2) Using Table C-4A (masonry and concrete):

a) Required anchor bolt diameter ________ in.

b) Required anchor bolt spacing ________ in.

c) Using Table C-3A:

(1) Rebar size ________ *

(2) Lap splice ________ in.

(3) Rebar hook length ________ in.

3) Using Table C-4B (wood), which capacity is greater than the required Av? (#52a, Design Wksht.)
If using concrete or masonry wall, skip to item #68b. ________ lbs./lineal ft. of wall

4) Using Table C-4B (wood):

a) Required nailing ________ *

b) Minimum plywood thickness ________ in.

c) Required anchor bolt diameter ________ in.

d) Required anchor bolt spacing ________ in.

b. ***Vertical Anchor Capacity for Piers***
(Types C, I, or type E with interior pier anchorage)
(603-2.D.1)

	Exterior	Interior (when used for anchorage in multi-section units)	
1) Using Table C-3, which capacity in the table is greater than the required Av? (From #52a, Design Wksht.)	________	________	lbs./pier

	Exterior	Interior	
2) Using Table C-3:			
a) Number of anchor bolts	1 or 2	1 or 2	
b) Anchor diameter	1/2" or 5/8"	1/2" or 5/8"	
3) Using Table C-3A:			
a) Rebar size	#4 or #5	#4 or #5	
b) Lap splice	________	________	in.
c) Rebar hook length	________	________	in.

Horizontal Anchorage and Reinforcement for Transverse Foundation Walls (603-3)

69. Using Appendix C, Table C-5A or C-5B, verify that the foundation anchorage will resist sliding at the transverse end foundation walls. Use for types C, E, or I.

	End Wall	Interior Wall	
a. *For continuous foundations.*			
Using Table C-5A (concrete & masonry) or C-5B (wood), which capacity is greater than the required (Ah) (603-3) (item #56)?	________	________	lbs./ft.
1) Using Table C-5A, find:			
a) Required anchor bolt diameter	________	________	in.
b) Required anchor bolt spacing	________	________	in.
c) Using Table C-3A:			
(1) Rebar size	________	________	*
(2) Lap splice	________	________	in.
(3) Rebar hook length	________	________	in.
2) Using Table C-5B, find:			
a) Required nailing	________	________	*

	End Wall	Interior Wall	
b) Minimum plywood thickness	________	________	in.
c) Required anchor bolt diameter	________	________	in.
d) Required anchor bolt spacing	________	________	in.

b. ***For transverse short foundation walls completed with diagonal braces.*** (603-5)

Using Appendix C, Table C-5A, verify the diagonal anchorage capacity to the short foundation wall.

	End	Interior	
1) Record the required horizontal force (Ah × Wt) from 602-5.G.1.a and item #56.	________	________	lbs.
2) Table C-5A capacity for one 1/2" diameter bolt at 12" o.c.	1800	1800	lbs.
3) Number of bolts (Ah × Wt ÷ 1800; one minimum) at concrete or masonry top of short wall.	________	________	*
4) Size of anchor bolts	________	________	in.
5) Using Table C-3A:			
a) Rebar size	________	________	*
b) Lap splice	________	________	in.
c) Rebar hook length	________	________	in.

c. ***For vertical X-bracing planes in the transverse direction.*** (603-6)

Using Appendix C, Table C-5A, verify the diagonal anchorage to the pier footings and the tension capacity of the diagonals.

1) Record the required horizontal force (C) from item #59c.	________	lbs.
2) Table C-5A capacity for one 1/2" diameter bolt at 12" o.c.	1800	lbs.

3) Number of bolts (C ÷ 1800; one minimum) at top of a footing. ____________ *

4) Record the required tension force (T_t) from item #59e. ____________ lbs./diag.

5) Select tension strap capacity greater than or equal to T_t from owner's product supplier or manufacturer's supplied capacity (item #60). ____________ lbs./diag.

6) Record diagonal strap data ____________

Horizontal Anchorage for Longitudinal Foundation Walls (603-4)

70. Using Appendix C, Table C-5A or C-5B, verify that the foundation horizontal anchorage will resist sliding at the long foundation walls. Use for types C, E and I.

a. ***For continuous exterior foundation walls.***

Using Table C-5A (concrete and masonry) or Table C-5B (wood), which capacity is greater than the required exterior Ah? (602-6.E) (item #62a) ____________ lbs./ft.

1) Using Table C-5A, find:

a) Required anchor bolt diameter ____________ in.

b) Required anchor bolt spacing ____________ in.

c) Using Table C-3A:

(1) Rebar size ____________ *

(2) Lap splice ____________ in.

(3) Rebar hook length ____________ in.

2) Using Table C-5B, find:

a) Required nailing ____________ *

b) Minimum plywood thickness ____________ in.

c) Required anchor bolt diameter ____________ in.

d) Required anchor bolt spacing ____________ in.

b. ***For vertical X-bracing planes.***
(603-6.A.(2))

Using Appendix C, Table C-5A, verify the diagonal anchorage to the pier footings and the tension capacity of the diagonals.

1) Record the required horizontal force (B) from item #62b.2. ________ lbs.

2) Table C-5A capacity for one 1/2" diameter bolt at 12" o.c. __1800__ lbs.

3) Number of bolts (B ÷ 1800; one minimum) ________ *

4) Record the required tension force (T_L) from item #62b.4. ________ lbs./diag.

5) Select tension strap capacity greater than or equal to T_L from owner's product supplier or manufacturer's supplied capacity (item #60). ________ lbs./diag.

6) Record diagonal strap data ________

SUMMARY SHEET
(Accompanies Chapter 7)

71. Compare values from preceding questions.
Select the largest value.

a. **Bearing area and vertical anchorage**

1. *Pier footings: types C, E & I.*

	Piers				
			Marriage Wall		
	Exterior	Interior	Cont.	At Post	
Required Effective Footing Area from questions #49, #50, & #51.	______	______	______	______	sq.ft.
Required footing area to resist withdrawal due to uplift from Question #67. (for single-section or 2 tie-down system, only the exterior piers resist uplift, for 4 tie-down only the interior piers and exterior walls resist uplift)	______	______	sq.ft.		

	Piers			
			Marriage Wall	
	Exterior	Interior	Cont.	At Post
Pier Footing Sizes (largest of above)	______	______	______	______ sq.ft.
"Dead-man" footing size.	______ sq.ft.			

Reinforcing for pier footings:
Bring forward answers from previous questions. (#68b)
(Types C , I, or E with interior pier anchorage.)

	Exterior	Interior
Number of anchor bolts	______	______
Anchor bolt diameter	______	______ in.
Rebar size	______	______
Lap splice	______	______ in.
Rebar hook length	______	______ in.

	Exterior	Interior	Marriage Wall
Footing depth: grade to bottom of footing	______	______	______ in.
Pier footing and "dead-man" footing reinforcing bars:			#4 at 10" o.c.
"Dead-man" footing depth: grade to bottom of footing			______ in.

2. *Long Foundation wall footing: type E or I:*

Required Effective Footing Width

Required Footing Width for soil bearing (#49)	______ ft.
Required Footing Width to resist uplift withdrawal (#67a.6)	______ ft.
Wall Footing Size (largest of above)	______ ft.
Footing Depth: Grade to bottom of footing (#67a.5)	______ in.

Footing reinforcing bars. 2 #4 bars

Reinforcing for longitudinal foundation walls: Record answers from item #68a and record sizes and spacings.

From 68a.2: masonry and concrete:

Required anchor bolt diameter ____________ in.

Required washer size Standard Oversized

Required anchor bolt spacing ____________ in

Rebar size ____________

Lap splice ____________ in.

Rebar hook length ____________ in.

From 68a.4: wood: Record answers from item #68a.4 and record sizes and spacings.

Required nailing ____________

Minimum plywood thickness. ____________ in.

Required anchor bolt diameter ____________

Required anchor bolt spacing ____________ in

b. **Horizontal anchorage in the transverse direction - foundation walls**

1. *Continuous foundation walls* (#69a)

Number of transverse foundation walls (#55a) 2 4 6

Required Footing Width (minimum) 12 in.

From #69a.1: concrete / masonry:

	End Wall	Interior Wall	
Anchor bolt diameter	____________	____________	in.

	End Wall	Interior Wall	
Anchor bolt spacing	________	________	in.
Rebar size	________	________	
Lap splice	________	________	in.
Rebar hook length	________	________	in.
<u>From #69a.2: wood:</u>			
Required nailing	________	________	
Minimum plywood nailer	________	________	
Anchor bolt diameter	________	________	
Anchor bolt spacing	________	________	in.

2. *For transverse short foundation walls completed with diagonal braces* (#69b)

	End	Interior	
Number of pairs of diagonals (1 for single-section units, 2 for multi-section units) times number of short walls (end or interior) (#55a)	________	________	
Diagonal spacing (same as number of short walls)	________	________	
<u>From #69b: concrete / masonry:</u>			
Anchor bolt diameter	________	________	in.
Number of bolts	________	________	
Rebar size	________	________	
Lap splice	________	________	in.
Rebar hook length	________	________	in.

3. *For vertical X-bracing planes in lieu of short walls.* (#69c)

Number of X-brace locations (#59) ________

Spacing of vertical X-brace planes (#59) __________ ft.

Items from #69c.3 and #69c.5

Required anchor bolt diameter __________ in.

Number of bolts at top of footing to connect diagonal __________

Diagonal strap size __________

Connection to top flange of chassis beam (describe) ____________________

c. **Horizontal anchorage in the longitudinal direction - exterior foundation walls**

1. *Continuous foundation walls*

Reinforcing for longitudinal foundation walls: record only if larger sizes or closer spacing than recorded for vertical anchorage (#71a.2).

From #70a.1: concrete / masonry:

Anchor bolt diameter __________ in.

Anchor bolt spacing __________ in.

Rebar size __________

Lap splice __________ in.

Rebar hook length __________ in.

From #70a.2: wood: record only if larger sizes or closer spacings than recorded for vertical anchorage (#71a.2)

Required nailing __________

Minimum plywood nailer __________

Anchor bolt diameter __________

Anchor bolt spacing __________ in.

APPENDIX G
SAMPLE PROBLEMS

All the data necessary for the approval of the adequacy of a permanent foundation for the manufactured home can be located in this handbook and on worksheets submitted by the homeowner. The HUD field office (or user) must refer to Design Worksheet as a guide through the process of collecting and verifying data.

There are two steps in the approval process: (1) the Owner's Site Acceptability / Manufacturer's Worksheets, with accompanying forms as required, from the owner, and (2) the Design Worksheet. The reader is referred to the completed worksheet samples in Appendix E.

Example #1 is a proposed site for a **multi-section** manufactured home in Champaign, Illinois. The **marriage wall has two adjacent large openings of 16 and 12 feet respectively**. The remainder of the wall is continuous. Both the Owner's Site Acceptability / Manufacturer's Worksheet and the Design Worksheet for Example 1 have been filled out. Asterisks (*) on the Design Worksheet mark the items that were filled in based on data submitted by the owner. The remaining data on the Design Worksheet must be collected from the handbook as described herein.

COMMENTS - EXAMPLE # 1

Item # DESIGN WORKSHEET

Part 1 -- Site Conditions

9. Refer to the Average Depth of Frost Penetration map on page H-4. The average frost depth for Champaign Illinois is 30 inches.

14. Refer to the Termite Infestation map on page H-10. The site is in a moderate to heavy infestation region.

15. The owner has indicated compliance with CABO R.308.

Part 3 -- Design Loads

21. Calculate the distributed weight per foot of length by dividing the total weight of the home by its length: 33,040/56=590 lbs./ft.

Dead Load

25. From Table 4-1 (402-1.A1). The light dead load value is 560 lbs./ft.

26. From Table 4-1, the heavy dead load value is 805 lbs./ft.

27. Yes, the distributed weight of the home is within the limits defined by this document. The design tables may be used.

Snow Load

28. Refer to the Ground Snow Load (Pg) map on page H-12 for the central United States. The average ground snow load is 20 psf.

29. Refer to Section D-200.2.B for minimum roof live load based on roof slope. For a 2 in 12 roof slope, the minimum roof live load is 20 psf.

30. Comparison of roof snow load (14 psf) and minimum roof live load, minimum

roof live load is greater; therefore, it controls.

Wind Load

31. Refer to the Design Windspeed map on page H-14. The site location is near the 70 mph design wind isobar. Use minimum 80 mph for *MPS* in lieu of map value.

32. Based on the map provided by the owner, the site is not near a hurricane coastline. The site is Inland.

Seismic Load

38a. Refer to the maps for Seismic acceleration <u>Aa</u> and <u>Av</u> on pages H-15 and H-16. The site has Seismic acceleration values: <u>Aa</u> = 0.05 and <u>Av</u> = 0.05.

38b. Residential construction is exempt from seismic considerations if <u>Av</u> is less than 0.15.

41. Checking the Foundation Design Concept Tables for Type **E1**, this foundation type is not recommended for seismic areas where <u>Aa</u> and <u>Av</u> are greater than or equal to 0.3. This is because the piers are unreinforced. The Type E1 concept is permitted in seismic areas where <u>Aa</u> and <u>Av</u> are greater than 0.3, if the piers are reinforced.

Part 4 -- Final Design Procedure

42. From the table (600-2.A.1), the nominal width for a 13'-6" home width is 14'-0".

44. The user will compare the Foundation Design Concept, Figures 6-7 and 6-8 with foundation drawings and details provided by the owner. The concept drawings identify the bearing and vertical anchorage locations. An anchorage system for the transverse and longitudinal directions must be clearly shown on the documents provided by the owner.

Required Footing Size

49. In order to determine the Required Footing sizes, the user needs the data from the following items on the Owner's Site Acceptability Worksheet: Nos. 10 or 11 and on the Design Worksheet: Nos. 24, 30, 43, 48.

Item Number

#10 or #11	Net allowable soil bearing pressure = 1000 psf
#24	Foundation System, Multi-Section type **E1**
#30	Ground snow load Pg = 20 psf. Use 30 psf for the Foundation Design Table. The 30 psf value with load factors applied is equivalent to a minimum live load of 20 psf.
#43	Nominal Building width: Wt = 14'-0"
#48	Pier Spacing: Interior and exterior piers, 5'-0"; Continuous Marriage wall piers, 8'-0".

Next the user will locate the Required Effective Footing Area tables in Appendix B, Part 1. The user locates the table for a multi-wide E with a nominal width of 14 feet.

49. The user finds a note which indicates that the minimum longitudinal foundation wall footing width is 1 foot.

50. Interior pier and exterior pier

1) For the interior and exterior piers, the user finds the block of values for minimum roof live load of 20 psf.

2) Next, the user finds the two rows of values for a Net Allowable Soil pressure of 1000 psf (read ext, int row).

3) Under the column for a pier spacing of 5 feet, the required pier footing area is 2.1 square feet (1'-6" x 1'-6").

51a. Continuous Marriage Wall Piers

1) Refer to the same block of values as for the exterior/interior footings.

2) Next the user finds the second line of values for a Net Soil Pressure of 1000 psf (labeled mar).

3) Under the column for a marriage wall pier spacing of 8 feet, the required pier footing area is 6.9 square feet (2'-8" x 2'-8").

51b. Marriage Wall Openings

1) Refer to the lower block of values as for the ext/int footing.

2) Next, the user finds the average of two adjacent openings from item#48 (14 feet). Read area of footing at piers under posts as 11.4 sq.ft. (3'-6"x3'-6").

Vertical Anchorage Requirements In The Transverse Direction

52. In order to determine the Required Vertical Anchorage the user needs the data from the following items on the Design Worksheet: Nos. 24, 31, 32, 43. With this information, the user can determine Vertical Anchorage in the transverse direction by using the appropriate table in Appendix B, Part 2.

1) The user locates the Tables for a Multi-Section E with a nominal width of 14 feet and 2 tie-downs.

2) Then the user finds a block of values for the Inland condition.

3) To the right of the 80 mph wind value, the user finds a value of 130 lbs./ft along the longitudinal exterior walls.

53. The user verifies that the manufacturer's design value (200 lbs./ft.) shown on line 16b of the Manufacturer's Worksheet is greater than the required value shown on line 52a. Otherwise repeat the process with four tie-downs.

Horizontal Anchorage Requirements In The Transverse Direction

55. Two (2) transverse foundation shear walls are initially selected in order to compare the required horizontal anchorage with the values provided by the manufacturer. This is trial #1.

56. In order to determine the Required Horizontal Anchorage the user needs data from the same items on the Design Worksheet that were required for Approval item number 52a plus item No. 22 (namely, the building length L = 56'-0"), No. 30, roof snow/minimum roof live load and No. 36, Seismic Acceleration values. Proceed knowing that you are exempt from seismic considerations.

Next, the user will locate the Required Horizontal Anchorage table in Appendix B, Part 3.

1) The user locates the tables for a Multi-Section E with a width of 14 feet and two (2) transverse walls.

2) Then the user finds the block of values for the Inland condition and the line of values for a design wind speed of 80 mph.

3) Then the user finds Seismic <u>Aa</u> range 0.05-0.2 and snow load range 0-100 psf. Only one row of values remains.

4) For a length L of 56 feet, the user rounds the value to the next highest number shown on the top line of the table -- 60 feet.

5) Under the column of values for 60 feet, the user finds the required anchorage Ah = 420 pounds per lineal foot along the length of each transverse shear wall. Note that the value was not grayed over, indicating the force calculations were controlled by wind.

Note: if the manufacturer has specified (1) diagonal metal straps to complete the transverse short foundation walls, or (2) vertical x-bracing in place of transverse foundation walls, for comparative purposes, the user shall use the formulas in section 602-5.G.1 or 602-5.G.2 and proceed with item #55b or #59 respectively.

58. Verify the Manufacturer's design value shown on line 57a (400 plf) is greater than the required value shown on line 56. Since it is not (420 > 400), attempt trial #2 ans consider 4 short walls. Repeat steps 1) to 5). Read (Ah) exterior 140 plf and (Ah) interior 280 plf, both less than the manufacturer's value 400 plf. Thus, 4 short walls will provide adequate sliding resistance.

Horizontal Anchorage in the Longitudinal Direction

62a. In order to determine the Required Horizontal anchorage in the longitudinal direction the user needs the same data as used in steps 52 and 56 from the Design Worksheet.

Next, the user will locate the Required Horizontal Anchorage in the Longitudinal Direction tables in Appendix B, Part 4.

1) The user locates the table for a Multi-section unit Type E with a nominal width of 14 feet.

2) Then the user finds Seismic <u>Aa</u> range 0.05-0.1 and snow load range 0-100 psf.

3) Then the user finds the block of values for the Inland condition and the row of values for a design wind speed of 80 mph.

4) For a length L of 56 feet, the user rounds the value to the next highest number shown on the top row of the table -- 60 feet.

5) Under the column for 60 feet, the user finds the required anchorage force Ah = 67 plf along each of the longitudinal exterior shear walls. Note that the value was not grayed over indicating that the force calculations were controlled by wind, not seismic.

Note: if the manufacturer has specified a diagonal metal strap X-bracing in place of the shear wall, for comparative purposes, the user shall use the formulas in section 602-5.F, which are based n the required anchorage (Ah) found in the tables. This could be the case for Type C or I units.

64. Verify the manufacturer's design value on line 63 is greater than the required value shown on line 62a.

Withdrawal Resistance Verification

67. For type E foundations answer item 67a.

67a. For this example, a masonry foundation fully grouted was depicted on the documents submitted by the owner.

1) Checking the tabular columns of Table C-1, Appendix C, for Masonry-Fully Grouted, the lowest value greater than (Av) is 231 lbs. per foot. Thus, 231 > 130 (from item #52).

2) The footing depth (Hw) is found in the far left column, hw = 2'0'. This value corresponds to the minimum depth of the footing below grade which is shown in the illustration above the table.

3) The width of the footing is found at the top of the column, 12".

4) Based on item #9, the frost depth for Champaign, IL. is 30 inches. Based on Table C-1, the depth of the base of the footing below grade is :

from Table C-1:

hw =	24"	
	+ 6"	(footing thickness)
	30"	for withdrawal resistance

for frost protection:

hw =	30"	(depth below grade)
	+ 12"	(min. wall height above grade)
	42"	

therefore; frost protection controls over withdrawal resistance

42"	
- 12"	(min. wall height above grade)
30"	(bottom of footing to grade)

for establishing the number of block courses:

42"	
- 6"	(footing depth)
36"	min. required foundation wall height

Use hw = 40", which is a multiple of the 8" masonry unit -- 40" = 5 block courses.

5) Interior piers under (item #67b.3.) chassis beams do not participate in vertical anchorage for this example. Frost depth considerations are accounted for at the perimeter walls. Interior piers may be set below the 18 " of topsoil on undisturbed soil. See item #50 for required footing size.

6) Item #67c.; Marriage wall piers do not participate in vertical anchorage in any case, and do not need to set at frost depth. Again, set footings below the 18" of topsoil onto undisturbed soil.

Vertical anchorage and reinforcement for longitudinal foundation walls and piers

68. For type E foundations answer item 68a.

68a.

1) From item #52, the value for (Av) was 130 lbs./ft. Using Table C-4A for a ma-

sonry foundation wall, the first value in the left hand column is 146 lbs. per foot of wall. The 146 lbs./ft. value utilizes the maximum recommended anchor spacing by code as 6'-0" o.c. The wood material connected to the anchor bolt with a standard washer controls the final capacity. (Note the similarity in capacities with a treated wood foundation wall, Table C-4B, since wood bearing on washer controls).

2) For a masonry wall grouted solid, the following sizes are required:

On Table C-4A - on the same line as +146 lbs./ft., read:

a) Anchor Bolt diameter = 1/2"

b) Anchor Bolt spacing = 72"

On Table C-3A - on the same line as 1/2" anchor bolt diameter read:

c.1) Rebar size = #4

2) Lap splice = 16"

3) Rebar hook length = 6"

Horizontal Anchorage and Reinforcement for Transverse Foundation Walls

69a. From item number 56, the value for transverse (Ah) is 140 lbs. per foot along the transverse end (shear) wall and 280 lbs. per foot along the interior transverse walls. Using Table C-5A for a masonry foundation wall, the first value in the left hand column is 300 lbs. per foot of wall which is greater than either end or interior (Ah). The 300 lbs./ft. value is based on the maximum recommended anchor spacing of 6'-0" o.c. by code. The material connected to the anchor bolt will control the final capacity.

1) For masonry walls grouted solid, the following sizes are required:

On Table C-5A: On the same line as Ah = 300 lbs./ft., read:

a. Anchor bolt diameter = 1/2"

b. Anchor bolt spacing 72" (cores must be grouted solid)

On Table C-3A: On the same line as 1/2" anchor bolt diameter, read:

c.1) Rebar = #4

2) Lap splice = 16"

3) Rebar hook length = 6"

Horizontal Anchorage and Reinforcement for Longitudinal Foundation Walls

70a. From item #62a, the value for longitudinal (Ah) is 67 plf. From Table C-5A, again the 300 plf value is adequate. All other information for reinforcement is the same along the exterior longitudinal walls.

Summary Sheet

The values can be brought forward on to the summary sheet and the design approved.

EXAMPLE 2

Example #2 is a proposed site for a **single-section** manufactured home in Tampa Florida. The data on the Owner's Site Acceptability Worksheet remains the same as Example #1, with the exception of item 1. The grade elevation is 28 feet. The data on the Manufacturer's Worksheet, regarding the superstructure, remains the same as Example #1 with the exception of the following items:

Item #	Data
1.	Single-section (Nominal 14' wide unit)
2.	Type C
7.	Roof slope = 4 in 12
8.	Unit weight = 16,500 lbs.
10.	Type C1
11a.	Pier Spacing = 7 ft.
11b.	NA
11c.	NA
11d.	7 Tie-down straps at 8'-8" spacing **Note**: Tie-downs are required to be at 2'-0" in from each end of the unit. (Section 601-2.B.)
14.	Design wind = 120 mph
16b.	Uplift capacity = 3,150 lbs./tie-down
16c.	Sliding capacity = 4,800 lbs./diag. set
16d.	Sliding capacity = 4,800 lbs./diag. set
16e.	Vertical X-bracing tension capacity = 5600 lbs./strap

Asterisks (*) on the HUD Approval Worksheet mark the items that were filled in based on data submitted by the owner. As demonstrated in Example #1, the remaining data must be collected from the handbook as described herein.

Item # DESIGN WORKSHEET

Part 1 -- Site conditions

9. Refer to the Frost Penetration map on page H-4. The average **frost depth** for Tampa Florida is **zero inches.**

14. Refer to the Termite Infestation map on page H-10. The site is in a **very heavy** infestation area.

15. Yes, the owner has indicated compliance with CABO R-308.

Part 3 -- Design Load

23. The distributed weight is the weight of the home divided by its length:

    16,500 / 56 = 295 lbs./ft.

25. From Table 4-1 (402-1), the light dead load value is 290 lbs./ft.

26. From Table 4-1, the heavy dead load value is 425 lbs./ft.

27. Yes, the distributed weight of the home is within the limits defined by this document. The design tables may be used.

Snow Load

28. Refer to the Ground Snow Load map on page H-13 for the Eastern United States. The average **ground snow load** is **zero.**

29. Based on a 4 in 12 roof slope, the minimum roof live load is 15 psf (D-200.2.B).

30. The **15 psf minimum roof live load** controls.

Wind Load

31. Refer to the Design Wind Load map for the Eastern United States on page H-14. The average wind load is near the 100 mph design wind isobar, which exceeds the *MPS* minimum of 80 mph. Thus, **100 mph wind speed** is used for the foundation design.

32. Based on the map provided by the owner, the site is located on a hurricane coastline. The site is **Coastal.**

33-36. The manufacturer should supply details consistent with a coastal high wind site.

Seismic Load

38. Refer to the Seismic Acceleration maps on pages H-15 and H-16. The seismic coefficients for Hillsborough County, Aa and Av = 0.05. Residential construction is exempt from seismic consideration since Av < 0.15.

41. Checking the Foundation Design concepts for Type C1, it is permitted for use when seismic coefficient Av < 0.15. It is not acceptable for use in areas where Aa and Av greater than or equal to 0.3.

Part 4 -- Final Design Procedure

43. From the Section 600-2.A table, the nominal width for a 13'-8" home width is 14'-0".

44. The user will compare the Foundation Design concept illustrations with foundation drawings and details provided by the owner. The concept drawings identify the anchorage locations. An anchorage system must be clearly shown on the documents provided by the owner.

Required Footing Size

49. In order to determine the Required Footing size, the user needs the data from the following items on the Owner's Site Acceptability Worksheet item #10 or #11 and on the Design Worksheet: Nos. 24, 28-30, 43, 48.

<u>Item Number</u>

#10 or #11 Net allowable soil bearing pressure = 1000 psf from Owner's Worksheet.

#24 Foundation System, Single-section type C1

#28-#30 Ground Snow Load Pg = 0 psf. Use a minimum roof live load of 15 psf for the Foundation Design Load Tables.

#43 Building nominal width: Wt = 14'-0"

#48 Pier Spacing: Exterior = 7'-0"

Next the user will locate the Required Effective Footing Area Tables in Appendix B, Part 1.

1) The user locates the tables for a Single-section Type C with a width of 14 feet.

2) Find the block of values for a Minimum Roof Live Load of 15 psf.

3) Next the user finds the row of values for a net allowable soil pressure of 1000 psf.

4) Last, the user finds the intersection of that row with the column for a 7'-0" foot pier spacing. The required footing area is 5.3 square feet (2'-4" x 2'-4").

Vertical Anchorage Requirements in the Transverse Direction

52a. In order to determine the Required Vertical Anchorage the user needs the data from the following items on the Design Worksheet: Nos. 24, 31, 32, 43 and 48. With this information, the field officer can locate and determine the Required Vertical Anchorage tables in Appendix B, Part 2.

Use the tables for a Type C1 system. Then multiply Av x Tie-down spacing.

<u>Item No.</u> <u>Data</u>

#24 Foundation System: Type C1 - Single-section

#31 Design Windspeed: 100 mph

#32 Site Location: Coastal

#43 Building Nominal Width: 14'-0"

#48 Tie-down Spacing: s_t = 8'-8". Number of tie-downs is 7 from (N):

$$N = \frac{L - 2 \times 2'}{s_t} + 1$$

1) The user locates the Required Vertical Anchorage (Appendix B, Part 2) tables for a Single-section Type C1 with a nominal width of 14 feet.

2) Then the user finds a block of values for the Coastal condition.

3) Locate the row for a wind speed 100 mph. The user finds the required vertical anchorage Av = 350 lbs./ft. of home length and multiplies this by a tie-down spacing of 8.667 feet (3033 lbs.) or conservatively move across the row to the next largest anchor spacing (10') and reads 3460 lbs. as an approximate check.

4) The Required Vertical Anchorage force for a tie-down is 3033 lbs.

54. The manufacturer's supplied value, item #53, is 3,150 pounds, which is more than the Required Vertical Anchorage of 3,033 pounds. Note: see optional details in Appendix A for Type **C1**. If the manufacturer's supplied value had been less than Av, the owner would have been notified. The owner would need to contact the manufacturer in order to have a licensed structural engineer verify the existing design or modify the anchor design or spacing to comply with the required anchorage.

Horizontal Anchorage in the Transverse Direction

56. In order to determine the Required Horizontal Anchorage, the user needs data from the same items on the Design Worksheet that were required for Approval item number 52a and item No. 22 (the building length L = 56'-0"). Also required is item #9 (6'-10") from the Manufacturer's Worksheet.

Next, the user will locate the Required Horizontal Anchorage table in the transverse direction in Appendix B, Part 3.

1) The user locates the tables for a Single-section Type C, E or I with a nominal width of 14 feet and initially selects two transverse walls for trial #1. This is required to initiate the process of selecting vertical X-bracing planes for horizontal anchorage in the transverse direction.

2) Then the user finds the block of values for the Coastal condition and the row of values for a design wind speed of 100 mph. All Seismic is on the same horizontal line, even though it need not be checked.

3) For a length L of 56 feet, the user rounds the value to the next highest number shown on the top row of the table -- 60 feet.

4) Under the column of values for 60 feet, the user finds the required anchorage (Ah) of 1240 pounds per lineal foot

along the length of each transverse foundation wall (2 shear walls).

59c. The required horizontal anchorage per X-brace set (C) is calculated using the procedure of Section 602-5.G.2, illustrated in Figure 6-10.

Process always begins by selecting 2 short walls, then:

1. From item #56, Ah = 1240 lbs./ft.

2. Solving equation for H:

$$H = \frac{1240 \times 13.67 \times 2}{56} = 605 \text{ lbs./ft. of unit length}$$

Note: actual unit width, rather than nominal width is used here.

3. For a first trial, set spacing equal to a multiple of pier spacing: try 14'-0". Solving equation for horizontal force at each X-brace set (C):

C = 605 × 14'-0" = 8475 lbs./X-brace set.

Note: number of vertical X-brace planes =

$$\frac{L}{\text{spacing}} + 1 = \frac{56}{14} + 1 = 5$$

therefore, number of X-braced planes equals 5.

61a. Verify that the Manufacturer's design value on line #57a is greater than the required value (C) shown on line #59c. In this example, the manufacturer's design value of 4800 lbs. (#57) is less than the Required Horizontal Anchorage (C) = 8475 lbs. This indicates that the connection of unit to a foundation diagonal is inadequate for sliding resistance.

The owner would be contacted at this point and notified that the horizontal anchorage is not adequate. If an inspector or owner wanted to determine how many vertical X-bracing planes would be required, they could use the following:

<u>Trial #2:</u>

Piers must be present at the extremities of any vertical X-bracing plane; therefore, the next logical choice is the actual pier spacing of 7'-0".

1. From item #56, Ah = 1240 lbs./ft.

2. Solving equation for H:

$$H = \frac{1240 \times 13.67 \times 2}{56} = 605 \text{ lbs./ft. of unit length}$$

Note: actual unit width, rather than nominal width is used here.

3. C = 605 × 7'-0" = 4235 lbs./X-brace set.

Number of vertical planes =

$$\frac{56'}{7'} + 1 = 9$$

The required horizontal anchorage of 4235 is less than the manufacturer's rated capacity of 4800 lbs., thus 9 vertical X-bracing planes are required at the same spacing as the piers (7'-0").

59d. The user must estimate a height (h) on Figure 6-10, which can be revised later if necessary. Try h = 4 feet.

59e. From item #9, Manufacturer's Worksheet, Wt - 2 dc = 6.83':

$$\cos\theta_t = \frac{6.83}{\sqrt{4^2 + (6.83)^2}} = 0.863$$

therefore: $\theta_t = 30.4°$

$$T_t = \frac{4235}{0.863} = 4907 \text{ lbs. tension in strap}$$

61b. The rated capacity of a strap in tension, item #60 is greater than the required T_t (item #59e) for 9 vertical X-bracing planes 5600 > 4907, therefore OK.

Horizontal Anchorage Requirements in the Longitudinal Direction

62a. In order to determine the Required Horizontal Anchorage (Ah) in the Longitudinal Direction, the user needs data from the same items in the Design Worksheet that were required for item #56.

Next, the user will locate the Required Horizontal Anchorage Table in the Longitudinal Direction (Appendix B, Part 4).

1) The user locates the table for a Single-section, Type C, E, or I with a nominal width of 14 feet.

2) Then, the user finds the block of values for <u>Aa</u> = 0.05-0.10, ground snow 0-100 psf and coastal site.

3) The user finds the row of values for wind speed of 100 mph.

4) For a length (L) of 56 feet, the user rounds to the next highest length shown across the top row of the table - 60 feet.

5) Under the column for 60 feet, the user finds the intersection value with the row for 100 mph wind speed. Read Ah = 47 lbs./ft. of length along the longitudinal exterior foundation walls, if shear walls exist.

62b For this example of a Type C1 foundation, no structural exterior longitudinal walls exist, thus vertical X-bracing planes are required between piers under chassis beam lines. Follow the procedure of Section 602-6.F and use the illustration in Figure 6-11 and Figure D-26.

Begin Trial 1 with the minimum required vertical X-bracing planes: n = 2; one pair under each chassis at both ends of the unit length. Follow the equation:

$$B = \frac{47 \text{ plf} \times 56}{2} = 1316 \text{ lbs. of horizontal}$$

force carried by each X-brace set.

64. Verify that the manufacturer's rated value (item #63) is greater than the required horizontal anchorage force (B) of item #62b.2. In this example the manufacturer's value of 4800 lbs. is greater than B. Thus, vertical X-bracing planes at both ends of the unit and under each chassis beam line is adequate.

62b.3 Approximate the height (h) in Figure 6-11 by assuming the chassis beam is 1 foot deep, thus: h = 4' - 1' = 3'.

62b.4 Return to the calculation procedure of section 602.6.F and determine the tension force in a diagonal strap:

$$\text{first: } \cos\theta_L = \frac{7}{\sqrt{3^2 + 7^2}} = 0.919$$

therefore: $\theta_L = 23.2°$

next: $T_L = \dfrac{1316}{.919} = 1432\,\text{lbs.}$

66. Verify that the manufacturer's (or product supplier) rated value (item #65) is greater than the required tension (T_L) from item #62b.4. In this example, the manufacturer's value of 5600 lbs. is greater than (T_L). Thus, the straps proposed are adequate as tension diagonals.

Withdrawal Resistance Verification

67b. For Type C1 foundation answer item 67b for concrete "deadman" footings.

For this example, square concrete footings used as "deadmen" are depicted on the documents submitted by the owner to anchor the tie-down straps. See Appendix A - concept details for Type C1 foundation.

1. From item number 52a, the value for Av is 3033 lbs. per tie-down anchor.

2. Use Table C-2, The Withdrawal Resistance for Piers, in Appendix C. Table C-2 can conservatively be used for concrete footings used as "deadman" anchors. The footing depth (hp) in the far left column can either be hp = 3'-4" for a 3'-0" sq. ft. footing or hp = 2'-0" for a 4'-0" sq. ft. footing. Assume the least costly solution is the 3'-0" square footing.

3. Based on item #9, the frost depth for Tampa, FL. is 0". Thus, the "deadman" footings are at an adequate depth. The pier footings under the chassis beams can be set 8" below grade, if undisturbed soil (not organic material) is available, otherwise, footing must extend to firm bearing strata.

Vertical Anchorage and Reinforcement for Long Foundation Walls and Piers

68. For type C foundations answer item 68b.

1) From item number 52a, the value for Av is 3033 lbs. per foot. The lowest value greater than Av on Table C-3 is 4240 pounds.

2) For the size of bolt set in concrete "deadman" to complete connection to the tie-down rod, from Table C-3:

   a) Number of anchor bolts = 1

   b) Anchor bolt diameter = 1/2"

3) Use Table C-3A for the reinforcement of the piers under the chassis beams. Even though these piers do not directly receive anchorage overturning force, it is desirable to reinforce them to assist in force distribution in the vertical X-bracing planes.

   a) Rebar size = #4

   b) Lap splice = 16"

   c) Rebar hook length = 6"

Horizontal Anchorage and Reinforcement for Transverse Foundation Walls

69c. From item number 59c (Assume the owner decided to use 9 X-bracing planes), the value for (C) is 4235 lbs. per diagonal. Use Table C-5A for concrete. The horizontal capacity of a single bolt is shown at a spacing of 12".

Bolt size	Capacity
1/2"	1800 lbs.

Three 1/2" bolts would be required to connect the diagonal to the footing. Detail the pier footing as shown in Table C-5A. Verify that the rated capacity of the strap exceeds the required tension (T_t).

Horizontal Anchorage and Reinforcement for Longitudinal Foundation Walls

70b. From item #62b.2, record the horizontal anchorage force (B) as 1316 lbs. per X-brace. Again, from Table C-5A, the shear capacity of a 1/2" diameter bolt in concrete is 1800 lbs. One anchor bolt is sufficient into the concrete footing. Detail the pier footing as shown in Table C-5A. Verify that the rated strap capacity exceeds the required tension (T_L).

Summary Sheet

The values can be brought forward on to the summary sheet and the design approved.

APPENDIX E
OWNER'S SITE ACCEPTABILITY WORKSHEET

Owner's Name:	JOHN DOE
Address:	1600 S. FIRST ST.
	CHAMPAIGN, IL
Telephone:	
Site Location:	CHAMPAIGN, IL
Legal Description:	

Have you provided a copy of a map pinpointing the site? yes no

Have you submitted a foundation plan? yes no
(See #10 of Manufacture's Worksheet)

Preliminary Site Information

Before approval of the site can begin, the applicant must provide preliminary site information to the field office. Refer to Chapter 2, "Site Acceptability Criteria" for clarification.

1. Provide survey results showing existing grade elevation. (201-1) N.A. ft.
2. Is the building in a flood-prone area? (201-2) yes no
   If the answer to 2 is Yes, answer 3, 4, & 5.
   If the answer to 2 is No, answer 6, below.

3. What is the Base Flood Elevation? — ft.

What is the Flood Protection Elevation? — ft.

4. Has approval for drainage, grading and berming been approved for flood-prone sites? yes no

5. Have permits been provided? yes no
(Permits must be obtained for any alteration of the building site in a flood protection area.)

6. Provide geotechnical report in areas of known high water table. (201-4) yes no

7. Provide geotechnical report if adverse site conditions are found or suspected. (203) yes no

8. Provide site-drainage plan complying with CABO R301.3 or local requirements. (301) yes no

9. Provide fill specifications if site is to be prepared with earth fill. (303-2) yes no

10. If a geotechnical report is required, what is the net allowable soil bearing pressure? (202) — psf.

11. If no adverse soil conditions are known or suspected, and if the home is individually sited, assume a soil bearing pressure of 1,000 psf. and use this value when a determination of soil bearing pressure is called for. 1,000 psf.

APPENDIX E
MANUFACTURER'S WORKSHEET

Manufacturer's Company Name: HOWARD SMITH CO., INC.

Address: 1904 W. 75TH ST.
NEW YORK, N.Y. 10031

Telephone: (314) 329 - XXXE

Determination of Building Structure and Size

The manufacturer shall provide the following information:

1. Type of unit — Single-Section / Multi-Section
2. Method, location and types of support:
   Refer to Figures 6-7 and 6-8 and Section 601-4
   Is the home a **C, E, or I**? E
3. Length of unit L — 56 ft.
4. Actual width of unit Wt — 13'-8 ft.
5. Height of exterior wall ** — 7'-6 ft.
6. Height of roof peak ** — 2'-3 ft.
7. Roof slope ** — 2 :: 12
8. Self weight of total unit (W) including mechanical equipment ** — 33040 lbs.
9. Distance between chassis members — 6'-10 ft.
10. One foundation design concept (See Appendix A) **(C1-C4; E1-E8; or I)** — E1

11. Recommended pier spacing **

a. Exterior 5.0 ft.

b. Interior 5.0 ft.

c. Continuous Marriage Wall 8.0 ft.

Length of largest isolated marriage wall opening or average of largest two adjacent openings $\frac{16+12}{2} = 14$ 14.0 ft.

d. Tie-down Strap (C1 concept only) N.A. (Number) N.A. ft. (Spacing)

12. One installation method recommendations (include documentation showing connection details pertinent to geographic area for seismic or wind). ** yes no

13. Interior shear wall locations (include documentation showing locations). ** yes no

14. Design wind speed used in designing connection details for horizontal anchorage (Ah) and vertical anchorage (Av) in the transverse direction. ** 100 mph.

15. Seismic acceleration values used in designing connection details for horizontal anchorage (Ah) in the transverse and longitudinal directions. ** Av 0.05 Aa 0.05

16. Shear wall connection details with rated capacity for wind and seismic are provided. ** † yes no

a. Connection locations at foundation end and interior walls shown? ** yes no

b. Rated connection capacity for uplift and overturning ** 200 lbs./ft. (or lbs./tie-down)

c. Rated connection capacity for sliding in transverse direction ** 400 lbs./ft. (or lbs./diag. strap)

d. Rated connection capacity for sliding in longitudinal direction ** 400 lbs./ft.

e. Vertical X-bracing tension strap capacity ** N.A. lbs./diag. strap

f. Engineering calculation by licensed structural engineer? ** yes no

**** Optional values:** It is optional for the manufacturer to provide these values. If the manufacturer does not provide the values, it is the responsibility of the owner to supply values, based on engineering analysis by a licensed structural engineer.

† Item 16 is provided in California.

APPENDIX F
DESIGN WORKSHEET

Owner's Name: JOHN DOE

Address: 1600 S. FIRST ST., CHAMPAIGN, IL

Builder's Name: ACME, LTD

Site Location: CHAMPAIGN, IL

PART 1: SITE CONDITIONS
(Accompanies Chapter 2)

1. Has the Manufacturer's Worksheet been provided? yes no

Existing Grade Elevation (201-1)

2. Does the site require a survey? yes no
   (Answer yes if: 1) elev. to be altered by grade or fill; 2) site near flood zone; 3) subdivision. Answer no if individually-sited with no alteration of building site.)

3. If yes to above, what is the surveyed existing grade elevation? N.A. ft.

Flood Protection Elevation (201-2)

4. Is the building site in a flood zone? yes no
   (If yes to 4, then answer 5, 6, 7 & 8. If no, skip to 9.)

5. What is the Base Flood Elevation or the Flood Protection Elevation (use highest value)? – ft.

6. Is the site to be graded, filled, or bermed? yes no
   (If no, skip to 9.)

7. If yes to 6, have all permits been provided? yes no

8. If no to 6, then are the buildings to be built on elevated foundations? yes no
   (If yes, this handbook cannot be used. Refer to FEMA Manual.)

Frost Penetration Depth (201-3)

9. What is the maximum frost penetration depth? (see Appendix H, page H-4) — 30 in.

10a. Does foundation plan show base of footing extending below frost penetration depth? (If yes proceed; if no, applicant should revise plans.) — yes no

10b. Does foundation plan show base of footing extending below top-soil layer (min. 12") to undisturbed soil? — yes no

Ground Water Table Elevation (201-4)

11. For subdivisions, does a Geotechnical Engineer recommend drainage of subsurface water? (If no, skip to 13.) — yes no

12. Has groundwater drainage plan been provided? — yes no

Soil Conditions (202, 203)

13. If any of the following adverse site conditions are discovered, specific recommendations by a Geotechnical Engineer will be required (applies to subdivisions and individually-sited homes.)

Organic soil (8" topsoil layer)	18" layer of topsoil as a site condition	yes	no
Expansive (shrink-swell) soil		yes	no
Sloping site		yes	no
Subsidence		yes	no

(Applicant may be referred to Geotechnical Engineer if any of the above are yes. If no, to all of above, move to next step.)

14. Is area in a known termite infestation area? — yes no

Region classification? MODERATE TO HEAVY
(See Appendix H, Termite Infestation Map, page H-10) (If no, skip to 16.)

15. Has applicant complied with CABO R-308 or local ordinance for construction procedures and treatment? (If yes, continue; if no, refer applicant to CABO requirements.) — yes no

PART 2: SITE PREPARATION

(Accompanies Chapter 3)

16. Acceptable surface drainage plan provided? (301) (If no, one must be provided for subdivision) — yes no

17. Grading plan provided? (302) — yes no

18. Fill specifications conforming to those cited in HUD Land Planning Data Sheet (79g)? (303) (If fill is used, below the home's foundation, a report by Geotech. Eng. should be submitted to provide fill specifications.) — yes no

19. Finish grade elevation? (304) (Check answers to Part 1: #4 & #5. The finish grade elevation must be higher than #5 if in flood zone.) — ______–______ *

PART 3: DESIGN LOADS

(Accompanies Chapter 4)

Information from Manufacturer's Worksheet

20. Has all the information been provided on the Manufacturer's Worksheet? (Appendix E) — yes no

21. What is the building self weight (W)? (Mfg. Wksht. #8) — 33,040 lbs.

22. What is the building length (L)? (Mfg. Wksht. #3) — 56 ft.

23. What is the distributed weight per foot of unit length? (w=W/L) (402-1.B, C) — 590 lbs./ft.

24. What is the building type? (Mfg. WkSht. #2) — Single-Section / Multi-Section

    C, E, or I

    Foundation design concept? (C1, C2, C3, C4, E1, E3, E4, E5, E6, E7, E8, I) — E1 *

Dead Load (402-1)

25. What is the light dead load value from Table 4-1? (402-1.A.1) — 560 * (lbs./ft.)

26. What is the heavy dead load value from Table 4-1? (402-1.A.2) — 805 * (lbs./ft.)

27. Does the answer from Question #23 fall within the values in #25 and #26? (402-1.D) (If the answer is yes, continue. If no, the foundation is not within the limits of this document and must be redesigned by a structural engineer.) — yes no

Snow Load (402-2) / Minimum Roof Live Load (402-2.C)

28a. What is average annual ground snowfall (Pg)? (See Ground Snow Load map, pages H-11, H-12 and H-13.) — 20 * (lbs./sq.ft.)

28b. What is 0.7 multiplied by Pg? — 14 psf.

29a. What is the roof slope? (Mfg. Wksht. #7) — 2::12

29b. What is the minimum roof live load for the roof slope? (D-200.2.B) — 20 psf.

30. Record the larger magnitude of item 28b or item 29b. Use this magnitude for roof load where required. — 20 psf.

Wind Load (402-3)

31a. What is the basic wind speed (V)? (See Wind Speed map, page H-14.) — 70 mph.

31b. If V is less than 80 mph, record *MPS* min. 80 mph for wind design. (402-3.A) — 80 mph.

32. Is the site inland or coastal? (402-3.B) (If inland, skip to question #38.) — Inland Coastal

33. If a coastal area, has the manufacturer provided connection details? (402-3.D) (Mfg. Wksht. #12) — yes no

34. If yes to #33, what design wind speed has the manufacturer used in designing connection details? (Mfg. Wksht. #14) ________ mph. *

35. Are the connection locations shown? (Mfg. Wksht. #16a) yes no

36. Are connection details provided for foundation shear walls? (For an answer of yes, all questions under Mfg. Wksht #16 must be answered satisfactorily.) yes no

37. Is the value for Question 34 equal to or greater than the number given in Question 31? (If yes, proceed. If no, return design to manufacturer for clarification.) yes no

Seismic Load

38a. What are the seismic acceleration values? (See Seismic maps, pages H-15 and H-16)

Aa 0.05 *

Av 0.05 *

38b. Is Av < 0.15? (if no, proceed. If yes, seismic need not be considered, skip questions 39 to 41.) yes no

39. Seismic performance category. (See H-300 for Special Requirements of Foundation Design.) N.A.

40. What is the applicant's proposed design concept? (Design Wksht. #24) N.A. *

41. Do the Foundation Design Concept Tables approve the foundation system for use in seismic areas of Question #38 above? (See Appendix A) (If yes, proceed. If no, return to applicant for foundation design choice more suited to high seismic areas.) yes no

PART 4-FINAL DESIGN PROCEDURE

(Accompanies Chapter 6)

42. What is the actual building width? (Mfg. Wksht. #4) 13'-8" ft.

43. The nominal building width to be used in the Foundation Design Tables, (Aftg, Av & Ah) is Wt: (600-2.A and Figure 6-1) — 14'-0 ft.

44. Where are the foundation supports located? Check drawings submitted by the owner and Foundation Design Concepts in Appendix A. Circle the support locations shown on the Manufacturer's foundation concept plan.

Chassis Beams
Exterior Walls
Marriage Wall

45. Do these locations match the Foundation Concept shown in Appendix A? Do the locations match Question #24 on the Design Worksheet?
(If yes, proceed. If no, return to Owner for clarification.) — yes no

46. Is Vertical Anchorage present?
(601-2.B, 601-3.B & 601-4.B (Figures 6-7 & 6-8); Mfg. Wksht. #12 & #16) — yes no

APPENDIX A

47. What is the basic system type?
(From Part 3: #24; Mfg. Wksht. #2) — E1 *

48. What is the spacing between piers?
(Mfg. Wksht. #11)
(602-2)

Exterior: 4' 5' 6' 7' 8'

Interior: 4' 5' 6' 7' 8'

Continuous Marriage Wall: 4' 5' 6' 7' 8'

$\frac{16+12}{2} = 14'$ Largest or Average Marriage Wall Opening: 14 ft.

Tie Down (C1) N.A. ft.

APPENDIX B

Required Footing Size

49. The required Exterior Wall Footing, for the foundation type, is found in the Required Effective Footing Area table in App. B, Part 1. (Use maximum value from item #30.) — E1 *

The Required Exterior Square Footing size is: Type C N.A. sq.ft.

Type E or I 1.0 ft. MIN.
(width)

50. The Required Interior Footing area is: (Also exterior piers for foundation type E) 2.1 sq.ft.

51a. The Required Continuous Marriage Wall Footing area is: 6.9 sq.ft.

51b. The Required Footing area under posts at the ends of marriage wall opening(s) is: 11.4 sq.ft.

Vertical Anchorage Requirements in the Transverse Direction (602-4)

52a. Using the Foundation Design Load Tables (Appendix B, Part 2), determine the Required Vertical Anchorage.
Exterior Av 130 *
(lbs./pier spacing;
lbs./ft for E type;
lbs./tie-down spacing)

52b. Number of vertical tie-down locations for multi-section units: 2 or 4 or 6

52c. For units with additional vertical anchorage at the interior piers, determine the Required Vertical Anchorage.
Interior Av N.A *
(lbs./int pier spacing)

53. What is the manufacturer-supplied value? (#16b, Mfg. WkSht.)
Exterior 200 * lbs/ft
Interior N.A *

54. Is this value (#53) greater than the value given in #52a? (If yes, continue. If no, return to owner for clarification.) yes no

Horizontal Anchorage Requirements In The Transverse Direction (602-5)

	trial 1	trial 2	trial 3	
55a. What number of transverse foundation walls was selected? (602-5.E) (If vertical X-bracing planes are used, complete items #55a, #56 and #57 for 2 transverse walls, and then skip to item #59.)	2	4	6	
55b. Are diagonal ties used to complete the top of the transverse short wall for horizontal anchorage? (602-5.G.1)	yes no	yes no	yes no	
Estimate height (h) for appropriate illustration in Figure 6-10.	N.A.			ft.

Item		trial 1	trial 2	trial 3	
56. Using the tables, find the Required Horizontal Anchorage (Ah). (Appendix B; Part 3)	End Wall Ah	420	140		lbs./ft.
	Int Wall Ah	-	280		lbs./ft.
57a. What is the manufacturer's-supplied rated capacity for sliding? (#16c, Mfg. WkSht.)		400	400		lbs./ft.
57b. If answer to item #55b is yes, record manufacturer or product supplier rated strap tension capacity		N.A.	N.A.		lbs./strap
58a. Is value #57a greater than item #56? If yes, continue. If no, return to section 602-4.C and to question #55a and select a larger number of transverse foundation walls. If the maximum number selected (6) does not work, return to owner (who may wish to contact the manufacturer for clarification).		yes / no	yes / no	yes / no	
58b. If answer to #55b is yes, required tension in diagonal (T_t). (Complete procedure in Section 602.5.G.1.)			N.A.		lbs.
58c. Is value #57b greater than #58b? If yes, continue to item #62. If no, return to owner for product with greater capacity.		yes / no	yes / no	yes / no	

59. If using vertical X-bracing planes in lieu of transverse short walls (and the formulas in section 602-5.G.2), determine anchorage values and sizes for diagonal members. (If shear walls are selected in item #55, skip to item #62.)

Item	trial 1	trial 2	trial 3	
a. Vertical X-bracing spacing proposed.	N.A.	N.A.		ft. *
b. Number of vertical X-bracing locations proposed. (Item #13, Mfg. WkSht. for trial 1.)				*

		trial 1	trial 2	trial 3	
c.	Required horizontal anchorage (C) value, based on formula. (602-5.G.2.c)				lbs./ x-brace set
d.	Estimated height (h) in Figure 6-10.				ft.
e.	Tension (T_t) required. (602-5.G.2.d)				lbs./diag.
60.	What is the manufacturer-supplied rated strap tension capacity? (#16, Mfg. WkSht.) (or capacity defined by literature supplied by product supplier)				lbs. *
61a.	Is value #57 greater than value #59c? If yes, continue. If no, return to Section 602-5.G and to question #59 and select a greater number of X-brace locations as a next trial. Repeat until answer is yes, then continue.	yes no	yes no	yes no	
61b.	Is value #60 greater than value #59e? If yes, continue. If no, return to section 602-5.G and to question #59 and select a greater number of X-bracing locations. If the maximum number selected does not work, return to owner (who may wish to contact the manufacturer for clarification or product supplier for clarification).	yes no	yes no	yes no	

Horizontal Anchorage Requirements In The Longitudinal Direction (602-6)

62a. Using the tables, find the required horizontal anchorage (Ah) in the longitudinal direction. (Appendix B, Part 4) (602.6.E) Exterior Wall Ah 67 lbs./ft.

62b. If using vertical X-bracing planes (and the formulas in section 602-6.F) determine anchorage value for X-bracing planes. (If using exterior long walls, skip to item #63.)

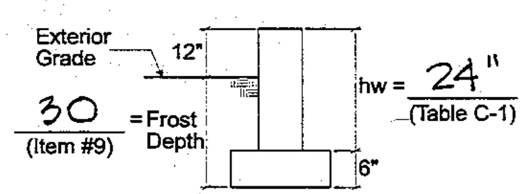
		trial 1	trial 2	trial 3
1.	Number of chassis beam lines used for vertical X-bracing planes.	2 or 4	2 or 4	2 or 4

	trial 1	trial 2	trial 3	
Number of X-bracing planes proposed under each chassis beam along the length of the unit.				
2. Horizontal anchorage (B) required force, based on formula.				lbs.
3. Assumed height (h-b) based on Figure 6-11.				ft.
4. Tension (T_L) based on formula. (602-6.F.(3)).				lbs.
63. What is the manufacturer-supplied value for horizontal anchorage? (#16d, Mfg. WkSht.)	400			lbs./ft.
64a. For shear walls: is value #63 greater than #62a? If yes, skip to item #67. If no, contact owner for clarification.	yes / no	yes / no	yes / no	
64b. For X-bracing: is value #63 greater than value #62b.2? If yes, return to item #62b.3. If no, increase number of vertical X-bracing planes and repeat items 62b.1 and 62b.2 until answer is yes. For multi-section units consider 4 lines of vertical X-bracing under all chassis beams.	yes / no	yes / no	yes / no	
65. What is the manufacturer-supplied rated strap tension? (#16e, Mfg. WkSht. or product supplier)				lbs.
66. Is value #65 greater than #62b.4? If yes, continue. If no, contact owner to obtain straps with greater capacity, or return to item #62b.1 and increase the number of vertical X-bracing planes until answer is yes.	yes / no	yes / no	yes / no	

APPENDIX C

Withdrawal Resistance Verification (603-2.B)

67. Using Appendix C, Table C-1 or C-2, verify that the foundation system will resist withdrawal. Answer question #67a for type E. Answer question #67b for types C, I, or type E with interior pier anchorage.



a. ***Withdrawal Resistance for long foundation wall.*** (Type E)
Circle the type of material that is to be used.

Reinforced Concrete
Masonry-Fully Grouted
Masonry-Grouted @ 48" o.c.
All-Weather Wood / Footing

1) Using Table C-1, which capacity is greater than required Av? (603-2.B.(1)) (#52a) — 231 lbs./ft.

2) Using Table C-1, what is the height of the wall + footing for required withdrawal resistance? (hw + 6") 24 — 30 in.

3) What is the height of the wall + footing for frost protection? (frost depth (#9) + 12") 30 — 42 in.

4) What is the greatest height #67a.2 or #67a.3? — 42 in.

Circle the height which controls.

Withdrawal
Frost Depth

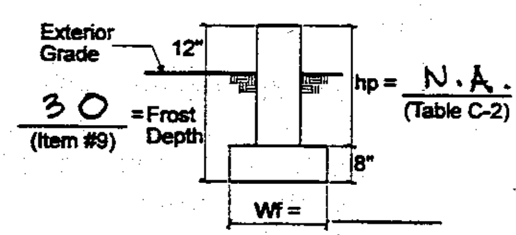
5) Record the bottom of footing depth from grade. (Item #67a.4 - 12") 42 — 30 in.

6) Using Table C-1, what is the required width of the wall footing for withdrawal? — 12" in.

7) Is item #67a.6 greater than or equal to item #49? If yes, continue. If no, change footing width to item #49. — yes no

8) Record design exterior wall footing width. — 12" in.

b. ***Withdrawal Resistance for Piers.*** (Types C, C1 (concrete dead-man), I or type E with interior pier anchorage - multi-section units.)



Circle pier type:

Reinforced Concrete
Reinforced Masonry - fully grouted
Reinforced Concrete Dead-man

		Exterior	Interior (when used)	
1)	Using Table C-2, which capacity is greater than required Av? (#52a and #52c) (603-2.B.(2))	N.A.		lbs./pier *
2)	Using Table C-2, what is the height of the pier + footing for required withdrawal resistance? (hp + 8")	N.A.		in. *
3)	What is the required height of pier + footing for frost protection? (frost depth (#9) + 12") N.A.	min. 18	min. 18	in. below topsoil @ undisturbed soil
4)	What is the greatest height #67b.2 or #67b.3?			in.
	Circle the height which controls.	~~Withdrawal~~ Frost Depth	~~Withdrawal~~ Frost Depth	
5)	Record the bottom of footing depth from grade. (Item #67b.4 - 12")	min. 18	min. 18	in.
6)	Using Table C-2, what is the required width of the square footing if withdrawal resistance controls or if frost depth controls?	N.A.		in. *

c. ***Frost depth for marriage walls.*** What is the required depth of footing below grade for frost protection? (frost depth (#9)) (no withdrawal resistance) — min. 18 in. below topsoil @ undisturbed soil level

Vertical Anchorage and Reinforcement for Longitudinal Foundation Walls and Piers (603-2.D)

68. Using Appendix C, Table C-3, C-4A or C-4B, verify that the foundation anchors will resist uplift. Answer question #68a for type E. Answer question #68b for types C, I, or type E with interior pier anchorage.

   a. ***Vertical Anchor Capacity for longitudinal foundation wall*** (type E). (603-2.D.2)

      1) Using Table C-4A (concrete & masonry), which capacity is greater than the required Av? 130 (#52a, Design Wksht.) If treated wood wall, skip to item #68a.3. — 146 lbs./lineal ft. of wall

Circle correct washer choice for the capacity selected

Standard Washer
Oversized Washer

2) Using Table C-4A (masonry and concrete):

a) Required anchor bolt diameter — ½" φ in.

b) Required anchor bolt spacing — 72 in.
max. allow.

c) Using Table C-3A:

(1) Rebar size — #4 *

(2) Lap splice — 16 in.

(3) Rebar hook length — 6 in.

3) Using Table C-4B (wood), which capacity is greater than the required Av? (#52a, Design Wksht.)
If using concrete or masonry wall, skip to item #68b. — N.A. lbs./lineal ft. of wall

4) Using Table C-4B (wood):

a) Required nailing ________ *

b) Minimum plywood thickness ________ in.

c) Required anchor bolt diameter ________ in.

d) Required anchor bolt spacing ________ in.

b. *Vertical Anchor Capacity for Piers*
(Types C, I, or type E with interior pier anchorage)
(603-2.D.1)

	Exterior	Interior (when used for anchorage in multi-section units)	
1) Using Table C-3, which capacity in the table is greater than the required Av? (From #52a, Design Wksht.)	N.A.	________	lbs./pier

	Exterior	Interior	
2) Using Table C-3:			
a) Number of anchor bolts	1 or 2	1 or 2	
b) Anchor diameter	1/2" or 5/8"	1/2" or 5/8"	
3) Using Table C-3A:			
a) Rebar size	#4 or #5	#4 or #5	
b) Lap splice	______	______	in.
c) Rebar hook length	______	______	in.

Horizontal Anchorage and Reinforcement for Transverse Foundation Walls (603-3)

69. Using Appendix C, Table C-5A or C-5B, verify that the foundation anchorage will resist sliding at the transverse end foundation walls. Use for types C, E, or I.

	End Wall	Interior Wall	
a. *For continuous foundations.*			
Using Table C-5A (concrete & masonry) or C-5B (wood), which capacity is greater than the required (Ah) (603-3) (item #56)? 140 280	300	300	lbs./ft.
1) Using Table C-5A, find:			
a) Required anchor bolt diameter	½ φ	½ φ	in.
b) Required anchor bolt spacing	72	72	in.
	max allow.		
c) Using Table C-3A:			
(1) Rebar size	#4	#4	*
(2) Lap splice	16	16	in.
(3) Rebar hook length	6	6	in.
2) Using Table C-5B, find:			
a) Required nailing	______	______	*

	End Wall	Interior Wall	
b) Minimum plywood thickness	______	______	in.
c) Required anchor bolt diameter	______	______	in.
d) Required anchor bolt spacing	______	______	in.

b. ***For transverse short foundation walls completed with diagonal braces.*** (603-5)

Using Appendix C, Table C-5A, verify the diagonal anchorage capacity to the short foundation wall.

	End	Interior	
1) Record the required horizontal force (Ah × Wt) from 602-5.G.1.a and item #56.	N.A.	______	lbs.
2) Table C-5A capacity for one 1/2" diameter bolt at 12" o.c.	1800	1800	lbs.
3) Number of bolts (Ah × Wt ÷ 1800; one minimum) at concrete or masonry top of short wall.	______	______	*
4) Size of anchor bolts	______	______	in.
5) Using Table C-3A:			
a) Rebar size	______	______	*
b) Lap splice	______	______	in.
c) Rebar hook length	______	______	in.

c. ***For vertical X-bracing planes in the transverse direction.*** (603-6)

Using Appendix C, Table C-5A, verify the diagonal anchorage to the pier footings and the tension capacity of the diagonals.

1) Record the required horizontal force (C) from item #59c. N.A. lbs.

2) Table C-5A capacity for one 1/2" diameter bolt at 12" o.c. 1800 lbs.

	End Wall	Interior Wall	
Anchor bolt spacing	72	72	in.
Rebar size	#4	#4	
Lap splice	16	16	in.
Rebar hook length	6	6	in.
From #69a.2: wood:			
Required nailing	-		
Minimum plywood nailer			
Anchor bolt diameter			
Anchor bolt spacing			in.

2. *For transverse short foundation walls completed with diagonal braces* (#69b)

	End	Interior	
Number of pairs of diagonals (1 for single-section units, 2 for multi-section units) times number of short walls (end or interior) (#55a)	-		
Diagonal spacing (same as number of short walls)			
From #69b: concrete / masonry:			
Anchor bolt diameter	-		in.
Number of bolts			
Rebar size			
Lap splice			in.
Rebar hook length			in.

3. *For vertical X-bracing planes in lieu of short walls.* (#69c)

Number of X-brace locations (#59)	-

Spacing of vertical X-brace planes (#59) __________ ft.

<u>Items from #69c.3 and #69c.5</u>

Required anchor bolt diameter __________ in.

Number of bolts at top of footing to connect diagonal __________

Diagonal strap size __________

Connection to top flange of chassis beam (describe) __________________

c. **Horizontal anchorage in the longitudinal direction - exterior foundation walls**

1. *Continuous foundation walls*

<u>Reinforcing for longitudinal foundation walls:</u> record only if larger sizes or closer spacing than recorded for vertical anchorage (#71a.2).

<u>From #70a.1: concrete / masonry:</u>

Anchor bolt diameter same in.

Anchor bolt spacing __________ in.

Rebar size __________

Lap splice __________ in.

Rebar hook length __________ in.

<u>From #70a.2: wood:</u> record only if larger sizes or closer spacings than recorded for vertical anchorage (#71a.2)

Required nailing __________

Minimum plywood nailer __________

Anchor bolt diameter __________

Anchor bolt spacing __________ in.

2. *Vertical X-bracing planes under chassis beam lines* (#70b.)

Number of X-brace locations along one chassis beam line. N. A.

Spacing of X-brace locations along one chassis beam line. ______ ft.

Required anchor bolt diameter. ______ in.

Number of bolts at top of footing at connection to the diagonal. ______

Diagonal strap size. ______

Connection to bottom flange of chassis beam (describe). ______

72. Do foundation dimensions and details comply with Foundation Capacities Table, based on Foundation Design Table Values? yes no

73. If #72 yes, approve. If no, return to applicant. **APPROVE**

DISAPPROVE

APPENDIX E
OWNER'S SITE ACCEPTABILITY WORKSHEET

Owner's Name: JOHN SMITH

Address: 35 BRANDYWINE
TAMPA, FL

Telephone: ______

Site Location: ______

Legal Description: ______

Have you provided a copy of a map pinpointing the site? yes no

Have you submitted a foundation plan? yes no
(See #10 of Manufacture's Worksheet)

Preliminary Site Information

Before approval of the site can begin, the applicant must provide preliminary site information to the field office. Refer to Chapter 2, "Site Acceptability Criteria" for clarification.

1. Provide survey results showing existing grade elevation. (201-1) 28 ft.
2. Is the building in a flood-prone area? (201-2) yes no
   If the answer to 2 is Yes, answer 3, 4, & 5.
   If the answer to 2 is No, answer 6, below.

3. What is the Base Flood Elevation? ____________ ft.

   What is the Flood Protection Elevation? ____________ ft.

4. Has approval for drainage, grading and berming been approved for flood-prone sites? yes no

5. Have permits been provided? yes no
   (Permits must be obtained for any alteration of the building site in a flood protection area.)

6. Provide geotechnical report in areas of known high water table. (201-4) yes no

7. Provide geotechnical report if adverse site conditions are found or suspected. (203) yes no

8. Provide site-drainage plan complying with CABO R301.3 or local requirements. (301) yes no

9. Provide fill specifications if site is to be prepared with earth fill. (303-2) yes no

10. If a geotechnical report is required, what is the net allowable soil bearing pressure? (202) ____–____ psf.

11. If no adverse soil conditions are known or suspected, and if the home is individually sited, assume a soil bearing pressure of 1,000 psf. and use this value when a determination of soil bearing pressure is called for. 1,000 psf.

APPENDIX E
MANUFACTURER'S WORKSHEET

Manufacturer's Company Name: NEW HOMES

Address: 39 PEACHTREE LANE
ATLANTA, GA

Telephone: 219/333-1792

Determination of Building Structure and Size

The manufacturer shall provide the following information:

1. Type of unit — Single-Section / Multi-Section
   Single-Section
2. Method, location and types of support:
   Refer to Figures 6-7 and 6-8 and Section 601-4
   Is the home a **C, E, or I**? C
3. Length of unit L — 56 ft.
4. Actual width of unit Wt — 13'-8 ft.
5. Height of exterior wall ** — 7'-6 ft.
6. Height of roof peak ** — 2'-4 ft.
7. Roof slope ** — 4::12
8. Self weight of total unit (W) including mechanical equipment ** — 16,500 lbs.
9. Distance between chassis members — 6'-10 ft.
10. One foundation design concept (See Appendix A) **(C1-C4; E1-E8; or I)** — C1

(Appendix G - Example 2)

11. Recommended pier spacing **

a. Exterior 7 ft.

b. Interior - ft.

c. Continuous Marriage Wall - ft.

Length of largest isolated marriage wall opening or average of largest two adjacent openings - ft.

d. Tie-down Strap (C1 concept only) 7 (Number) 8'-8" ft. (Spacing)

12. One installation method recommendations (include documentation showing connection details pertinent to geographic area for seismic or wind). ** yes no

13. Interior shear wall locations (include documentation showing locations). ** yes no

14. Design wind speed used in designing connection details for horizontal anchorage (Ah) and vertical anchorage (Av) in the transverse direction. ** 120 mph.

15. Seismic acceleration values used in designing connection details for horizontal anchorage (Ah) in the transverse and longitudinal directions. **

Av 0.05

Aa 0.05

16. Shear wall connection details with rated capacity for wind and seismic are provided. ** † yes no

a. Connection locations at foundation end and interior walls shown? ** yes no

b. Rated connection capacity for uplift and overturning ** 3150 lbs./ft. (or lbs./tie-down)

c. Rated connection capacity for sliding in transverse direction ** 4800 lbs./ft. (or lbs./diag. strap)

d. Rated connection capacity for sliding in longitudinal direction ** 4800 lbs./ft.

e. Vertical X-bracing tension strap capacity ** 5600 lbs./diag. strap

f. Engineering calculation by licensed structural engineer? ** yes no

** **Optional values:** It is optional for the manufacturer to provide these values. If the manufacturer does not provide the values, it is the responsibility of the owner to supply values, based on engineering analysis by a licensed structural engineer.

† Item 16 is provided in California.

APPENDIX F
DESIGN WORKSHEET

Owner's Name: JOHN SMITH

Address: 35 BRANDYWINE, TAMPA, FL

Builder's Name: GRAPPO INDUSTRIES

Site Location: TAMPA, FL.

PART 1: SITE CONDITIONS
(Accompanies Chapter 2)

1. Has the Manufacturer's Worksheet been provided? yes no

Existing Grade Elevation (201-1)

2. Does the site require a survey? yes no
   (Answer yes if: 1) elev. to be altered by grade or fill; 2) site near flood zone; 3) subdivision. Answer no if individually-sited with no alteration of building site.)

3. If yes to above, what is the surveyed existing grade elevation? 28 ft.

Flood Protection Elevation (201-2)

4. Is the building site in a flood zone? yes no
   (If yes to 4, then answer 5, 6, 7 & 8. If no, skip to 9.)

5. What is the Base Flood Elevation or the Flood Protection Elevation (use highest value)? – ft.

6. Is the site to be graded, filled, or bermed? yes no
   (If no, skip to 9.)

7. If yes to 6, have all permits been provided? yes no

8. If no to 6, then are the buildings to be built on elevated foundations? yes no
   (If yes, this handbook cannot be used. Refer to FEMA Manual.)

 (Appendix G - Example 2)

Frost Penetration Depth (201-3)

9. What is the maximum frost penetration depth? (see Appendix H, page H-4) — 0 in.

10a. Does foundation plan show base of footing extending below frost penetration depth? (If yes proceed; if no, applicant should revise plans.) — yes no

10b. Does foundation plan show base of footing extending below top-soil layer (min. 12") to undisturbed soil? — yes no

Ground Water Table Elevation (201-4)

11. For subdivisions, does a Geotechnical Engineer recommend drainage of subsurface water? (If no, skip to 13.) — yes no

12. Has groundwater drainage plan been provided? — yes no

Soil Conditions (202, 203)

13. If any of the following adverse site conditions are discovered, specific recommendations by a Geotechnical Engineer will be required (applies to subdivisions and individually-sited homes.)

Organic soil (8" topsoil layer)	yes no
Expansive (shrink-swell) soil	yes no
Sloping site	yes no
Subsidence	yes no

(Applicant may be referred to Geotechnical Engineer if any of the above are yes. If no, to all of above, move to next step.)

14. Is area in a known termite infestation area? — yes no

Region classification? (See Appendix H, Termite Infestation Map, page H-10) (If no, skip to 16.) — VERY HEAVY

15. Has applicant complied with CABO R-308 or local ordinance for construction procedures and treatment? (If yes, continue; if no, refer applicant to CABO requirements.) — yes no

PART 2: SITE PREPARATION
(Accompanies Chapter 3)

16. Acceptable surface drainage plan provided? (301) (If no, one must be provided for subdivision) — yes no

17. Grading plan provided? (302) — yes no

18. Fill specifications conforming to those cited in HUD Land Planning Data Sheet (79g)? (303) (If fill is used, below the home's foundation, a report by Geotech. Eng. should be submitted to provide fill specifications.) — yes no

19. Finish grade elevation? (304) (Check answers to Part 1: #4 & #5. The finish grade elevation must be higher than #5 if in flood zone.) — 28' *

PART 3: DESIGN LOADS
(Accompanies Chapter 4)

Information from Manufacturer's Worksheet

20. Has all the information been provided on the Manufacturer's Worksheet? (Appendix E) — yes no

21. What is the building self weight (W)? (Mfg. Wksht. #8) — 16,500 lbs.

22. What is the building length (L)? (Mfg. Wksht. #3) — 56 ft.

23. What is the distributed weight per foot of unit length? (w=W/L) (402-1.B, C) — 295 lbs./ft.

24. What is the building type? (Mfg. WkSht. #2) — Single-Section / Multi-Section

C, E, or I

Foundation design concept? (C1, C2, C3, C4, E1, E3, E4, E5, E6, E7, E8, I) — C1 *

Dead Load (402-1)

25. What is the light dead load value from Table 4-1? (402-1.A.1) — 290 (lbs./ft.) *

26. What is the heavy dead load value from Table 4-1? (402-1.A.2) — 425 (lbs./ft.) *

27. Does the answer from Question #23 fall within the values in #25 and #26? (402-1.D) (If the answer is yes, continue. If no, the foundation is not within the limits of this document and must be redesigned by a structural engineer.) — yes no

Snow Load (402-2) / Minimum Roof Live Load (402-2.C)

28a. What is average annual ground snowfall (Pg)? (See Ground Snow Load map, pages H-11, H-12 and H-13.) — 0 (lbs./sq.ft.) *

28b. What is 0.7 multiplied by Pg? — 0 psf.

29a. What is the roof slope? (Mfg. Wksht. #7) — 4:12

29b. What is the minimum roof live load for the roof slope? (D-200.2.B) — 15 psf.

30. Record the larger magnitude of item 28b or item 29b. Use this magnitude for roof load where required. — 15 psf.

Wind Load (402-3)

31a. What is the basic wind speed (V)? (See Wind Speed map, page H-14.) — 100 mph.

31b. If V is less than 80 mph, record *MPS* min. 80 mph for wind design. (402-3.A) — 100 mph.

32. Is the site inland or coastal? (402-3.B) (If inland, skip to question #38.) — Inland / Coastal

33. If a coastal area, has the manufacturer provided connection details? (402-3.D) (Mfg. Wksht. #12) — yes no

34. If yes to #33, what design wind speed has the manufacturer used in designing connection details? (Mfg. Wksht. #14) — 120 mph. *

35. Are the connection locations shown? (Mfg. Wksht. #16a) — yes no

36. Are connection details provided for foundation shear walls? (For an answer of yes, all questions under Mfg. Wksht #16 must be answered satisfactorily.) — yes no

37. Is the value for Question 34 equal to or greater than the number given in Question 31? (If yes, proceed. If no, return design to manufacturer for clarification.) — yes no

Seismic Load

38a. What are the seismic acceleration values? (See Seismic maps, pages H-15 and H-16) — Aa 0.05 * Av 0.05 *

38b. Is Av < 0.15? (if no, proceed. If yes, seismic need not be considered, skip questions 39 to 41.) — yes no

39. Seismic performance category. (See H-300 for Special Requirements of Foundation Design.) — N.A.

40. What is the applicant's proposed design concept? (Design Wksht. #24) — C1 *

41. Do the Foundation Design Concept Tables approve the foundation system for use in seismic areas of Question #38 above? (See Appendix A) (If yes, proceed. If no, return to applicant for foundation design choice more suited to high seismic areas.) — yes no

PART 4-FINAL DESIGN PROCEDURE

(Accompanies Chapter 6)

42. What is the actual building width? (Mfg. Wksht. #4) — 13'-8 ft.

43. The nominal building width to be used in the Foundation Design Tables, (Aftg, Av & Ah) is Wt: (600-2.A and Figure 6-1) — 14'-0 ft.

44. Where are the foundation supports located? Check drawings submitted by the owner and Foundation Design Concepts in Appendix A. Circle the support locations shown on the Manufacturer's foundation concept plan.

Chassis Beams
Exterior Walls
Marriage Wall

45. Do these locations match the Foundation Concept shown in Appendix A? Do the locations match Question #24 on the Design Worksheet? (If yes, proceed. If no, return to Owner for clarification.) — yes no

46. Is Vertical Anchorage present? (601-2.B, 601-3.B & 601-4.B (Figures 6-7 & 6-8); Mfg. Wksht. #12 & #16) — yes no

APPENDIX A

47. What is the basic system type? (From Part 3: #24; Mfg. Wksht. #2) — C1 *

48. What is the spacing between piers? (Mfg. Wksht. #11) (602-2)

Exterior: 4' 5' 6' 7' 8'

Interior: 4' 5' 6' 7' 8' N.A.

Continuous Marriage Wall: 4' 5' 6' 7' 8' N.A.

Largest or Average Marriage Wall Opening: N.A. ft.

Tie Down (C1) 8'-8 ft.

APPENDIX B

Required Footing Size

49. The required Exterior Wall Footing, for the foundation type, is found in the Required Effective Footing Area table in App. B, Part 1. (Use maximum value from item #30.) — C1 *

The Required Exterior Square Footing size is:

Type C 5.3 sq.ft.

Type E or I — ft. (width)

50. The Required Interior Footing area is: (Also exterior piers for foundation type E) — sq.ft.

51a. The Required Continuous Marriage Wall Footing area is: — sq.ft.

51b. The Required Footing area under posts at the ends of marriage wall opening(s) is: — sq.ft.

Vertical Anchorage Requirements in the Transverse Direction (602-4)

52a. Using the Foundation Design Load Tables (Appendix B, Part 2), determine the Required Vertical Anchorage.
350 LB/FT × 8.667' = 3033 LB

Exterior Av 3033 *
(lbs./pier spacing; lbs./ft for E type; lbs./tie-down spacing)

52b. Number of vertical tie-down locations for multi-section units: 2 or 4 or 6

52c. For units with additional vertical anchorage at the interior piers, determine the Required Vertical Anchorage.

Interior Av N.A. *
(lbs./int pier spacing)

53. What is the manufacturer-supplied value? (#16b, Mfg. WkSht.)

Exterior 3150 *

Interior 3150 *

54. Is this value (#53) greater than the value given in #52a? (If yes, continue. If no, return to owner for clarification.) yes no

Horizontal Anchorage Requirements In The Transverse Direction (602-5)

	trial 1	trial 2	trial 3	
55a. What number of transverse foundation walls was selected? (602-5.E) (If vertical X-bracing planes are used, complete items #55a, #56 and #57 for 2 transverse walls, and then skip to item #59.)	2	4	6	
55b. Are diagonal ties used to complete the top of the transverse short wall for horizontal anchorage? (602-5.G.1)	yes no	yes no	yes no	
Estimate height (h) for appropriate illustration in Figure 6-10.	N.A			ft.

		trial 1	trial 2	trial 3	
56. Using the tables, find the Required Horizontal Anchorage (Ah). (Appendix B; Part 3)	End Wall Ah	1240			lbs./ft.
	Int Wall Ah	N.A.			lbs./ft.
57a. What is the manufacturer's-supplied rated capacity for sliding? (#16c, Mfg. WkSht.)		4800			lbs./ft.
57b. If answer to item #55b is yes, record manufacturer or product supplier rated strap tension capacity					lbs./strap
58a. Is value #57a greater than item #56? If yes, continue. If no, return to section 602-4.C and to question #55a and select a larger number of transverse foundation walls. If the maximum number selected (6) does not work, return to owner (who may wish to contact the manufacturer for clarification).		yes no	yes no	yes no	
58b. If answer to #55b is yes, required tension in diagonal (T_t). (Complete procedure in Section 602.5.G.1.)					lbs.
58c. Is value #57b greater than #58b? If yes, continue to item #62. If no, return to owner for product with greater capacity.		yes no	yes no	yes no	

59. If using vertical X-bracing planes in lieu of transverse short walls (and the formulas in section 602-5.G.2), determine anchorage values and sizes for diagonal members. (If shear walls are selected in item #55, skip to item #62.)

	trial 1	trial 2	trial 3	
a. Vertical X-bracing spacing proposed.	14	7'		ft. *
b. Number of vertical X-bracing locations proposed. (Item #13, Mfg. WkSht. for trial 1.)	5	9		*

		trial 1	trial 2	trial 3	
c.	Required horizontal anchorage (C) value, based on formula. (602-5.G.2.c)	8475	4235		lbs./ x-brace set
d.	Estimated height (h) in Figure 6-10.		4		ft.
e.	Tension (T_t) required. (602-5.G.2.d)		4907		lbs./diag.
60.	What is the manufacturer-supplied rated strap tension capacity? (#16, Mfg. WkSht.) (or capacity defined by literature supplied by product supplier)		5600		lbs. *
61a.	Is value #57 greater than value #59c? If yes, continue. If no, return to Section 602-5.G and to question #59 and select a greater number of X-brace locations as a next trial. Repeat until answer is yes, then continue.	yes no	yes no	yes no	
61b.	Is value #60 greater than value #59e? If yes, continue. If no, return to section 602-5.G and to question #59 and select a greater number of X-bracing locations. If the maximum number selected does not work, return to owner (who may wish to contact the manufacturer for clarification or product supplier for clarification).	yes no	yes no	yes no	

Horizontal Anchorage Requirements In The Longitudinal Direction (602-6)

62a. Using the tables, find the required horizontal anchorage (Ah) in the longitudinal direction. (Appendix B, Part 4) (602.6.E)

Exterior Wall Ah 47 lbs./ft.

62b. If using vertical X-bracing planes (and the formulas in section 602-6.F) determine anchorage value for X-bracing planes. (If using exterior long walls, skip to item #63.)

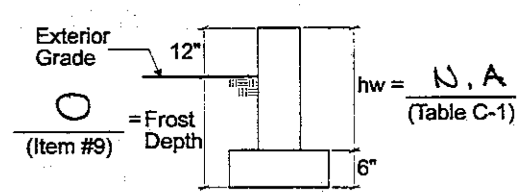
	trial 1	trial 2	trial 3
1. Number of chassis beam lines used for vertical X-bracing planes.	2 or 4	2 or 4	2 or 4

	trial 1	trial 2	trial 3	
Number of X-bracing planes proposed under each chassis beam along the length of the unit.	2			
2. Horizontal anchorage (B) required force, based on formula.	1316			lbs.
3. Assumed height (h-b) based on Figure 6-11.	3'			ft.
4. Tension (T_L) based on formula. (602-6.F.(3)).	1432			lbs.
63. What is the manufacturer-supplied value for horizontal anchorage? (#16d, Mfg. WkSht.)	4800			lbs./ft.
64a. For shear walls: is value #63 greater than #62a? If yes, skip to item #67. If no, contact owner for clarification.	yes no	yes no	yes no	N.A.
64b. For X-bracing: is value #63 greater than value #62b.2? If yes, return to item #62b.3. If no, increase number of vertical X-bracing planes and repeat items 62b.1 and 62b.2 until answer is yes. For multi-section units consider 4 lines of vertical X-bracing under all chassis beams.	yes no	yes no	yes no	
65. What is the manufacturer-supplied rated strap tension? (#16e, Mfg. WkSht. or product supplier)	5600			lbs.
66. Is value #65 greater than #62b.4? If yes, continue. If no, contact owner to obtain straps with greater capacity, or return to item #62b.1 and increase the number of vertical X-bracing planes until answer is yes.	yes no	yes no	yes no	

APPENDIX C

Withdrawal Resistance Verification (603-2.B)

67. Using Appendix C, Table C-1 or C-2, verify that the foundation system will resist withdrawal. Answer question #67a for type E. Answer question #67b for types C, I, or type E with interior pier anchorage.



a. ***Withdrawal Resistance for long foundation wall.*** (Type E)
Circle the type of material that is to be used.

Reinforced Concrete
Masonry-Fully Grouted
Masonry-Grouted @ 48" o.c.
All-Weather Wood / Footing

1) Using Table C-1, which capacity is greater than required Av? (603-2.B.(1)) (#52a) __________ lbs./ft.

2) Using Table C-1, what is the height of the wall + footing for required withdrawal resistance? (hw + 6") __________ in.

3) What is the height of the wall + footing for frost protection? (frost depth (#9) + 12") __________ in.

4) What is the greatest height #67a.2 or #67a.3? __________ in.

Circle the height which controls.

Withdrawal
Frost Depth

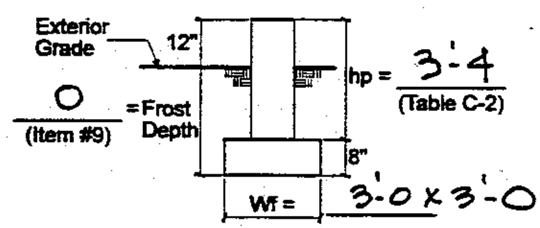
5) Record the bottom of footing depth from grade. (Item #67a.4 - 12") __________ in.

6) Using Table C-1, what is the required width of the wall footing for withdrawal? __________ in.

7) Is item #67a.6 greater than or equal to item #49? If yes, continue. If no, change footing width to item #49. yes no

8) Record design exterior wall footing width. __________ in.

b. ***Withdrawal Resistance for Piers.*** (Types C, C1 (concrete dead-man), I or type E with interior pier anchorage - multi-section units.)



Circle pier type:

Reinforced Concrete
Reinforced Masonry - fully grouted
Reinforced Concrete Dead-man

	Exterior	Interior (when used)	
1) Using Table C-2, which capacity is greater than required Av? (#52a and #52c) (603-2.B.(2))	3540		lbs./pier *
2) Using Table C-2, what is the height of the pier + footing for required withdrawal resistance? (hp + 8")	48 IN		in. *
3) What is the required height of pier + footing for frost protection? (frost depth (#9) + 12") 0	12		in.
4) What is the greatest height #67b.2 or #67b.3?	48		in.
Circle the height which controls.	Withdrawal Frost Depth	Withdrawal Frost Depth	
5) Record the bottom of footing depth from grade. (Item #67b.4 - 12") 48	36		in.
6) Using Table C-2, what is the required width of the square footing if withdrawal resistance controls or if frost depth controls?	36"		in. *

c. ***Frost depth for marriage walls.*** What is the required depth of footing below grade for frost protection? (frost depth (#9)) (no withdrawal resistance) N.A. in.

Vertical Anchorage and Reinforcement for Longitudinal Foundation Walls and Piers (603-2.D)

68. Using Appendix C, Table C-3, C-4A or C-4B, verify that the foundation anchors will resist uplift. Answer question #68a for type E. Answer question #68b for types C, I, or type E with interior pier anchorage.

   a. ***Vertical Anchor Capacity for longitudinal foundation wall*** (type E). (603-2.D.2)

      1) Using Table C-4A (concrete & masonry), which capacity is greater than the required Av? (#52a, Design Wksht.) If treated wood wall, skip to item #68a.3. N.A. lbs./lineal ft. of wall

Circle correct washer choice for the capacity selected — Standard Washer / Oversized Washer

2) Using Table C-4A (masonry and concrete):

a) Required anchor bolt diameter ________ in.

b) Required anchor bolt spacing ________ in.

c) Using Table C-3A:

(1) Rebar size ________ *

(2) Lap splice ________ in.

(3) Rebar hook length ________ in.

3) Using Table C-4B (wood), which capacity is greater than the required Av? (#52a, Design Wksht.) ________ lbs./lineal ft. of wall
If using concrete or masonry wall, skip to item #68b.

4) Using Table C-4B (wood):

a) Required nailing ________ *

b) Minimum plywood thickness ________ in.

c) Required anchor bolt diameter ________ in.

d) Required anchor bolt spacing ________ in.

b. *Vertical Anchor Capacity for Piers*
(Types C, I, or type E with interior pier anchorage)
(603-2.D.1)

	Exterior	Interior (when used for anchorage in multi-section units)	
1) Using Table C-3, which capacity in the table is greater than the required Av? 3033 (From #52a, Design Wksht.)	4240	________	lbs./pier

	Exterior	Interior	
2) Using Table C-3:			
a) Number of anchor bolts	1 or 2	1 or 2	
b) Anchor diameter	1/2" or 5/8"	1/2" or 5/8"	
3) Using Table C-3A:			
a) Rebar size	#4 or #5	#4 or #5	
b) Lap splice	16		in.
c) Rebar hook length	6		in.

Horizontal Anchorage and Reinforcement for Transverse Foundation Walls (603-3)

69. Using Appendix C, Table C-5A or C-5B, verify that the foundation anchorage will resist sliding at the transverse end foundation walls. Use for types C, E, or I.

	End Wall	Interior Wall	
a. ***For continuous foundations.***			
Using Table C-5A (concrete & masonry) or C-5B (wood), which capacity is greater than the required (Ah) (603-3) (item #56)?			lbs./ft.
1) Using Table C-5A, find:			
a) Required anchor bolt diameter			in.
b) Required anchor bolt spacing			in.
c) Using Table C-3A:			
(1) Rebar size			*
(2) Lap splice			in.
(3) Rebar hook length			in.
2) Using Table C-5B, find:			
a) Required nailing			*

	End Wall	Interior Wall	
b) Minimum plywood thickness			in.
c) Required anchor bolt diameter			in.
d) Required anchor bolt spacing			in.

b. ***For transverse short foundation walls completed with diagonal braces.*** (603-5)

Using Appendix C, Table C-5A, verify the diagonal anchorage capacity to the short foundation wall.

	End	Interior	
1) Record the required horizontal force (Ah × Wt) from 602-5.G.1.a and item #56.			lbs.
2) Table C-5A capacity for one 1/2" diameter bolt at 12" o.c.	1800	1800	lbs.
3) Number of bolts (Ah × Wt ÷ 1800; one minimum) at concrete or masonry top of short wall.			*
4) Size of anchor bolts			in.
5) Using Table C-3A:			
a) Rebar size			*
b) Lap splice			in.
c) Rebar hook length			in.

c. ***For vertical X-bracing planes in the transverse direction.*** (603-6)

Using Appendix C, Table C-5A, verify the diagonal anchorage to the pier footings and the tension capacity of the diagonals.

1) Record the required horizontal force (C) from item #59c.	4235	lbs.
2) Table C-5A capacity for one 1/2" diameter bolt at 12" o.c.	1800	lbs.

3) Number of bolts (C ÷ 1800; one minimum) at top of a footing. 3 *

4) Record the required tension force (T_t) from item #59e. 4907 lbs./diag.

5) Select tension strap capacity greater than or equal to T_t from owner's product supplier or manufacturer's supplied capacity (item #60). 5600 lbs./diag.

6) Record diagonal strap data A36 plate : 1/4" x 1"

Horizontal Anchorage for Longitudinal Foundation Walls (603-4)

70. Using Appendix C, Table C-5A or C-5B, verify that the foundation horizontal anchorage will resist sliding at the long foundation walls. Use for types C, E and I.

a. ***For continuous exterior foundation walls.***

Using Table C-5A (concrete and masonry) or Table C-5B (wood), which capacity is greater than the required exterior Ah? (602-6.E) (item #62a) ______ lbs./ft.

1) Using Table C-5A, find:

a) Required anchor bolt diameter ______ in.

b) Required anchor bolt spacing ______ in.

c) Using Table C-3A:

(1) Rebar size ______ *

(2) Lap splice ______ in.

(3) Rebar hook length ______ in.

2) Using Table C-5B, find:

a) Required nailing ______ *

b) Minimum plywood thickness ______ in.

c) Required anchor bolt diameter ______ in.

d) Required anchor bolt spacing ______ in.

b. ***For vertical X-bracing planes.***
(603-6.A.(2))

Using Appendix C, Table C-5A, verify the diagonal anchorage to the pier footings and the tension capacity of the diagonals.

1) Record the required horizontal force (B) from item #62b.2.	1316	lbs.
2) Table C-5A capacity for one 1/2" diameter bolt at 12" o.c.	1800	lbs.
3) Number of bolts (B ÷ 1800; one minimum)	1	*
4) Record the required tension force (T_L) from item #62b.4.	1432	lbs./diag.
5) Select tension strap capacity greater than or equal to T_L from owner's product supplier or manufacturer's supplied capacity (item #60).	5600	lbs./diag.
6) Record diagonal strap data	A36 PL	1/4" × 1"

SUMMARY SHEET
(Accompanies Chapter 7)

71. Compare values from preceding questions. Select the largest value.

a. **Bearing area and vertical anchorage**

1. *Pier footings: types C, E & I.*

	Piers				
			Marriage Wall		
	Exterior	Interior	Cont.	At Post	
Required Effective Footing Area from questions #49, #50, & #51.	5.3				sq.ft.
Required footing area to resist withdrawal due to uplift from Question #67. (for single-section or 2 tie-down system, only the exterior piers resist uplift, for 4 tie-down only the interior piers and exterior walls resist uplift)	N.A		sq.ft.		

	Piers			
			Marriage Wall	
	Exterior	Interior	Cont.	At Post
Pier Footing Sizes (largest of above)	5.3 (2'-4 x 2'-4)			sq.ft.

"Dead-man" footing size. 9.0 sq.ft. (3'-0 x 3'-0)

Reinforcing for pier footings:
Bring forward answers from previous questions. (#68b)
(Types C , I, or E with interior pier anchorage.)

	Exterior	Interior	
Number of anchor bolts	1		
Anchor bolt diameter	1/2 ϕ		in.
Rebar size	#4		
Lap splice	16		in.
Rebar hook length	6		in.

	Exterior	Interior	Marriage Wall	
Footing depth: grade to bottom of footing	0'-8"			in.

Pier footing and "dead-man" footing reinforcing bars: #4 at 10" o.c. E.W.

"Dead-man" footing depth: grade to bottom of footing 36 in.

2. *Long Foundation wall footing: type E or I:*

Required Effective Footing Width

Required Footing Width for soil bearing (#49) ________ ft.

Required Footing Width to resist uplift withdrawal (#67a.6) ________ ft.

Wall Footing Size (largest of above) ________ ft.

Footing Depth: Grade to bottom of footing (#67a.5) ________ in.

Footing reinforcing bars. 2 #4 bars

Reinforcing for longitudinal foundation walls: Record answers from item #68a and record sizes and spacings.

From 68a.2: masonry and concrete:

Required anchor bolt diameter ______ in.

Required washer size Standard Oversized

Required anchor bolt spacing ______ in

Rebar size ______

Lap splice ______ in.

Rebar hook length ______ in.

From 68a.4: wood: Record answers from item #68a.4 and record sizes and spacings.

Required nailing ______

Minimum plywood thickness. ______ in.

Required anchor bolt diameter ______

Required anchor bolt spacing ______ in

b. **Horizontal anchorage in the transverse direction - foundation walls**

1. *Continuous foundation walls* (#69a)

Number of transverse foundation walls (#55a) 2 4 6

Required Footing Width (minimum) 12 in.

From #69a.1: concrete / masonry:

	End Wall	Interior Wall	
Anchor bolt diameter	______	______	in.

	End Wall	Interior Wall	
Anchor bolt spacing	______	______	in.
Rebar size	______	______	
Lap splice	______	______	in.
Rebar hook length	______	______	in.
From #69a.2: wood:			
Required nailing	______	______	
Minimum plywood nailer	______	______	
Anchor bolt diameter	______	______	
Anchor bolt spacing	______	______	in.

2. *For transverse short foundation walls completed with diagonal braces* (#69b)

	End	Interior	
Number of pairs of diagonals (1 for single-section units, 2 for multi-section units) times number of short walls (end or interior) (#55a)	______	______	
Diagonal spacing (same as number of short walls)	______	______	
From #69b: concrete / masonry:			
Anchor bolt diameter	______	______	in.
Number of bolts	______	______	
Rebar size	______	______	
Lap splice	______	______	in.
Rebar hook length	______	______	in.

3. *For vertical X-bracing planes in lieu of short walls.* (#69c)

Number of X-brace locations (#59) 9

Spacing of vertical X-brace planes (#59) 7 ft.

Items from #69c.3 and #69c.5

Required anchor bolt diameter ½ ϕ in.

Number of bolts at top of footing to connect diagonal 3

Diagonal strap size A36 ℄ - ¼" x 1"

Connection to top flange of chassis beam (describe) mftr dt'l

c. **Horizontal anchorage in the longitudinal direction - exterior foundation walls**

1. *Continuous foundation walls*

Reinforcing for longitudinal foundation walls: record only if larger sizes or closer spacing than recorded for vertical anchorage (#71a.2).

From #70a.1: concrete / masonry:

Anchor bolt diameter ________ in.

Anchor bolt spacing ________ in.

Rebar size ________

Lap splice ________ in.

Rebar hook length ________ in.

From #70a.2: wood: record only if larger sizes or closer spacings than recorded for vertical anchorage (#71a.2)

Required nailing ________

Minimum plywood nailer ________

Anchor bolt diameter ________

Anchor bolt spacing ________ in.

2. *Vertical X-bracing planes under chassis beam lines* (#70b.)

Number of X-brace locations along one chassis beam line. 2

Spacing of X-brace locations along one chassis beam line. 49 ft.

Required anchor bolt diameter. 1/2 φ in.

Number of bolts at top of footing at connection to the diagonal. 1

Diagonal strap size. A36 ℄ : 1/4" x 1"

Connection to bottom flange of chassis beam (describe). mftr. Dt'l

72. Do foundation dimensions and details comply with Foundation Capacities Table, based on Foundation Design Table Values? yes no

73. If #72 yes, approve. If no, return to applicant. **APPROVE** **DISAPPROVE**

APPENDIX H
MAPS

H-100. GENERAL. The following collection of maps is intended to assist the user in the foundation selection and design process. The maps provide information for geographic locations within the 50 States of the United States covering a wide range of issues: flooding, frost penetration, expansive soils, landslides, subsidence, termites, snow, wind and earthquakes. The maps have been accumulated from various sources, most notably the U.S. Department of Commerce Weather Bureau, the U.S. Army Corps of Engineers Waterways Experiment Station, and the American Society of Civil Engineers.

H-200. SEISMIC PERFORMANCE CATEGORIES. Table H-1 is a condensed version of the ASCE 7-93 Seismic Performance Category Table as it applies to manufactured housing.

H-300. SPECIAL SEISMIC DESIGN CONSIDERATIONS FOR FOUNDATIONS.

H-300.1. General. Based on the Seismic Performance Category for the geographic location involved, special requirements must be satisfied that involve the foundation:

A. Seismic Performance Category A. There are no special requirements for the foundations of manufactured housing assigned to this Category.

B. Seismic Performance Category B. The site coefficient has been assumed as 2.0 for all Tables in Appendix B. The resulting capacities of the foundations, subjected to the prescribed seismic forces of the Tables in Appendix B shall meet the following requirements:

1. Structural Components. The design strength of foundation components subjected to seismic forces alone or in combination with other prescribed loads and their detailing requirements shall conform to the requirements of the applicable material codes (wood, concrete or masonry) referenced by the local authority having jurisdiction.

2. Soil Capacities. For the load combination including earthquake, the capacity of the foundation soil in bearing or the capacity of the soil interface between pile, pier or caisson and the soil must be sufficient to resist loads at acceptable strain considering both the short duration of loading and the dynamic properties of the soil.

C. Seismic Performance Category C. Foundations for buildings assigned to Category C shall conform to all of the Foundations for Categories A and B and to the following additional requirements of this section.

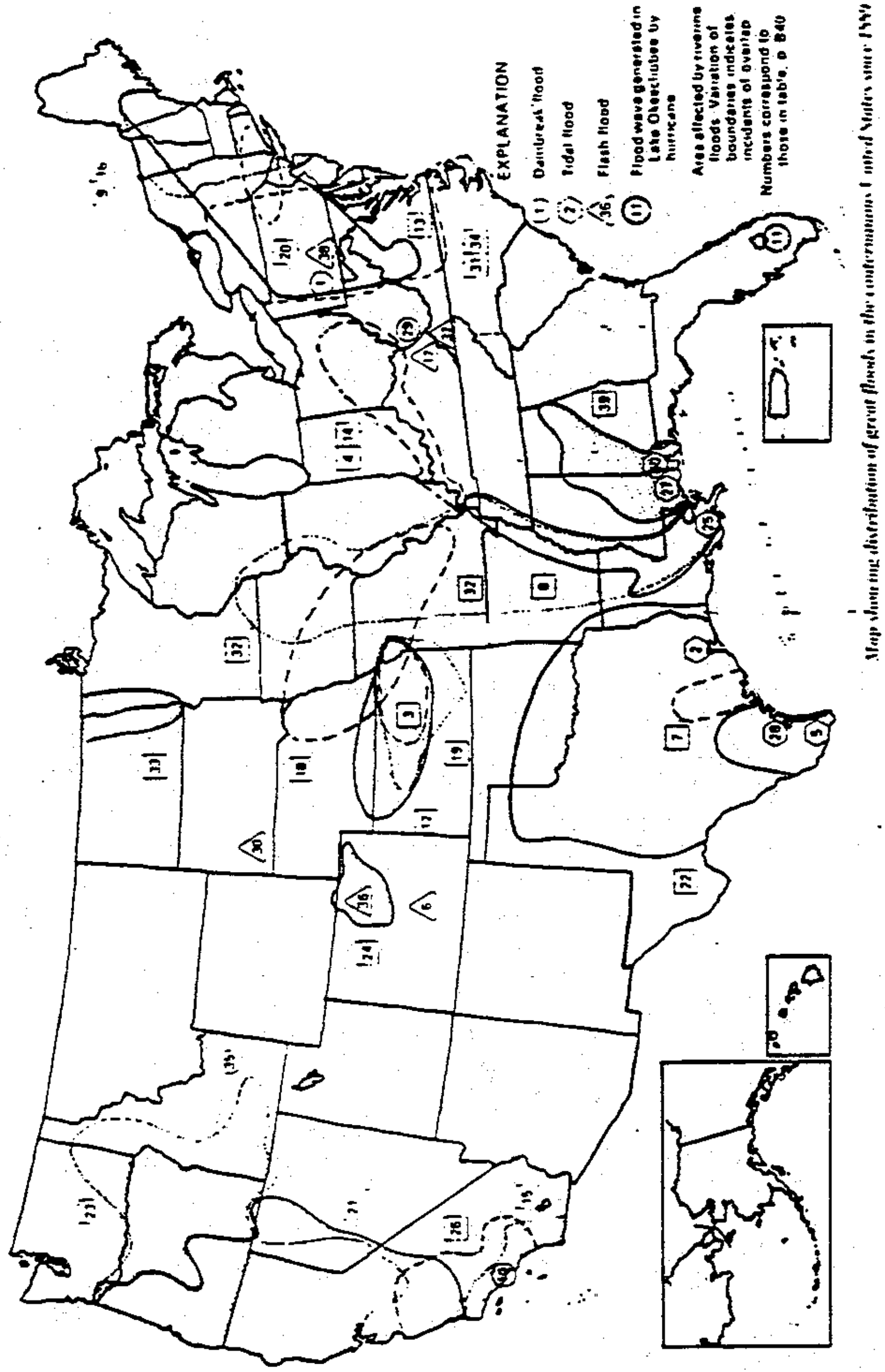
1. Investigation. The authority having jurisdiction may require the submission of a written report that shall include, in addition to the evaluations required in this section, the results of an investigation to deter-

mine the potential hazards due to slope instability, liquefaction and surface rupture due to faulting or lateral spreading, all as a result of earthquake motions.

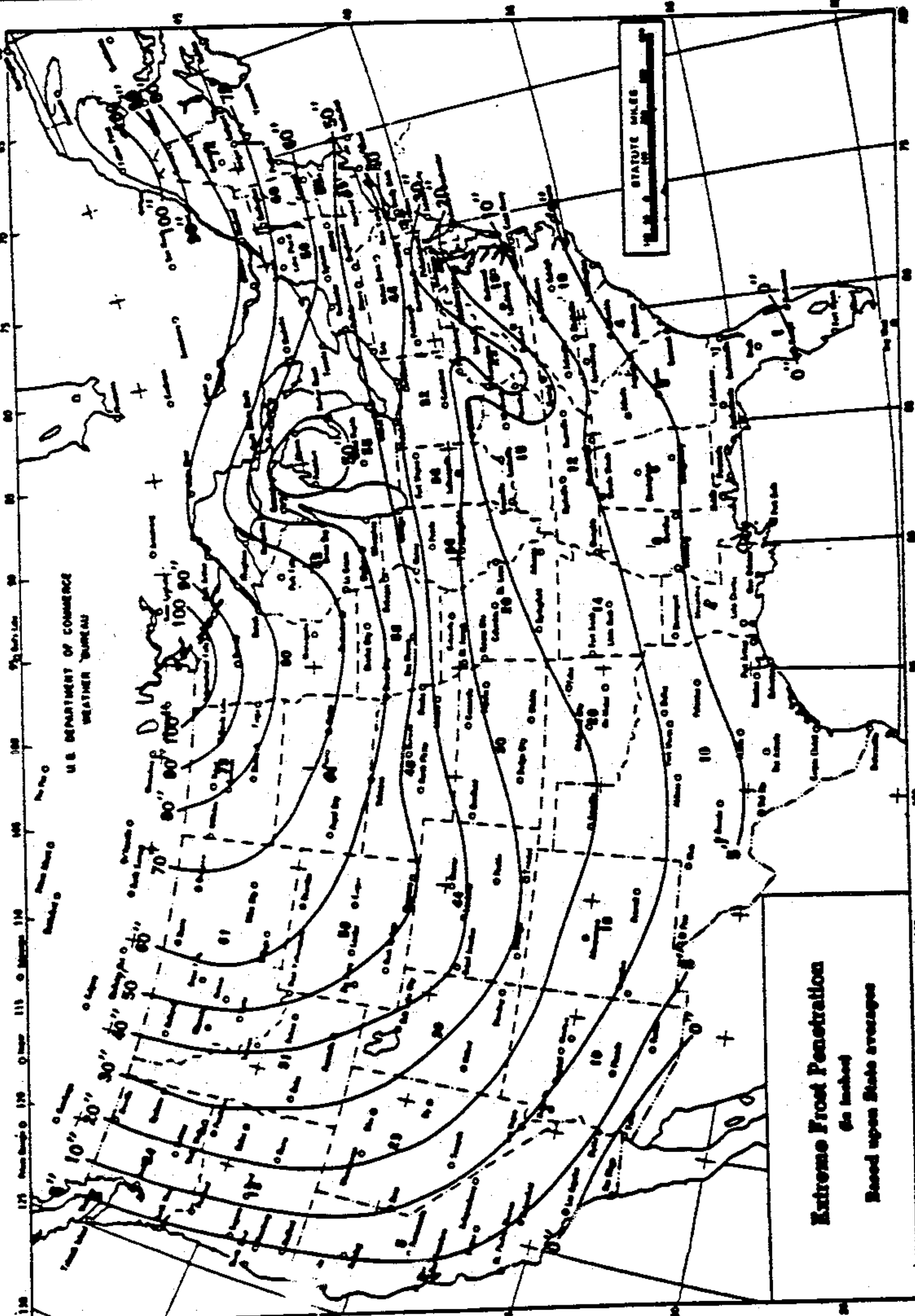
2. Foundation Ties. Individual drilled piers shall be interconnected by ties. All ties shall have a design strength in tension or compression, greater than a force equal to 25 percent of the effective peak velocity related acceleration (Av) time the larger column dead plus live load.

3. Special Pile Requirements. For uncased concrete drilled piers, there shall be a minimum of four longitudinal bars (with a minimum reinforcement ratio of 0.005) and No. 3 closed ties with maximum spacing of 3 inches.

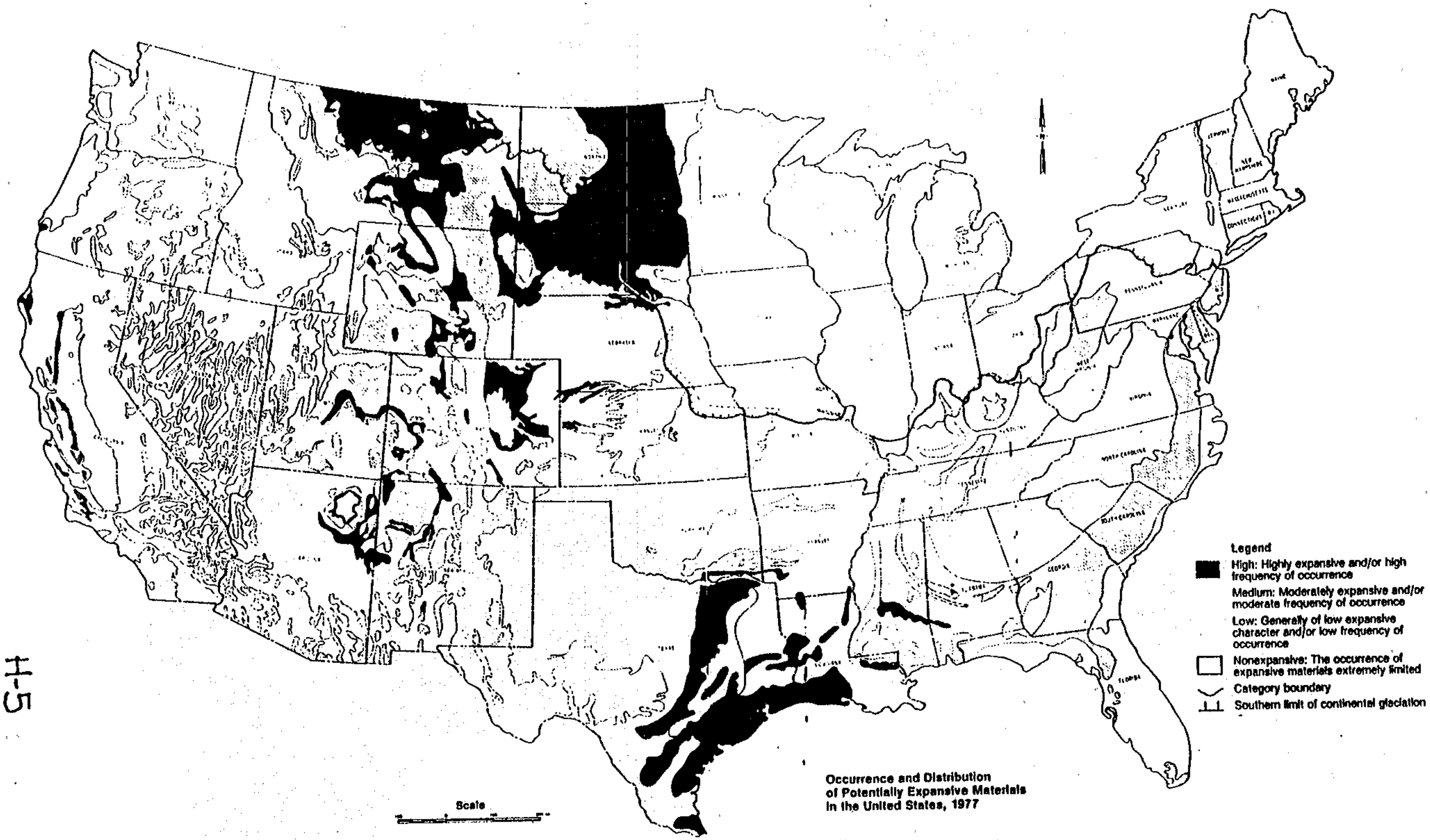
D. Seismic Performance Category D. Category D does not add any additional requirements for manufactured housing. The requirements of Category C plus A and B shall be followed.



FLOOD-PRONE SITES



FROST PENETRATION DEPTH



H-5

Fig. 10.
Occurrence and distribution of potentially expansive materials in the United States, 1977. (U.S. Army Corps Engineer Waterways Experiment Station)

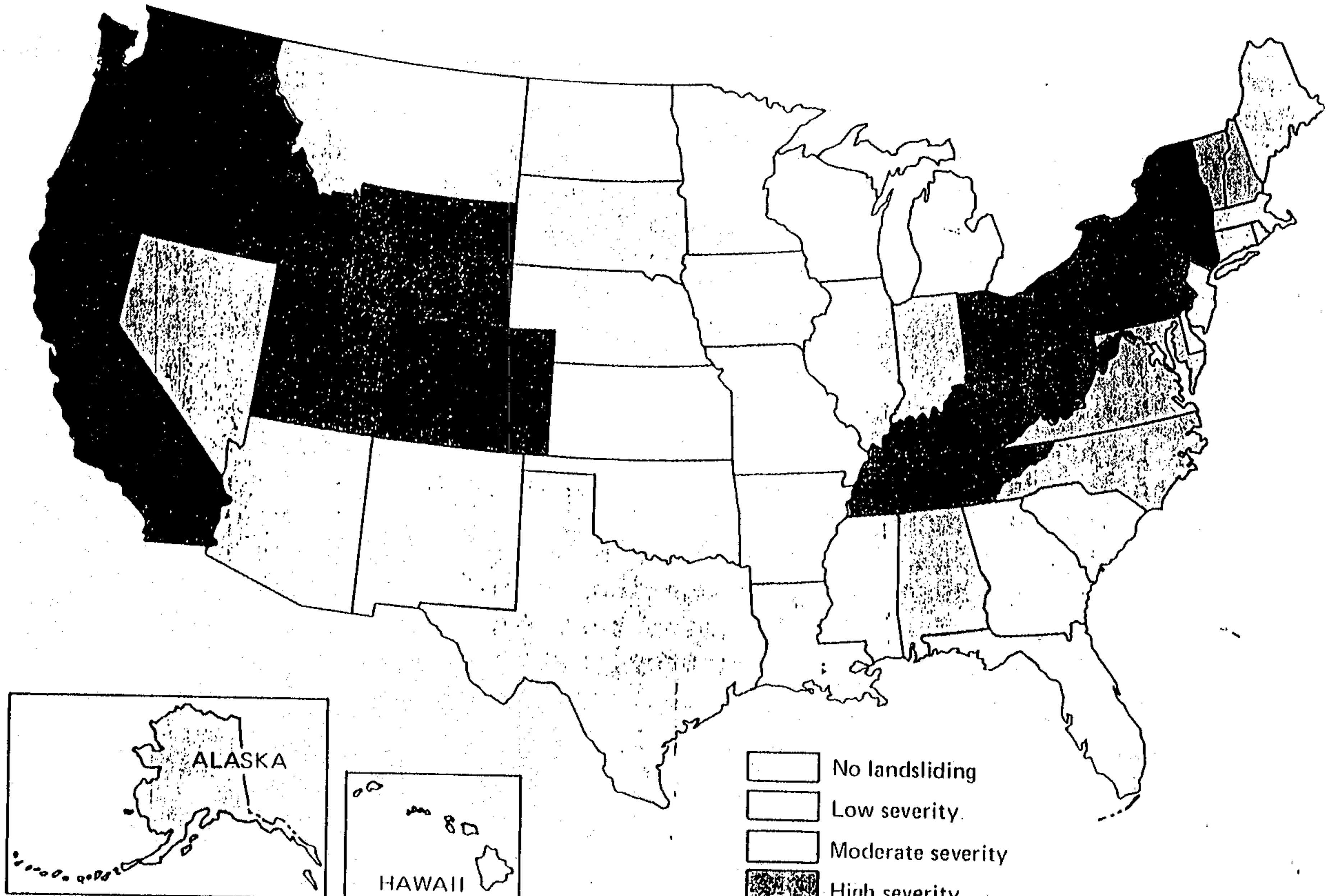
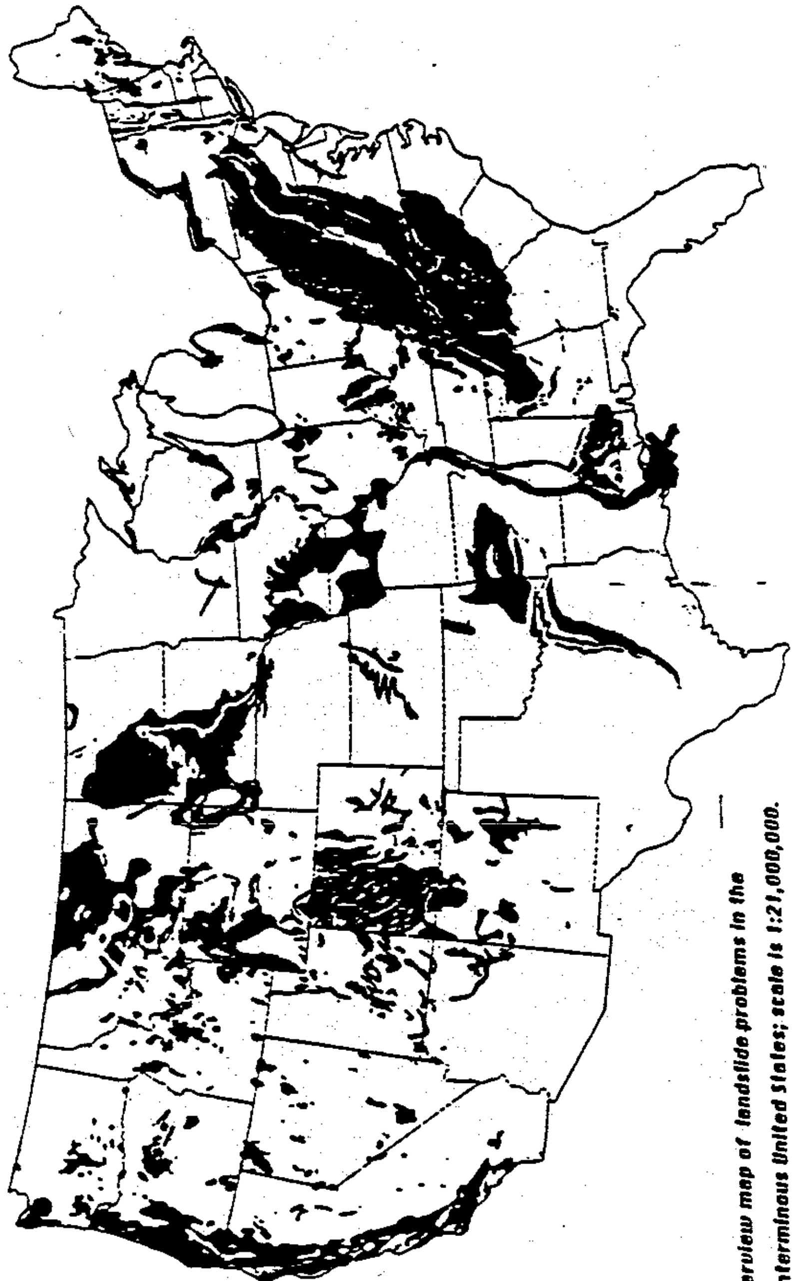
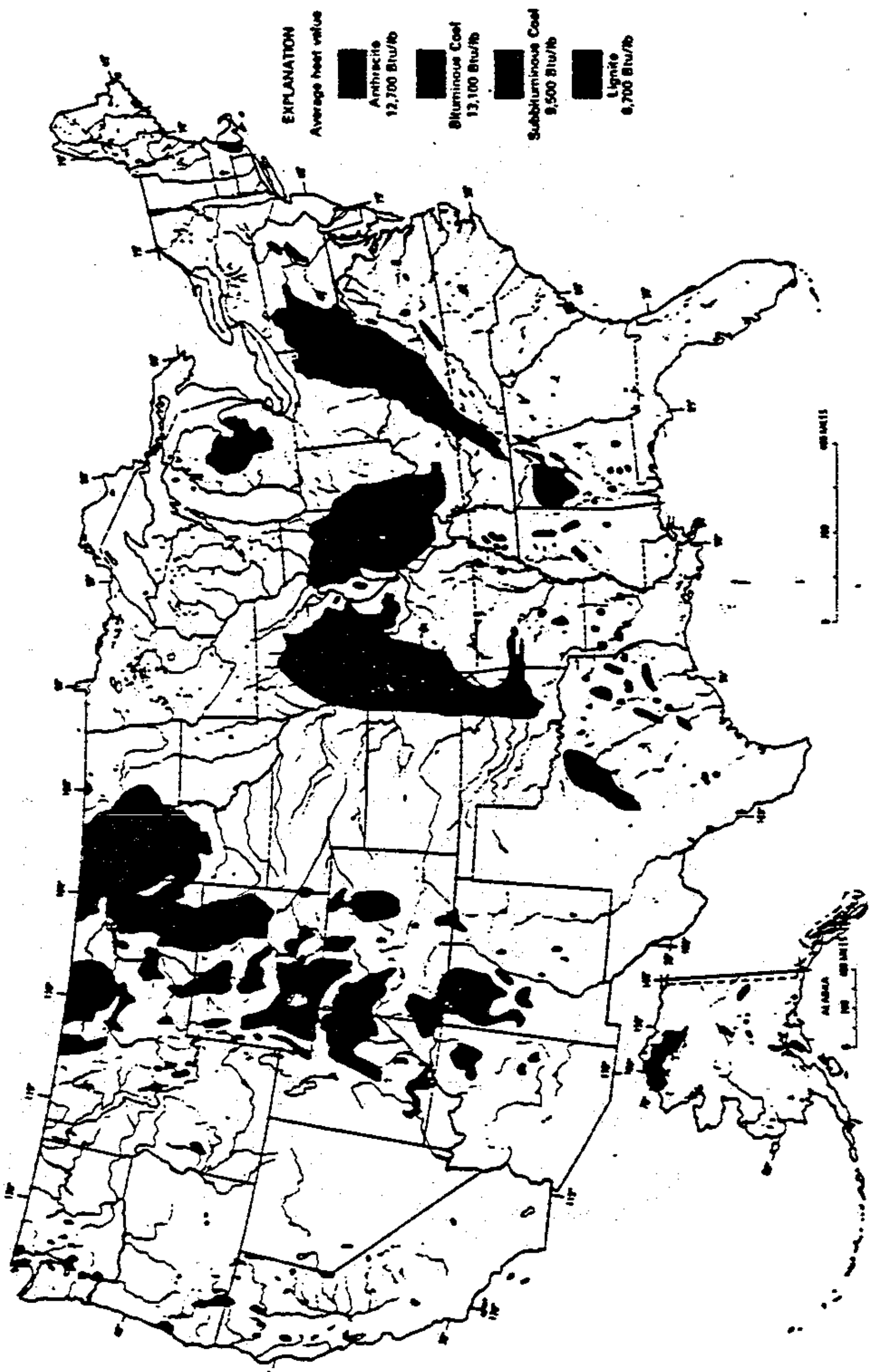


FIGURE 1 Qualitative indication of the severity of landsliding in the United States by state.



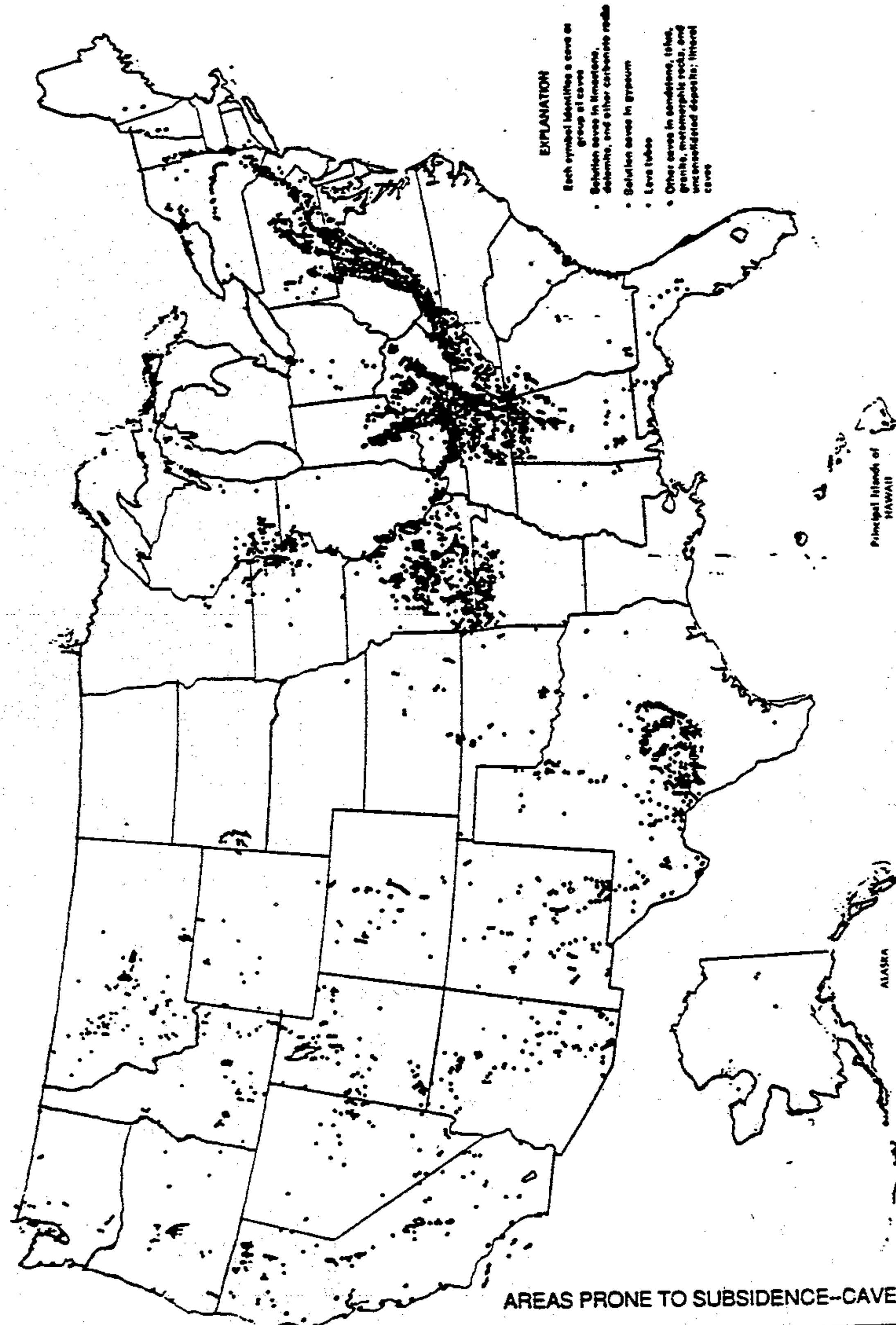
Overview map of landslide problems in the conterminous United States; scale is 1:21,000,000. The severity is highest in the lightly shaded areas, with severity lessening as the color darkens. (modified from Radbruch-Hall and others, 1976)

LANDSLIDE AREAS



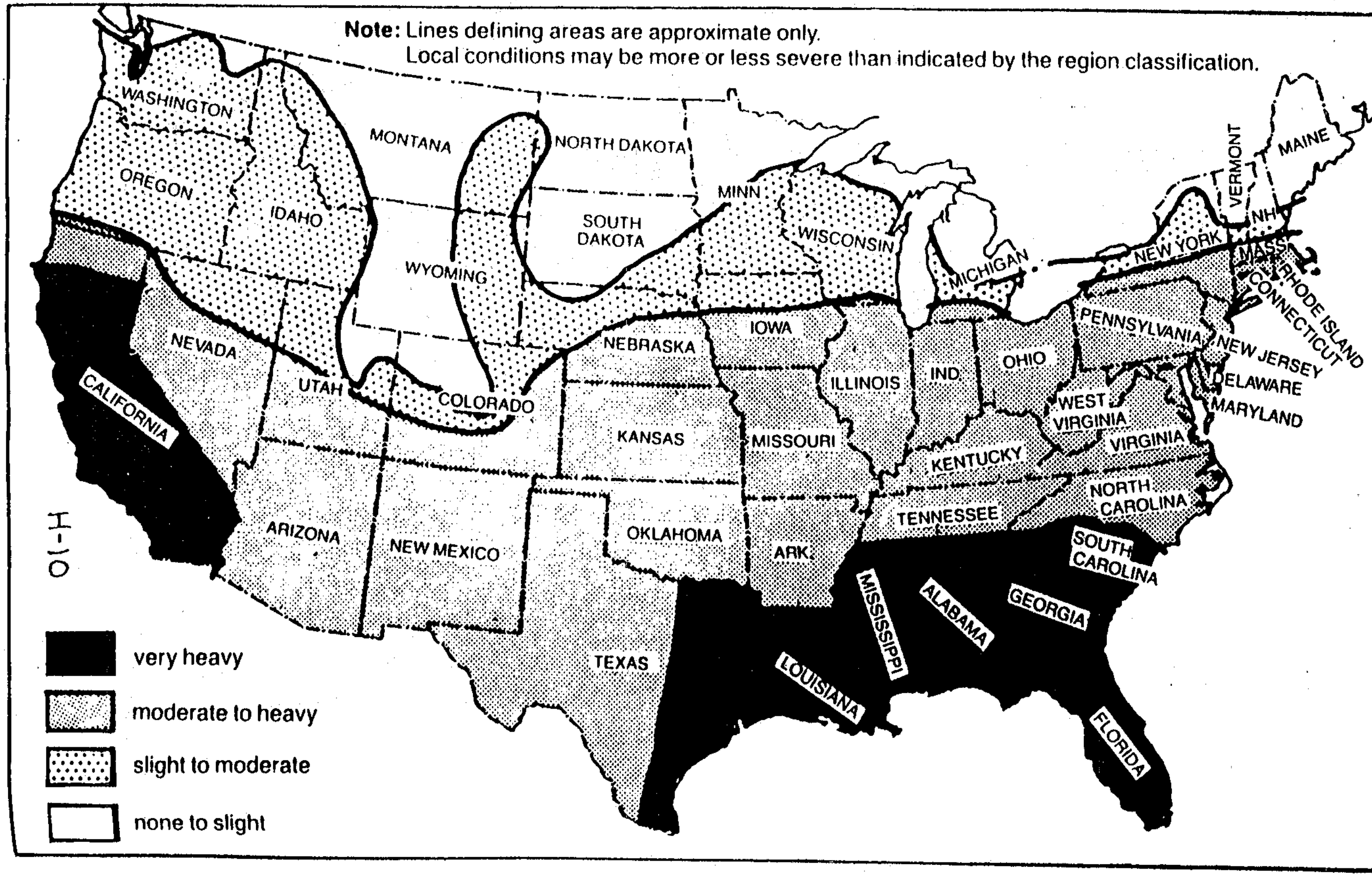
Location of coal fields in the United States (modified from Rickert and others, 1979).

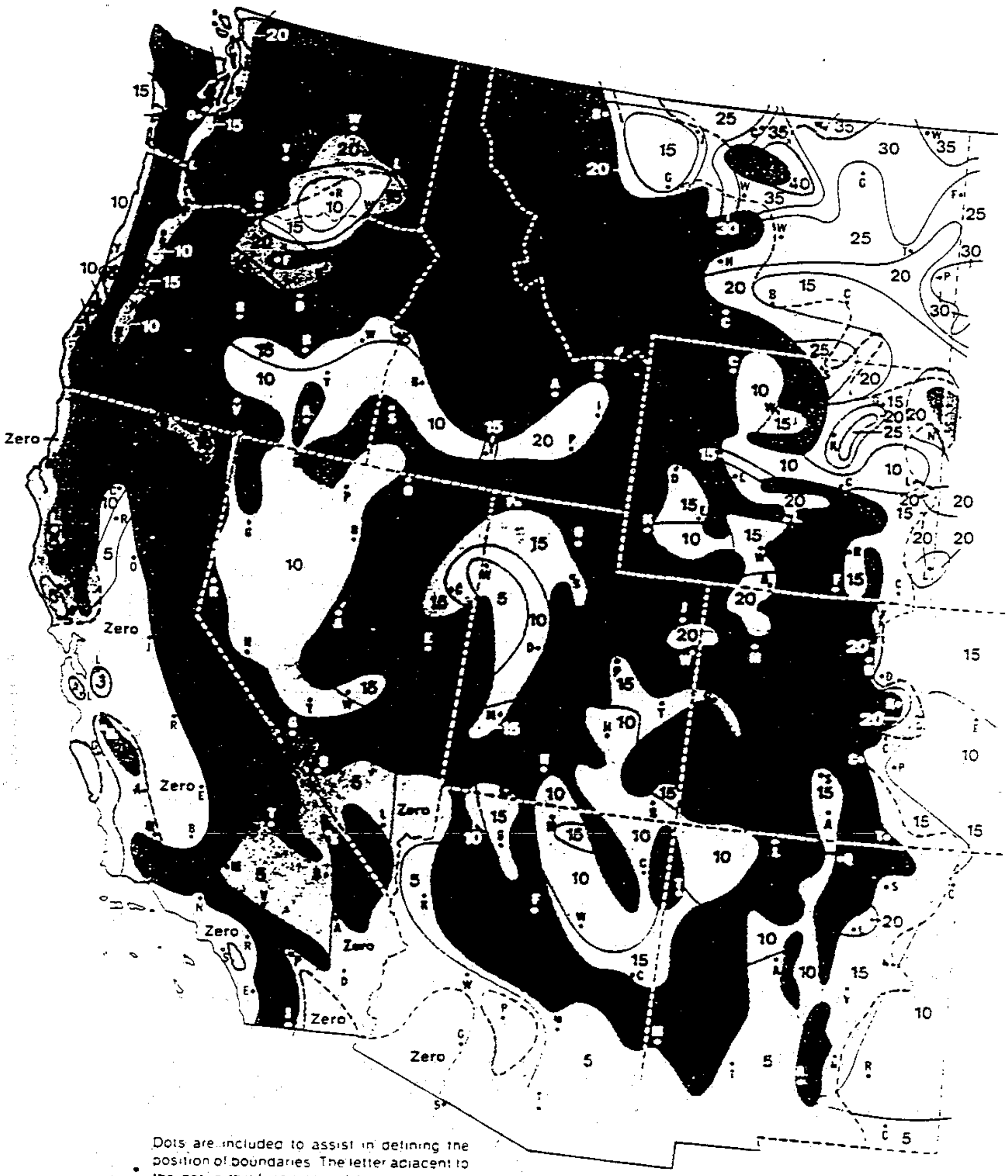
AREAS PRONE TO SUBSIDENCE–MINING



AREAS PRONE TO SUBSIDENCE--CAVES

TERMITE INFESTATION PROBABILITY MAP



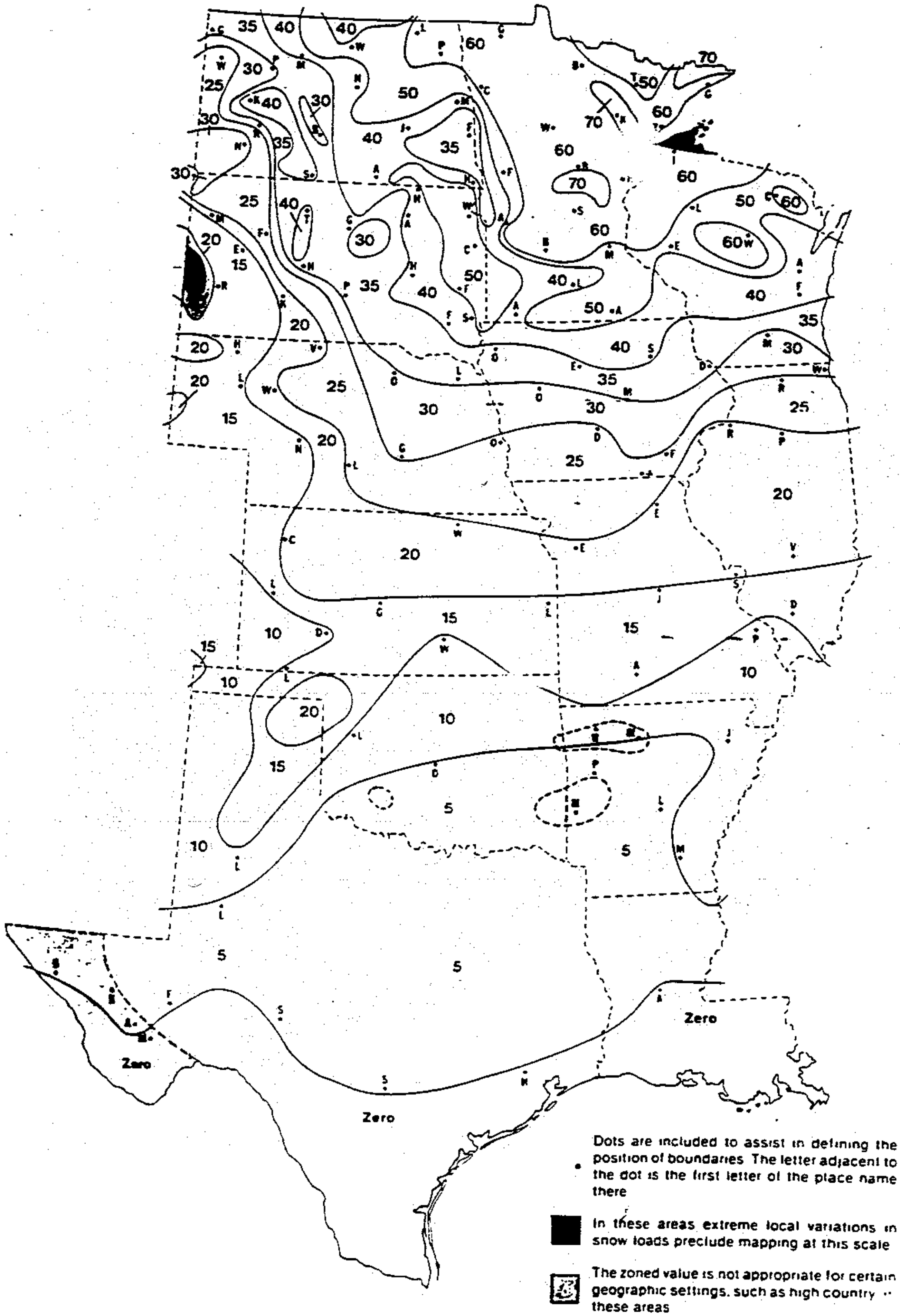


Dots are included to assist in defining the position of boundaries. The letter adjacent to the dot is the first letter of the place name there

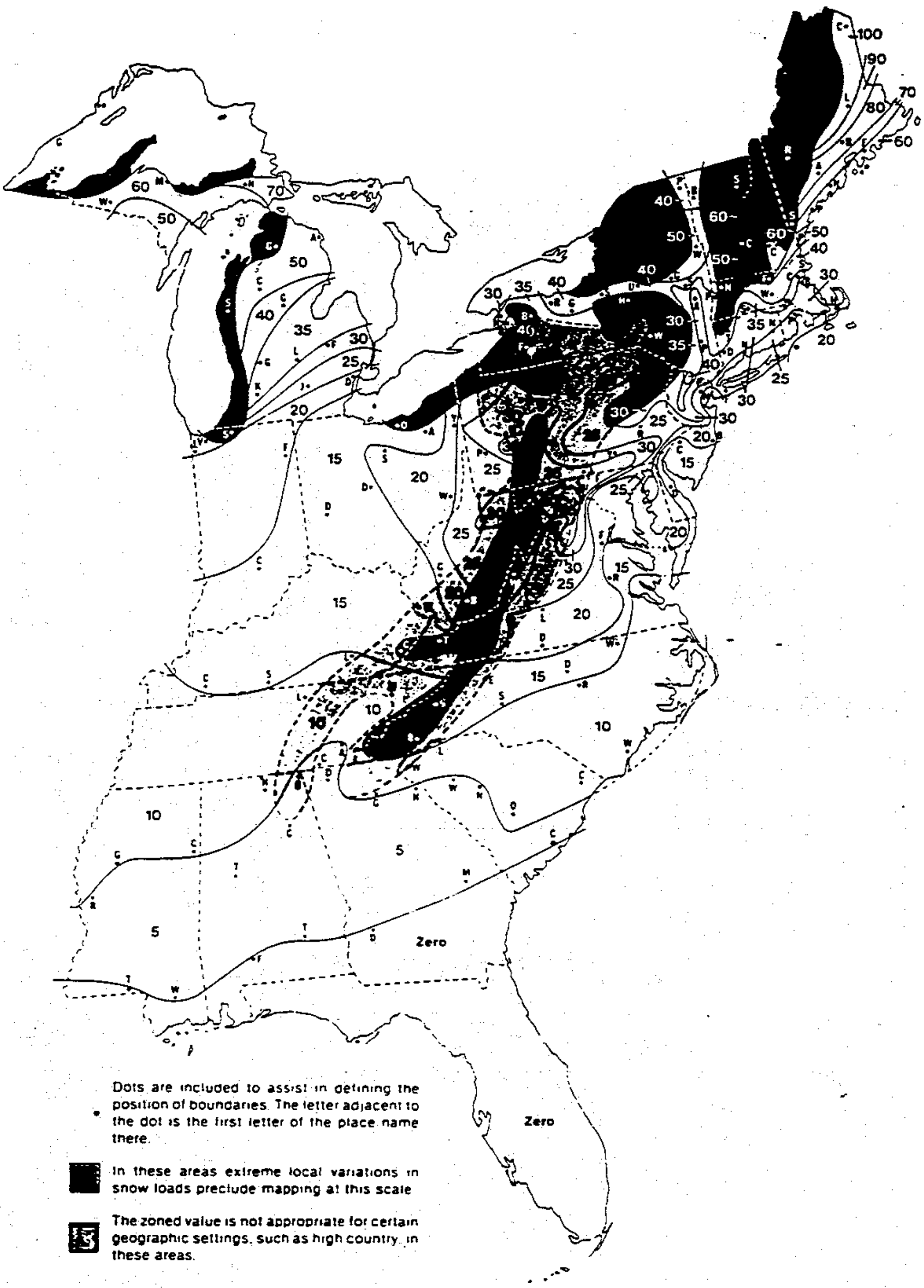
In these areas extreme local variations in snow loads preclude mapping at this scale

The zoned value is not appropriate for certain geographic settings, such as high country, in these areas

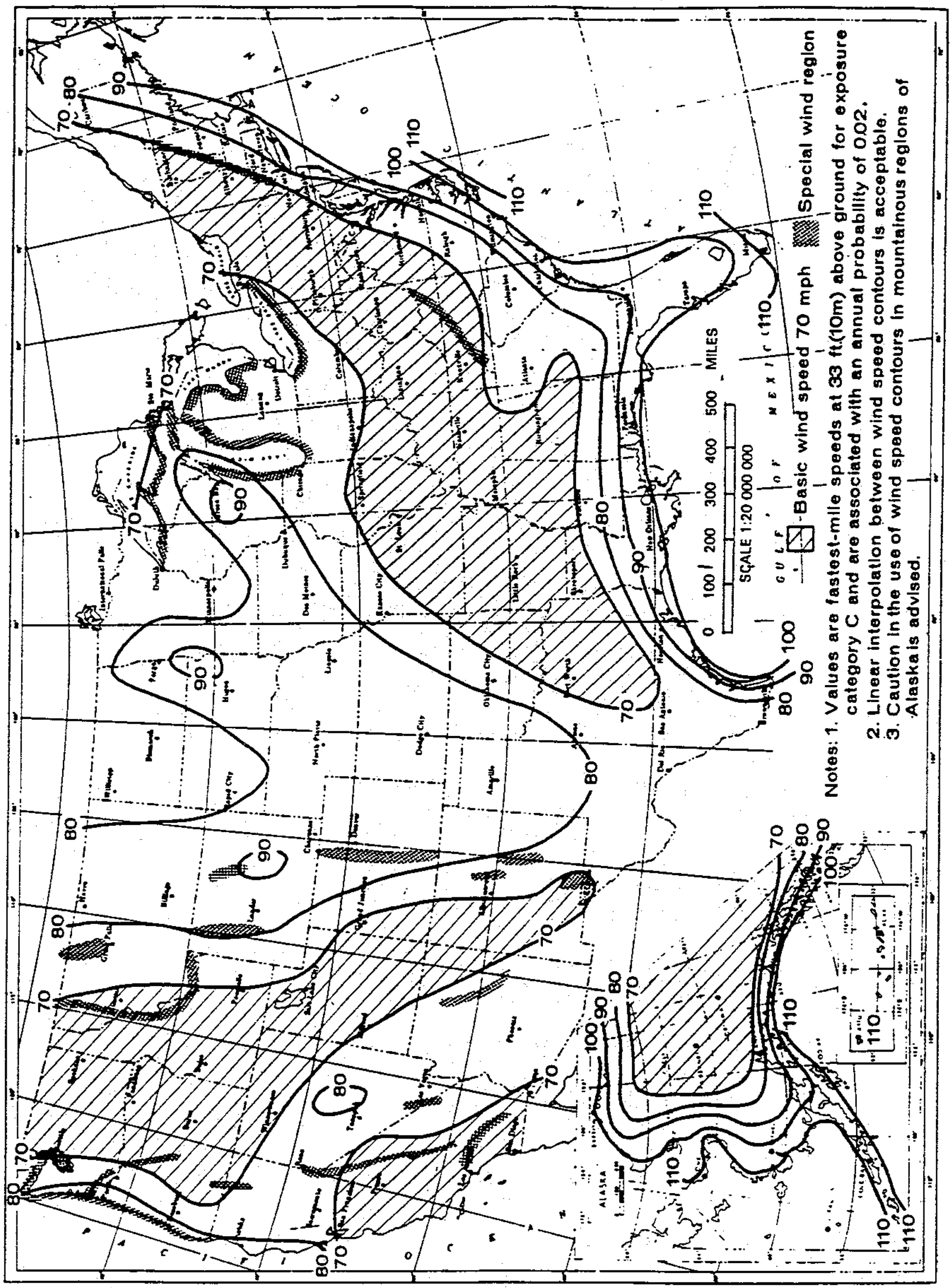
Ground Snow Loads, p_g, for the Western United States
(pounds per square foot)



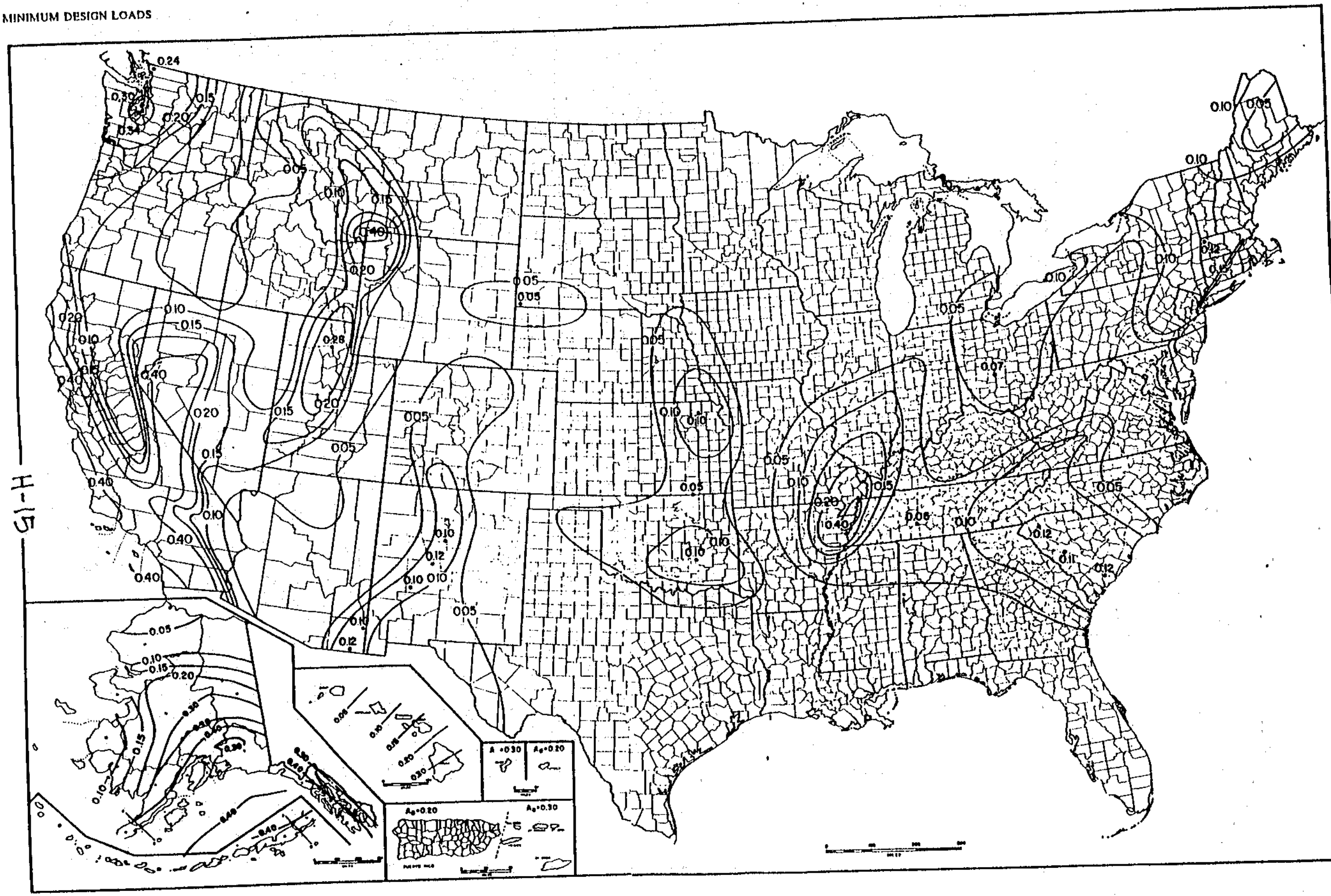
Ground Snow Loads, p_g, for the Central United States
(pounds per square foot)



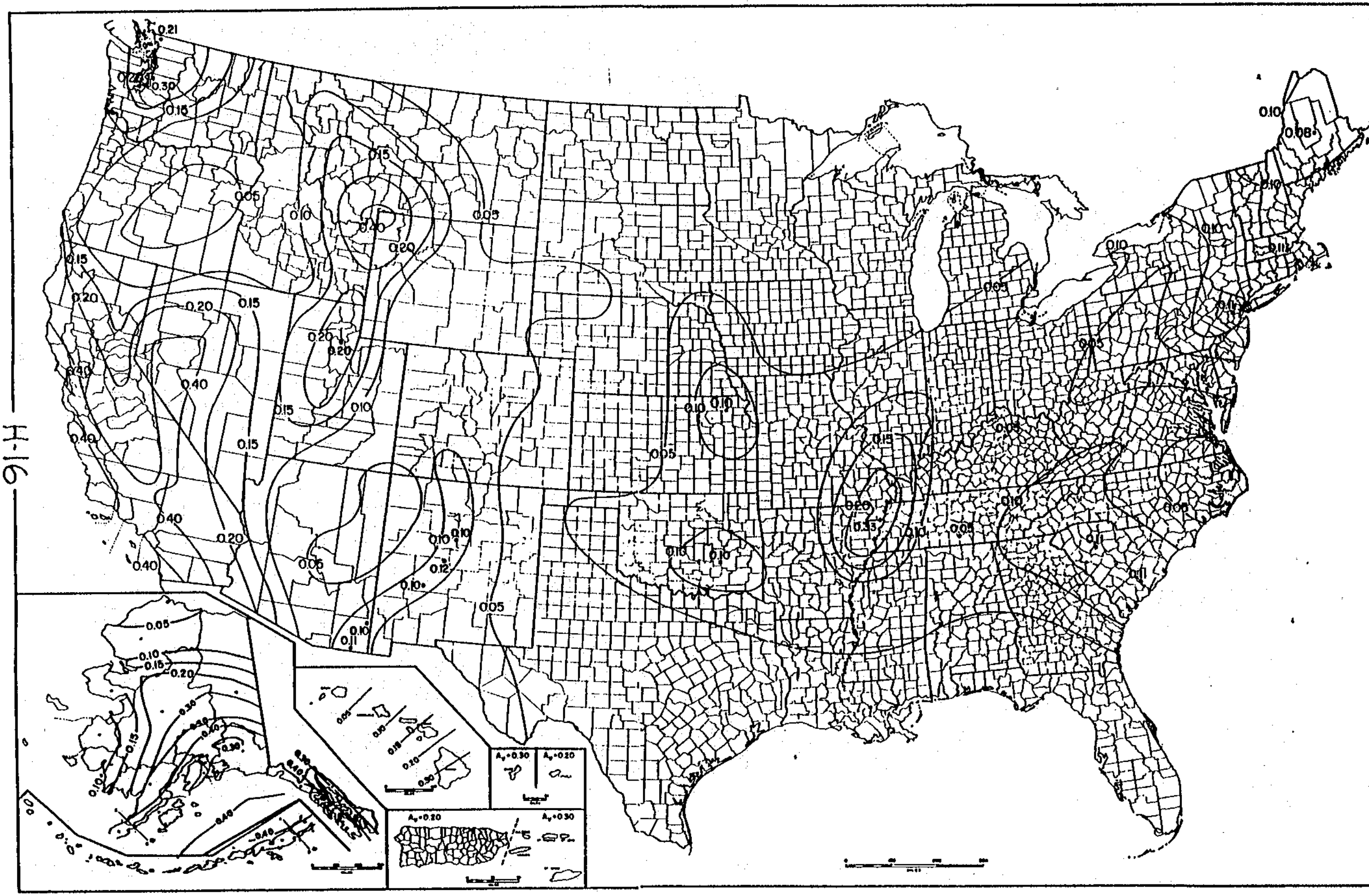
. **Ground Snow Loads, p_g, for the Eastern United States**
(pounds per square foot)



Basic Wind Speed (mph)



Contour Map for Coefficient A_a



Contour Map for Coefficient A_v

Table H-1
Seismic Performance Category for Seismic Hazard Exposure Group I

Effective Peak Velocity-Related Acceleration A_v	Seismic Performance Category
$A_v < 0.05$	A
$0.05 \leq A_v < 0.10$	B
$0.10 \leq A_v < 0.20$	C
$0.20 \leq A_v$	D

Manufactured Housing of Category A and B (one story detached one and two family dwellings which are located in seismic map area having an effective peak velocity-related acceleration (A_v) value less than 0.15) are exempt from the requirements of these provisions.

Manufactured Housing of Category C and D shall comply with all the requirements of these provisions.

APPENDIX I
REFERENCES AND ADDITIONAL RESOURCES

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