CHANGE

No. 1

HEADQUARTERS
DEPARTMENTS OF THE ARMY
AND THE AIR FORCE
Washington, DC 1 August 1993

STRUCTURAL DESIGN CRITERIA LOADS

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TECHNICAL MANUAL
No. 5-809-1
AIR FORCE MANUAL
No. 88-3, Chapter 1

HEADQUARTERS
DEPARTMENTS OF THE ARMY
AND THE AIR FORCE
Washington, DC *20 May 1992*

STRUCTURAL DESIGN CRITERIA LOADS

APPROVED FOR PUBLIC RELEASE; DISTRIBUTION IS UNLIMITED

*This manual supersedes TM 5-809-1/AFM 88-3, Chap. 1, dated 28 March 1986

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CHAPTER 2

COMBINATION OF LOADS

2-1. General

The following criteria stipulate combinations of loads to be considered in the design of structures and foundations. Combined loads produce the most unfavorable effect on foundations, structural members, and connections. Accordingly the designer will select the appropriate combined loads that create the most unfavorable affect when one or more of the contributing loads are present.

2-2. Combined loads for class A (bridge-type structures)

The design provisions of the American Association of State Highway and Transportation Officials (AASHTO) and the American Railway Engineering Association (AREA) will be used for class A structures.

2-3. Combined loads for class B (building-type structures) and class C (special structures)

The combined loads for class B and class C structures will be as specified in ASCE 7 with the following exceptions. For concrete construction, use the load combinations specified in ACI 318. However, for earthquake loading on concrete structures, use the load combinations specified in TM 5-809-10/AFM 88-3, Chap. 13. For timber construction, use the load combinations in the American Institute of Timber Construction (AITC) "Timber Construction Manual". As a clarification of the ASCE 7 requirements, note that allowable stresses will not be increased for wind, snow, or earthquake loads when used in conjunction with the ASCE 7 load combinations for allowable stress design. The increase is already considered in the combinations indicated in ASCE 7. The load combination factor for dead load and one transient load (e.g. wind load) is 1.0 for allowable stress design. Therefore, no increase in allowable stress is permitted for dead load and one transient load. However, the load combination factor is less than one for dead load combined with two or more transient loads. When designing for wind uplift and overturning due to loads such as wind and seismic, the minimum in lieu of maximum assumed dead loadings should be used in the load combinations.

2-4. Load reduction (Rescinded)

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AIR FORCE AFM 88-3, CHAP. 1

TECHNICAL MANUAL

STRUCTURAL DESIGN CRITERIA LOADS

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DEPARTMENTS OF THE ARMY AND THE AIR FORCE

MAY 1992

REPRODUCTION AUTHORIZATION/RESTRICTIONS

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CHAPTER 1

GENERAL

1-1. Purpose

This manual provides the structural criteria for loads to be used in the design and construction of buildings and other structures for the Army and the Air Force.

1-2. Scope

Load criteria presented in this manual apply to designs for new military construction and for modifications of existing buildings and other structures for the Army and the Air Force. Engineering judgment must be used in calculating design loads. The dead loads specified herein are for guidance only. The designer must determine and allow for the actual dead loads in the structure. The live, wind, and snow loadings specified herein are minimums. The designer should determine if special loadings must be considered.

1-3. References

Appendix A contains a list of references used in this document.

1-4. Basis for design

Except as modified herein, all design load criteria except seismic are based on the requirements in ASCE 7. ASCE 7 must be obtained and used in conjunction with this manual. Seismic loads are covered in TM 5-809-10/AFM 88-3, Chap. 13.

1-5. Classification of structures

The design load criteria in this manual is presented for three classes of structures as follows:

a. Class A (Bridge-Type Structures). Class A structures are those to which standard specifications for bridge-type structures are applicable. Included are bridges, trestles, viaducts (railway, highway, and pedestrian), and their components (beams, girders, columns, tension members, trusses, floors, bearings), certain weight-handling equipment, and piers carrying moving loads, as delineated in specific design manuals for these types of structures.

b. Class B (Building-Type Structures). Class B structures are those to which standard specifications for building-type structures are applicable. Typical examples of Class B structures are administration buildings, warehouses, and commissaries.

c. Class C (Special Structures). Class C covers special structures not readily classified in either of the above two categories, including storage tanks, cable guyed and supported structures, tension fabric structures, floating structures, and others designated as special structures in specific design manuals for these types of structures. Class C also covers temporary construction such as shoring, falsework, formwork, etc..

1-6. Application of design load criteria

The design load criteria for the above defined classes of structures will be based on the following sources.

a. Class A Structures. For Class A structures the provisions of the American Association of State Highway and Transportation Officials (AASHTO) and American Railway Engineering Association (AREA) design standards will be used.

b. Classes B and C Structures. For Classes B and C structures the applicable provisions of this manual will be used. Most of the criteria presented in this manual is for Class B (building- type) structures. Selected provisions for some Class C structures (including tension fabric structures) are included in Chapter 8.

1-7. Metal building systems

These are buildings which are supplied as a complete building unit. They are to be the product of one metal building supplier. As discussed below, Metal Building Systems may be either Standard Metal Building Systems or Special Purpose Metal Building Systems.

a. Standard Metal Building Systems. Standard Metal Building Systems are Metal Building Systems that are designed in accordance with "Low Rise Building Systems Manual" by the Metal Building Manufacturers Association (MBMA). These buildings typically have an eave height equal to or less than 20 feet, or have rigid frame spans less than or equal to 80 feet. However, as discussed below, Metal Building Systems may be considered Special Purpose Metal Building Systems due to factors other than size. Typical examples of Standard Metal Building Systems include warehouses, pump houses, and servicing facilities. Load combinations and procedures for developing the design loads for Standard Metal Building Systems will follow the criteria in the MBMA publication "Low Rise Metal Building Systems Manual". The following data will be used in developing design loads for Standard Metal Building Systems:

(1) Dead loads, floor live loads, basic wind speeds, and ground snow loads will be in accordance with the

requirements in this document. Roof live loads will be in accordance with MBMA requirements.

(2) Seismic zone will be obtained from TM 5-809-10/AFM 88-3, Chapter 13. Note that TM 5-809-10/AFM 88-3 , Chapter 13 has zones 2A and 2B instead of zone 2 as in MBMA. Zone 2A corresponds to zone 2 in MBMA. For buildings in zone 2B, use Z = 0.50 in the lateral force equation for seismic loads in MBMA.

(3) Importance factors for wind and snow loads will be obtained from ASCE 7. Importance factors for seismic loads will be obtained from TM 5-809-10/AFM 88-3, Chapter 13. The building category (I, II, III, IV), which is based on the building occupancy and is used in determining the importance factor, will be obtained from this document for wind and snow loads and from TM 5-809-10/AFM 88-3, Chapter 13 for seismic loads.

b. Special Purpose Metal Building Systems. Special Purpose Metal Building Systems are Metal Building Systems designed by the manufacturer to meet the loadings specified herein. These buildings have an eave height greater than 20 feet or rigid frame spans greater than 80 feet, or are buildings considered to be special application due to factors other than size, such as use, replacement value of contents, or location. Typical examples may be gymnasiums, aircraft hangars, maintenance shops, or other clear span industrial type buildings. For Special Purpose Metal Building Systems, the load criteria specified herein and in TM 5-809-10/AFM 88-3, Chapter 13 will be used in place of the MBMA load criteria.

1-8. Building categories for wind and snow loads

Buildings are categorized according to occupancy. The categories described in ASCE 7 (with the following modifications) will be used to determine wind and snow loads:

a. Add to the list of Category II buildings: Buildings housing expensive items, i.e. aircraft, computer equipment, etc.

b. Add to the list of Category III buildings:

(1) Facilities involving missile operations.

(2) Facilities involving sensitive munitions, fuels, and chemical and biological contaminants.

(3) Facilities involving strategic communications.

1-9. Wind, snow, and frost depth data

Appendices B and C provide wind, snow, and frost depth data for various major cities and military installations in the U.S. and outside the U.S., respectively. Appendix D contains a procedure for determining the design depth for building foundations based on the frost depth data from Appendices B and C.

1-10. Design examples

Design examples are included in appendices E, F, G, and H. These examples illustrate how specific load requirements in this manual are implemented. Unless noted otherwise, ASCE 7 requirements (i.e. tables, figures, equations, etc.) were used to solve the design examples.

CHAPTER 2

COMBINATION OF LOADS

2-1. General

The following criteria stipulate combinations of loads to be considered in the design of structures and foundations. Combined loads produce the most unfavorable effect on foundations, structural members, and connections. Accordingly the designer will select the appropriate combined loads that create the most unfavorable affect when one or more of the contributing loads are present.

2-2. Combined loads for class A (bridge-type structures)

The design provisions of the American Association of State Highway and Transportation Officials (AASHTO) and the American Railway Engineering Association (AREA) will be used for class A structures.

2-3. Combined loads for class B (building-type structures) and class C (special structures)

The combined loads for class B and class C structures will be as specified in ASCE 7 with the following exceptions. For concrete construction, use the load combinations specified in ACI 318. However, for earthquake loading on concrete structures, use the load combinations specified in TM 5-809-10/AFM 88-3, Chap. 13. For timber construction, use the load combinations in the American Institute of Timber Construction (AITC) "Timber Construction Manual". As a clarification of the ASCE 7 requirements, note that allowable stresses will not be increased for wind, snow, or earthquake loads when used in conjunction with the ASCE 7 load combinations for allowable stress design. The increase is already considered in the combinations indicated in ASCE 7. The load combination factor for dead load and one transient load (e. g. wind load) is 1.0 for allowable stress design. Therefore, no increase in allowable stress is permitted for dead load and one transient load. However, the load combination factor is less than one for dead load combined with two or more transient loads. When designing for wind uplift and overturning due to loads such as wind and seismic, the minimum in lieu of maximum assumed dead loadings should be used in the load combinations.

2-4. Load reduction

Criteria provided in this manual are based on permanent construction. Design for reduced wind, snow, and seismic loads is permissible for limited life structures, as well as for structural configurations during phases of construction, and for temporary works used to facilitate permanent construction. For structures having design service lives of one year or less, the wind, snow, and seismic loads which would apply for the design of a comparable permanent facility may be reduced to 0.75 times the full value. (For wind load, note that the reduction factor is applied to the wind pressure, not the wind velocity). For structures having design service lives between 1 and 5 years, the load reduction may be interpolated between a value of 0.75 and 1.0. Note that no increase in allowable stresses is permitted when these reduced loadings are used, since the reduced loadings have the same effect on design as raising the allowable stresses.

CHAPTER 3

DEAD LOADS

3-1. General

Except as modified herein, the criteria for dead loads will be as specified in ASCE 7.

3-2. Supplementary design dead loads

Design dead loads presented in this manual will supplement the design dead loads tabulated in the commentary section of ASCE 7. Unit weights are given in table 3-1. Design dead loads for assembled elements of construction are given in table 3-2. The dead loadings for reinforced hollow masonry unit construction should be based on the weights given in TM 5-809-3/AFM 88-3, Chapter 3. In case of a conflict between the dead loads in this manual and ASCE 7, the higher value should be used unless the designer has other information or guidance.

Table 3-1. Unit Weights[1]

Material	*pcf*
Metals, alloys, ores:	
Aluminum, cast, hammered	165
Gold, cast, hammered	1205
Gold, bars, stacked	1133
Gold, coin in bags	1084
Iron, spiegelcisen	468
Iron, ferrosilicon	437
Iron ore, hematit	325
Iron ore, hematite in bank	160-180
Iron ore, hematite loose	130-160
Iron ore, limonite	237
Iron ore, magnetite	315
Iron slag	172
Magnesium, alloys	112
Manganese	475
Manganese ore, pyrolusite	259
Mercury	849
Monel meta	1556
Nicke	1565
Platinum, cast, hammered	1330
Silver, cast, hammered	656
Silver bars, stacked	590
Silver coin in bags	590
Timber, U.S. seasoned: Moisture content by weight: (Seasoned timber, 15 to 20% green timber, up to 50%)	
Cedar, white, red	22
Chestnut	41
Cypress	30
Elm white	45
Hickory	49
Locust	46
Maple, hard	43

Table 3-2. Design Dead Loads[1]

Walls[2]	*psf*
4-inch clay brick, high absorption	34
4-inch clay brick, medium absorption	39
4-inch clay brick, low absorption	46
4-inch sand-lime brick	38
8-inch clay brick, high absorptin	69
8-inch clay brick, medium absorption	79
8-inch clay brick, low absorption	89
8-inch sand-lime brick	74
12 1/2-inch clay brick, high absorption	100
12 1/2-inch clay brick, medium absorption	115
12 1/2-inch clay brick, low absorption	130
12 1/2-inch sand-lime brick	105
12 1/2-inch concrete brick, heavy aggregate	130
12 1/2-inch concrete brick, light aggregate	98
17-inch clay brick, high absorption	134
17-inch clay brick, medium absorption	155
17-inch clay brick, low absorption	173
17-inch sand-lime brick	138
17-inch concrete brick, heavy aggregate	174
17-inch concrete brick, light aggregate	130
22-inch clay brick, high absorption	168
22-inch clay brick, medium absorption	194
22-inch clay brick, low absorption	216
22-inch sand-lime brick	173
22-inch concrete brick, heavy aggregate	216
22-inch concrete brick, light aggregate	160
4-inch brick, 4-inch load-bearing structural clay tile backing	60
4-inch brick, 8-inch load-bearing structural clay tile backing	75
8-inch brick, 4-inch load-bearing structural clay tile backing	102
8-inch load-bearing structural clay tile	42
12-inch load-bearing structural clay tile	58
2-inch furring tile, one side of masonry wall, add to above figures	12

Partitions[2]	*psf*
3-inch clay tile	17
4-inch clay tile	18
6-inch clay tile	28
8-inch clay tile	34
10-inch clay tile	40
2-inch facing tile	15
4-inch facing tile	25
6-inch facing tile	38
2-inch gypsum block	9-1/2
3-inch gypsum block	10-1/2
4-inch gypsum block	12-1/2

Table 3-2. Design Dead Loads[1] (continued)

Partitions[2] (Cont'd)	*psf*
5-inch gypsum block	14
6-inch gypsum block	18-1/2
2-inch solid plaster	20
4-inch solid plaster	32
4-inch hollow plaster	22
Glass block masonry:	
4-inch glass-block walls and partitions	18
Asbestos hard board (corrugated), per 1/4-inch of thickness	3
Stone, 4-inch	55
Split furring tile:	
1 1/2-inch	8
2-inch	8-1/2

Roof and Wall Coverings	*psf*
Cold applied sheet membrane and stone ballast	sec mfr.
Corrugated iron	2
Decking (non wood) per inch of thickness:	
Concrete plank	6.5
Poured gypsum	6.5
Vermiculite concrete	2.6
Glass:	
Single strength	1.2
Double strength	1.6
Plate, wired or structural, 1/8-inch	1.6
Insulating, double 1/8-inch plates w/air space	3.5
Insulating, double 1/4-inch plates w/air space	7.1
Insulation, per inch of thickness:	
Expanded polystyrene	0.1
Extruded polystyrene	0.2
Loose	0.5
Urethane	1.0
Cork	1.0
Batts and blankets	0.5
Insulating concrete	3.0
Marble, interior, per inch	**14.0**
Metal deck (22 gauge)	**1.9**

Table 3-2. Design Dead Loads[1] (continued)

Roof and Wall Coverings (Cont'd)	*psf*
Plastic, acrylic, 1/4-inch	**1.5**
Porcelain enamel on sheet steel	**3.0**
Stucco, 7/8-inch	**10.0**
Terra cotta tile	**25.0**

[1] This table supplements the dead loads tabulated in ASCE 7. For reinforced hollow masonry unit construction, the dead loadings should be based on the weights given in TM 5-809-3/AFM 88-3, Chapter 3.

[2] For masonry construction, add 5 psf for each face plastered.

CHAPTER 4

LIVE LOADS

4-1. General

Except as modified herein, the criteria for live loads will be as specified in ASCE 7.

4-2. Supplementary design live loads

The following live load requirements will supplement the live load criteria in ASCE 7:

a. Minimum Design Live Loads. Minimum uniformly distributed live loads are given in table 4-1. Uniform live loads for storage warehouses are given in table 4-2. In case of a conflict between the live loads in this manual and ASCE 7, the higher value should be used unless the designer has other information or guidance.

b. Provision for Partitions. In buildings where partitions are subject to rearrangement, the following equivalent load may be used as a suggested minimum load:

Partition Weight (pound per lineal foot of partition)	Equivalent Uniform Load (pounds per square foot)
0-50	0
51-100	6
101-200	12
201-300	20
Over 300	Use actual concentrated linear load

Note that the above loads may be smaller than the actual loads for one-way joist systems where the partition runs parallel to the joist. When designing these floor systems, the designer must consider the actual weight of the partition directly over the joist. Some distribution of partition loadings to adjacent floor joists or beams may be appropriate when the floor construction is a concrete slab.

c. Concentrated Live Loads. The following concentrated loads must be considered in addition to the dead loads:

(1) Accessible, open-web steel joists supporting roofs over manufacturing, commercial storage and warehousing, and commercial garage floors will be designed to support the uniformly distributed live load prescribed in ASCE 7 in addition to a concentrated live load of 800 pounds. For all other occupancies, a load of 200 pounds will be used instead of 800 pounds. The concentrated live load will be placed at any single panel point on the bottom chord, and will be located so as to produce the maximum stress in the member.

(2) As a clarification of the ASCE 7 requirements, accessible roof trusses or other primary roof-supporting members will be designed to support the concentrated live load prescribed in ASCE 7 in addition to the dead load and the uniformly distributed roof live load.

(3) Members such as floor decking, roof decking and rafters will be designed to support the uniformly distributed live loads prescribed in ASCE 7 or a concentrated live load of 200 pounds, whichever produces the greater stress. The concentrated live load will be assumed to be uniformly distributed over a 12- by 12-inch square area and will be located so as to produce the maximum stress in the member.

(4) Boiler rooms will be designed to support the uniformly distributed live loads prescribed in ASCE 7 or a 3000 pound concentrated live load, whichever produces the greater stress. The concentrated live load will be applied over an area of 2.5 feet square (6.25 sq ft) (in areas outside the limits of the boilers) and will be located so as to produce the maximum stress.

d. Impact Loads on Escalators. Escalator live loads will be increased by 15 percent for impact.

Table 4-1. Minimum Uniform Live Load Requirements[1]

Occupancy or Use	*Live Load (psf)*
Bag storage	125
Barber shop	75
Battery charging room	200
Car wash rooms	75
Canteens, general area	100
Canteens, general area	200
Catwalks, Marine	50
Chapels:	
Aisles, corridors, and lobbie	100
Balconies	60
Fixed seats	60
Offices and miscellaneous rooms	40
Day rooms	60
Drawing	100
Drum fillings	150
Drum washing	75
File rooms (drawing files)	200
Galleys:	
Dishwashing rooms (mechanical)	300
Provision storage (not refrigerated)	200
Galley Preparation room:	
Meat	250
Vegetable	100
Garbage storage rooms	125
Generator rooms	200
Guard house	75
Hangars	See Footnote[2]
Latrines	75
Linen storage	125
Lobbies, vestibules and large waiting rooms	100
Locker rooms	75
Lounges, day rooms, small recreation areas	60
Mechanical equipment rooms (general)	100
Mechanical room (air conditioning)	125
Mechanical telephone and radio equipment rooms	150
Mess halls	100
Post offices:	
General area	100
Work rooms	125
Power plants	200
Promenade roof	60
Pump houses	100
Rcreation rooms	100

Table 4-1. Minimum Uniform Live Load Requirements[1] (continued)

Occupancy or Use (Cont'd)	*Live Load (psf)*
Receiving rooms (radio) including roof areas supporting antennas and electronic equipment	150
Refrigeration storage rooms:	
Dairy	200
Meat	250
Vegetables	275
Rubbish storage rooms	100
Scrub decks	75
Shops:	
Aircraft utility	200
Assembly and repair	250 to 400
Blacksmith	125
Bombsight	125
Carpenter	125
Drum repair	100
Electrical	300
Engine overhaul	300
Heavy materials assembly	200 to 400
Light materials assembly	125
Machine	300
Mold loft	80
Plate (except storage areas)	300
Public works:	
First floor	125
Sheet metal	125
Shipfitters	300
Structural	300
Upper floors	100
Schools (shops)	60
Sidewalks not subject to trucking	250
Showers and washrooms	60
Store houses:	
Ammunition (one story)	2,000
Dry provisions	300
Fuse and detonator (one story)	500
High explosives (one story)	500
Inert materials (one story)	500 to 2,000
Light tools	150
Paint and oil (one story)	500
Pipe and metals (one story)	1,000
Pyrotechnics (one story)	500
Small arms (one story)	500

Table 4-1. Minimum Uniform Live Load Requirements[1] *(continued)*

Occupancy or Use (Cont'd)	*Live Load (psf)*
Subsistence buildings	200
Torpedo (one story)	350
Tailor shop	75
Telephone exchange rooms at locations subject to earth tremors, gunnery practice or other conditions causing unusual vibrations	250
Terminal equipment buildings (all areas other than stairs, toilets, and washrooms)	150

[1]This table supplements the live loads tabulated in ASCE 7.

[2]The designer must determine the wheel loads of aircraft and impact factors.

Table 4-2. Minimum Uniform Live Loads for Storage Warehouses[1]

Material	Weight per Cubic foot of Space (lb)	Height of Pile (ft)	Weight per Sq. Ft. of Floor[2] (lb)
Building materials:			
Asbestos	50	6	300
Bricks, building	45	6	270
Bricks, fire clay	75	6	450
Cement, portland	72 to 105	6	432 to 630
Gypsum	50	6	300
Lime and plaster	53	5	265
Tiles	50	6	300
Woods, bulk	45	6	270
Drugs, paints, oil:			
Alum, pearl, in barrels	33	6	198
Bleaching powder, in hogsheads	31	3- 1/2	102
Blue vitriol, in barrels	45	5	226
Glycerine, in cases	52	6	312
Linseed oil, in barrels	36	6	216
Linseed oil, in iron drums	45	4	180
Logwood extract, in boxes	70	5	350
Rosin, in barrels	48	6	288
Shellac, gum	38	6	228
Soaps	50	6	300
Soda ash, in hogsheads	62	2- 3/4	167
Soda, caustic, in iron drums	88	3 3/8	294
Soda, silicate, in barrels	53		
Sulphuric acid	60	1- 5/8	100
Toilet articles	35	6	210
Varnishes	55	6	330
White lead paste, in cans	174	3- 1/2	610
White lead, dry	86	4- 3/4	408
Red lead and litharge, dry	132	3- 3/4	495
Dry goods, cotton, wool:			
Burlap, in bales	43	6	258
Carpets and rugs	30	6	180
Coir yarn, in bales	33	8	264
Cotton, in bales, American	30	8	240
Cotton, in bales, foreign	40	8	320
Cotton bleached goods, in cases	28	8	224
Cotton flannel, in cases	12	8	96
Cotton sheeting, in cases	23	8	184
Cotton yarn, in cases	25	8	200
Excelsior, compressed	22	8	152

Table 4-2. Minimum Uniform Live Loads for Storage Warehouses[1] (continued)

Material (Cont'd)	Weight per Cubic Foot of Space (lb)	Height of Pile (ft)	Weight per Sq. Ft. pf Floor[2] (lb)
Hemp, Italian, compressed	22	8	176
Hemp, Manila, compressed	30	8	240
Jute, compressed	41	8	328
Linen damask, in cases	50	5	250
Linen goods, in cases	30	8	240
Linen towels, in cases	40	6	240
Silk and silk goods	45	8	360
Sisal, compressed	21	8	168
Tow, compressed	29	8	232
Wool, in bales, compressed	48	-	-
Wool, in bales, not compressed	13	8	104
Wool, worsteds, in cases	27	8	216
Groceries, wines, liquors:			
Beans, in bags	40	8	320
Beverages	40	8	320
Canned goods, in cases	58	6	348
Cereals	45	8	360
Cocoa	35	8	280
Coffee, roasted, in bags	33	8	264
Coffee, green, in bags	39	8	312
Dates, in cases	55	6	330
Figs, in cases	74	5	370
Flour, in barrels	40	5	200
Fruits, fresh	35	8	280
Meat and meat products	45	6	270
Milk, condensed	50	6	300
Molasses, in barrels	48	5	240
Rice, in bags	58	6	348
Sal soda, in barrels	46	5	230
Salt, in bags	70	5	350
Soap powder, in cases	38	8	304
Starch, in barrels	25	6	150
Sugar, in barrel	43	5	215
Sugar, in cases	51	6	306
Tea, in chests	25	8	200
Wines and liquors, in barrels	38	6	228
Hardware:			
Automobile parts	40	8	320
Chain	100	6	600
Cutlery	45	8	360
Door checks	45	6	270

Table 4-2. Minimum Uniform Live Loads for Storage Warehouses[1] (continued)

Material (cont'd)	*Weight per Cubic Foot of Space (lb)*	*Height of Pile (ft)*	*Weight per Sq. Ft. of Floor[2] (lb)*
Electrical goods and machinery	40	8	320
Hinges	64	6	384
Locks, in cases, packed	31	6	186
Machinery, light	20	8	160
Plumbing fixtures	30	8	240
Plumbing supplies	55	6	330
Sash fasteners	48	6	288
Screws	101	6	606
Shafting steel	125	-	-
Sheet tin, in boxes	278	2	556
Tools, small, metal	75	6	450
Wire cables, on reels	-	-	
425			
Wire, insulated copper, in coils	63	5	315
Wire, galvanized iron, in coils	74	4 1/2	333
Wire, magnet, on spools	75	6	450
Miscellaneous:			
Automobile tires	30	6	180
Automobiles, uncrated	8	-	64
Books (solidly packed)	65	6	390
Furniture	20	-	-
Glass and chinaware, in crates	40	8	320
Hides and leather, in bales	20	8	160
Leather and leather goods	40	8	320
Paper, newspaper, and strawboards	35	6	210
Paper, writing and calendared	60	6	360
Rope, in coils	32	6	192
Rubber, crude	50	8	400
Tobacco, bales	35	8	280

[1] This table supplements the live loads tabulated in ASCE 7.

[2] Tabulated live loads are for stack storage warehouses. For rack storage warehouses, the designer must consider the higher concentrated loads from the racks.

CHAPTER 5

WIND LOADS

5-1. General

Except as modified herein, the criteria for wind loads will be as specified in ASCE 7.

5-2. Supplementary requirements

The following requirements supplement or modify the criteria for wind loads given in ASCE 7.

a. Basic Wind Speed. Site-specific wind data for major cities and installations in the United States and outside the United States are tabulated in appendices B and C, respectively. Note that this data will be used in lieu of the wind data tabulated in ASCE 7. For locations not tabulated in Appendices B or C, the basic wind speed in ASCE 7 may be used.

b. Wind Pressures on Open Sheds. The wind force coefficient for open sheds is given in figure 5-1.

c. Minimum Design Wind Pressures on Interior Partitions. The minimum design wind pressure on interior partitions shall be five psf normal to the partition and its supporting parts; i.e., studs.

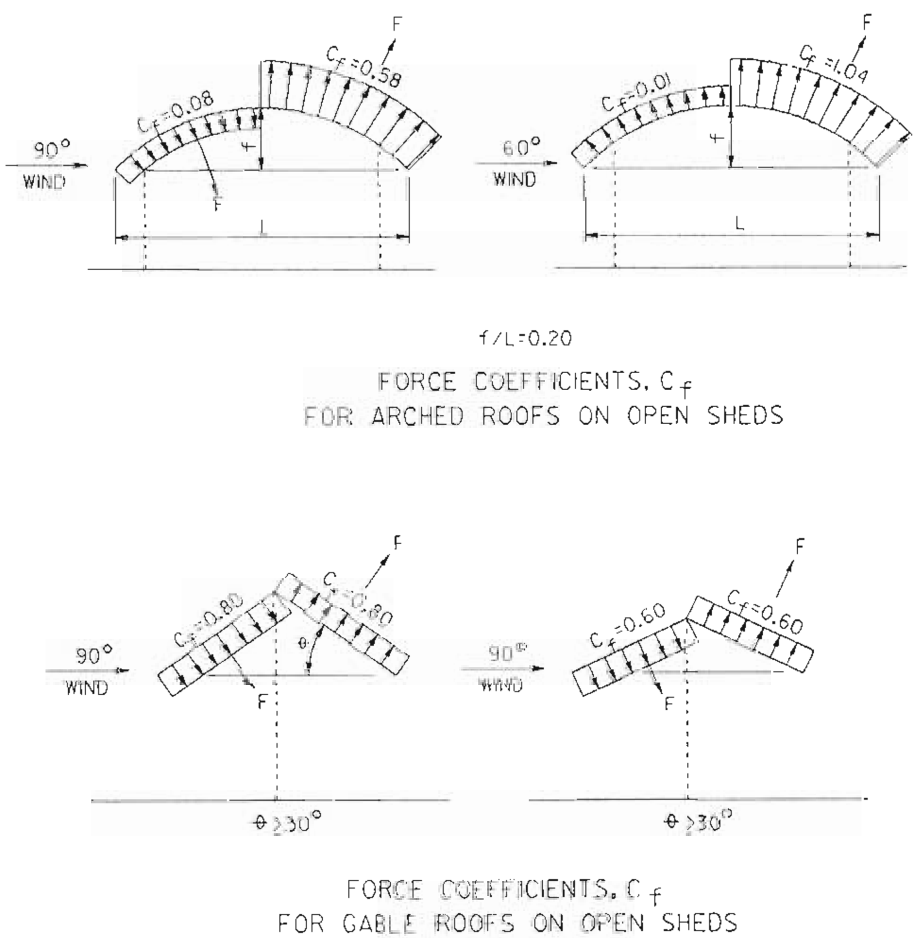


Figure 5-1. Wind force coefficients for open sheds.

CHAPTER 6
SNOW LOADS

6-1. General

Except as modified herein, the criteria for snow loads will be as specified in ASCE 7.

6-2. Definitions

The following definitions for the snow load requirements in ASCE 7 are provided:

a. Arched Roof. Curved roof; i.e., circular, parabolic, etc.

b. Balanced Snow Load. Snow load, either flat roof design load, p_f, or sloped roof design load, p_s, applied to the entire horizontal projection of a roof.

c. Barrel-Vaulted Roof. A roof consisting of a series of segmental arches.

d. Crown. The highest point on an arch.

e. Exposure Factor, C_e. A factor accounting for the nature of the site.

f. Eaves. A margin or lower part of a roof. For an arched roof with a slope exceeding 70 degrees, “eaves”, as used herein, refers to the point where the slope is equal to 70 degrees.

g. “Flat” Roof. As used herein, a roof with a slope less than 1 in./ft; i.e., less than 5 degrees.

h. Gable Roof. A double-sloped roof that forms a vertical triangular end of a building from the level of the eaves to the ridge of the roof.

i. Ground Snow Load. The reference snow load on the ground from which design roof snow loads are determined. The reference snow load has a 50-year mean recurrence interval (i.e. it is the snow load that has a 2 percent annual probability of being equaled or exceeded).

j. Hip Roof. A roof which rises by inclined planes from all four sides of a building. The line where two adjacent roof planes meet is called the “hip”.

k. Multiple Folded Plate Roof. A form of roof, consisting of a series of flat plates in a variety of shapes, such as V-shape, trapezoidal or Z-shape.

l. Slope Factor, C_s. A factor accounting for the decreased snow load on a sloped roof.

m. Snow Load Importance Factor, I. A factor accounting for variations in hazards to human life and damage to property for various structures.

n. Thermal Factor, C_t. A factor accounting for increases in snow load if the roof is cold.

o. Unbalanced Snow Load. Increased snow load applied to only a portion of a sloped roof. Unbalanced loads may develop on sloped roofs because of sunlight and wind. Wind tends to reduce snow loads on windward portions and increase snow loads on leeward portions.

6-3. Supplementary requirements

The following requirements supplement the criteria for snow loads given in ASCE 7.

a. Snow Data. Site-specific snow data for major cities and installations in the United States and outside the United States are tabulated in appendices B and C, respectively. The data in appendices B and C will be used in lieu of the mapped information in ASCE 7. The mapped information will be used for locations not tabulated in Appendix B. For locations in the black areas of the mapped information, consult the Cold Regions Research and Engineering Laboratory (CECRL).

b. Unbalanced Snow Loads for Multiple Folded Plate and Barrel Vault Roofs. For calculating the unbalanced snow load on the first windward and last leeward slope of multiple barrel vault roofs, use the criteria in ASCE 7 for unbalanced snow load on curved roofs. Balanced and unbalanced loading diagrams for a multiple folded plate roof are presented in figure 6-1. (Note that ASCE 7 does not address specifically the unbalanced snow load on the first windward and last leeward slopes for multiple folded plate and barrel vault roofs).

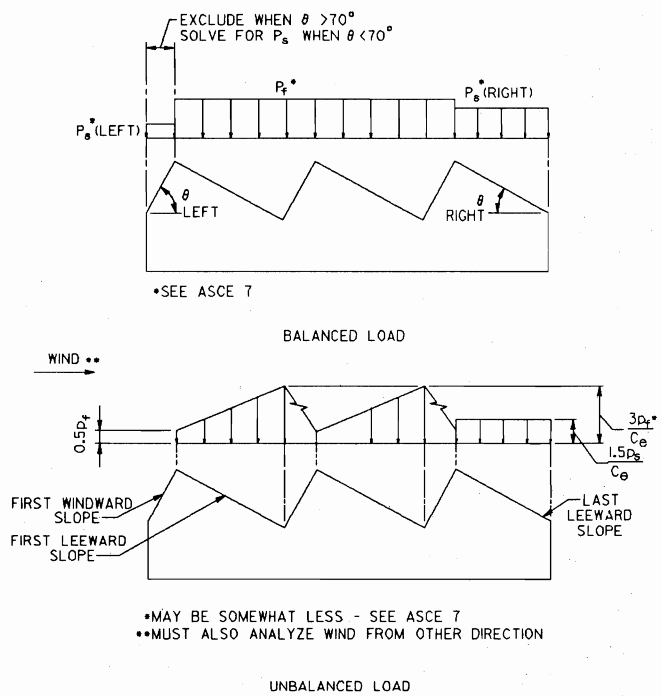


Figure 6-1. Balanced and unbalanced snow loads for multiple folded plate roofs.

6-4. Rain-on-snow loads

The recommendations for establishing the magnitude of rain-on-snow surcharge loads contained in ASCE 7 standard and commentary will be considered in structural design.

CHAPTER 7

OTHER LOADS

7-1. Earthquake loads

Criteria for developing earthquake loads for buildings and other structures are presented in TM 5-809-10/AFM 88-3, Chap. 13, and TM 5-809-10-1/AFM 88-3, Chap. 13, Sec. A.

7-2. Foundation loads and earth pressures

Standards for determining foundation loads, earth pressures, and foundation displacement and settlement are contained in TM 5-818-1/AFM 88-3, Chap. 7; NAVFAC DM-7.01 and NAVFAC DM-7.02.

7-3. Fluid pressures and forces

Consider the following fluid pressures and forces in structural design:

a. Hydrostatic Pressure. Use the hydrostatic pressure criteria in NAVFAC DM-7.02. For structures loaded with buoyant forces, the following additional guidance will be used: Adhesion resistance to flotation should not be used unless the designer knows that the buoyant forces will be short term and the adhesion will not be lost due to creep.

b. Wave and Current Forces. Wave force criteria are described in MIL-HDBK-1025/1, MIL-HDBK-1025/4, NAVFAC DM-25.05, and MIL-HDBK-1025/6.

7-4. Thermal forces

Provide for stresses or expansion/contraction resulting from variations in temperature. On cable structures, consider changes in cable sag and tension. Determine the rises and falls in the temperature for the localities in which structures are built. Establish these rises and falls from assumed temperatures at times of erection. Consider the lags between air temperatures and interior temperatures of massive concrete members or structures.

a. Temperature Ranges. Refer to the AASHTO design standard for the ranges of temperature for exterior, exposed elements.

b. Thermal Expansion/Contraction in Building Systems. The design of framing within enclosed buildings seldom need consider the forces or expansion/contraction resulting from a variation in temperature of more than 30 degrees to 40 degrees. The effects of such forces or expansion/contraction often are neglected in the design of buildings having plan dimensions of 250 feet or less, although movements of 1/4 to 3/8 inch can develop and may be important for buildings constructed with long bearing walls parallel to direction of movement.

c. Piping. To accommodate changes in length due to thermal variations, pipes frequently are held at a single point. If the pipes are held at more than one point, thermal forces must be included in the design of support framing.

7-5. Friction forces

a. Sliding Plates. Use 10 percent of the dead load reactions for clean bronze or copper-alloy sliding plates in new condition. Consult manufacturer for special systems.

b. Rockers or Rollers. Use 3 percent of the dead load reactions when employing unobstructed rockers or rollers.

c. Foundations on Earth. Criteria for foundations on earth are contained in NAVFAC DM-7.01.

d. Other Bearings. Use the "Standard Handbook for Mechanical Engineers" for coefficients of friction. Base the forces on dead load reactions plus any applicable long-time live load reactions.

7-6. Shrinkage

Investigate arches, fixed-fixed spans, indeterminate and similar structures for stresses induced by shrinkage and rib shortening.

7-7. Relaxation of initial forces

Cable structures, fabric structures, etc. are installed under initial tension which tends to slacken with time. This effect should be considered by handling the resulting stresses or providing the means to readjust the tension.

7-8. Blast loading

See TM 5-1300/AFM 88-22 and TM 5-855-1.

7-9. Nuclear weapon effects

See TM 5-858 Series.

7-10. Sway load on spectator stands

Provide for a lateral load effect equal to 24 pounds per linear foot of seating applied in a direction parallel to each row of seats and 10 pounds per linear foot of seating applied in a direction perpendicular to the row of seats. Apply these two components of sway load simultaneously. The sway load on spectator stands is considered to be concurrent with a wind load generated by a wind velocity equal to one-half the velocity of the design wind load, but not more than 50 miles per hour.

7-11. Impact due to berthing

See MIL-HDBK-1025/1 for evaluation of lateral and longitudinal forces due to berthing.

7-12. Vibrations

Vibrations are induced in structures by reciprocating and rotating equipment, rapid application and subsequent removal of a load, or by other means. Vibrations take place in flexural, extensional, or torsional modes, or any combination of the three.

a. Resonance. Resonance occurs when the frequency of an applied dynamic load coincides with a natural frequency of the supporting structure. In this condition, vibration deflections increase progressively to dangerous proportions. Prevent resonance by insuring in the design that the natural frequency of a structure and the frequency of load application do not coincide.

b. Foundation Considerations. For the reaction of different types of soils to vibratory loading and the determination of the natural frequency of the foundation-soil system see TM 5-818-1/AFM 88-3, Chap.7; NAVFAC DM-7.01 and NAVFAC DM-7.02.

c. Collateral Reading. For further information on vibratory loading, see "Vibration Problems in Engineering and Dynamics of Framed Structures" by Timoshenko, S.

CHAPTER 8

LOADS FOR SPECIAL STRUCTURES

8-1. Crane runways, trackage, and supports

Load criteria for crane runways, trackage, and supports are discussed in ANSI MH 27.1, ASME B30.2, ASME B30.11, ASME B30.17, and Crane Manufacturers Association of America (CMAA) No. 70 and No. 74.

8-2. Waterfront structures

Load criteria for piers, wharves, and waterfront structures are discussed in detail in MIL-HDBK-1025/1, MIL-HDBK-1025/4, NAVFAC DM-25.05, and MIL-HDBK-1025/6.

8-3. Antenna supports and transmission line structures

Consider the following loads in the design of antenna supports and transmission line structures:

a. Dead Load.

b. Live Load on Stairways and Walkways.

c. Wind Load.

d. Ice Load. Use figure 8-1 to determine the thickness of ice covering on guys, conductors insulation, and framing supports. Consult cognizant field agencies for determining the ice load in locations that may have severe icing conditions, such as coastal and waterfront areas that are subject to heavy sea spray or high local precipitation, or mountainous areas that are subject to in-cloud icing.

e. Thermal Changes. Consider changes in guy or cable sag or both due to temperature changes.

f. Pretension Forces. Consider pretension forces in guys and wires as per MIL-HDBK-1002/3.

g. Broken Wires. Design support structures to resist the dynamic effects and unbalanced pull or torsion resulting from a broken guy. Support structures should also be designed to survive broken transmission wires.

h. Erection Loads. Temporary erection loads are important in the design of antenna supports and transmission line structures. See the Electronic Industries Association (EIA) publication EIA-222-D for further information on load criteria for steel antenna towers and antenna supporting structures. For further information on design loads on transmission lines, refer to the American Society of Civil Engineers (ASCE) publication "Guidelines for Transmission Line Structural Loading".

8-4. Tension fabric structures

Design criteria written specifically for tension fabric structures does not exist, at present. Due to the complicated geometry of tension fabric structures, engineering judgment must be used in determining the design wind and snow loadings on these type structures. ASCE 7 criteria on wind and snow loadings may be used only if the geometry is similar to that covered in the criteria. Refer to the National Building Code of Canada for further information on load criteria for geometrical shapes not covered in ASCE 7. Furthermore, wind-tunnel tests, as discussed in ASCE 7, may be used in determining the design wind or snow loadings on unusual geometric shapes. As discussed earlier, the initial tension in tension fabric structures may slacken with time. This effect must be considered in design.

8-5. Turbine generator foundations

Consider the following loads in design of turbine generator foundations.

a. Vertical Loads. For component weights of the turbine generator and distribution of these weights, refer to the manufacturer's machine outline drawings. Increase machine loads 25 percent for impact for machines with speeds up to and including 1,800 revolutions per minute (rpm) and 50 percent for those with higher speeds. Consider additional loads (such as auxiliary equipment, pipes, and valves) supported by the foundations.

b. Steam Condenser Load. Determine the condenser or vacuum load from the method of mounting the condenser.

c. Torque Loads. Torque loads are produced by magnetic reactions of electric motors and generators which tend to retard rotation. Use five times the normal torque in the design of the supporting members. For turbine generators, normal torque may be computed by the following equation:

$$\text{Torque (ft lb)} = 7{,}040\ (\text{kw}) / \text{rpm} \qquad \text{(eq 8-1)}$$

d. Horizontal Loads on Support Framing.

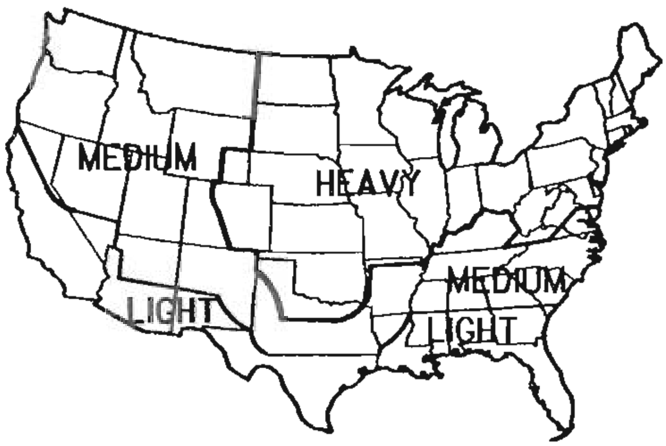
(1) Longitudinal Force. Assume a longitudinal force of 20 to 50 percent of the machine weight applied at the shaft centerline.

(2) Transverse Force. Assume a transverse force at each bent of 20 to 50 percent of the machine weight supported by the bent and applied at the machine centerline.

(3) Longitudinal and Transverse Forces. Do not assume longitudinal and transverse forces act simultaneously.

e. Horizontal Forces Within Structure. Assume horizontal forces to be equal in magnitude to the vertical loads of the generator stator and turbine exhaust hood as given on the manufacturer's machine outline drawings. Apply these forces at the top flange of the supporting girders; assume the forces to be equal and opposite.

f. External Piping. Make provisions to withstand loads from pipe thrusts, relief valves, and the weight of piping and fittings.



(a) GEOGRAPHIC DISTRIBUTION

LOADING DISTRICT	RADIAL THICKNESS OF ICE (In.)
HEAVY	0.50
MEDIUM	0.25
LIGHT	NONE

(b) THICKNESS OF ICE COVERING

Figure 8-1. Ice load on antenna support and transmission line structures.

APPENDIX A
REFERENCES

Government Publications

Department of Defense

MIL-HDBK-1002/3	Steel Structures
MIL-HDBK-1025/1	Piers and Wharves
MIL-HDBK-1025/4	Seawalls, Bulkheads, and Quaywalls
MIL-HDBK-1025/6	General Criteria for Waterfront Construction

Departments of the Army, Navy and Air Force

TM 5-809-3/ AFM 88-3, Chap. 3	Masonry Structural Design for Buildings
TM 5-809-10/ AFM 88-3, Chap. 13	Seismic Design for Buildings
TM 5-809-10-1/ AFM 88-3, Chap.13, Sec.A	Seismic Design Guidelines for Essential Buildings
TM 5-818-1/ AFM 88-3, Chap. 7	Soils and Geology: Procedures for for Foundation Designs of Buildings and Other Structures
TM 5-852-6	Arctic and Sub-Arctic Construction Calculation Methods for Determination of Depth of Freeze and Thaw in Soils
TM 5-855-1	Fundamentals of Protective Design for Conventional Weapons
TM 5-858 Series	Designing Facilities to Resist Nuclear Weapons Effects
TM 5-1300/ AFM 88-22	Structures to Resist the Effects of Accidental Explosions
NAVFAC DM-7.01	Soil Mechanics
NAVFAC DM-7.02	Foundations and Earth Structures
NAVFAC DM-25.05	Ferry Terminals and Small Craft Berthing Facilities
NAVFAC DM-38.01	Weight-Handling Equipment

Nongovernment Publications

American Association of State Highway and Transportation Officials (AASHTO), 444 North Capitol Street NW, Washington, DC 20001

	Standard Specifications for Highway Bridges (1989)

American Concrete Institute (ACI), Box 19150, Redford Station, Detroit, Michigan 48219

ACI 318-89	Building Code Requirements for Reinforced Concrete (1989)

American National Standards Institute (ANSI), 1430 Broadway, New York, NY 10018

ANSI MH 27.1	Specifications for Underhung Cranes and Monrail Systems (1981)

American Institute of Timber Construction (AITC), 333 West Hampton, Englewood, Colorado 80110

	Timber Construction Manual (1985)

American Society of Civil Engineers (ASCE), 345 East 47th Street, New York, NY 10017

ASCE 7-88	Minimum Design Loads for Buildings and Other Structures (1990)
	ASCE Publication "Guidelines for Transmission Line Structural Loading" (1984)

American Society of Mechanical Engineers (ASME), 345 East 47th Street, New York, NY 10017

ASME B30.2	Overhead and Gantry Cranes (Top Running Bridge, Single, or Multiple Girder, Top Running Trolley Hoist) (1990)
ASME B30.11	Monorails and Underhung Cranes (1988)
ASME B30.17	Overhead and Gantry Cranes (Top Running Bridge, Single Girder, Underhung Hoist) (1985)

American Railway Engineering Association (AREA), 2000 L Street NW., Washington, DC 20036

	Manual for Railway Engineering, Volumes I and II (1989)

Electronic Industries Association (EIA), 2001 Eye Street NW., Washington, DC 20006

	Structural Standards for Steel Antenna Towers and Antenna Supporting Structures (1986)

Metal Building Manufacturers Association (MBMA), 1230 Keith Building, Cleveland, Ohio 44115

	Low Rise Building Systems Manual (1986, with 1990 supplement)

National Research Council of Canada, Ottawa, Ontario, Canada

National Building Code of Canada (1990)

"Building Foundation Design Handbook," K. Labs, J. Carmody, R. Sterling, L. Shen, Y. Huang, D. Parker, Oak Ridge National Lab Report ORNL/Sub/86-72143/1 (May 1988)

"Standard Handbook for Mechanical Engineers," McGraw-Hill Book Co., New York, New York 8th Edition, 1978.

"Vibration Problems in Engineering", S. Timoshenko, D. Van Nostrand Co., Inc., New York, New York 10020 4th Edition, 1974.

Location	Ground Snow Load[b] (psf)	Frost Penetration[a] (in)	Basic Wind Speed[b] (mph)
KENTUCKY			
Fort Campbell	15	22	70
Fort Knox	15	32	70
Lexington	15	32	70
Louisville	15	32	70
LOUISIANA			
Barksdale AFB	5	7	70
Fort Polk	5	0	80
Lake Charles	0	0	95
Louisiana AAP	5	7	70
New Orleans	0	0	100
Shreveport	5	7	70
MAINE			
Bangor	80	98	90
Brunswick	60	86	85
Loring AFB	100	133	80
Portland	60	86	85
Winter Harbor	60	86	90
MARYLAND			
Aberdeen Proving Gd	20	29	70
Andrews AFB	20	26	70
Annapolis	20	26	70
Baltimore	20	29	70
Fort Detrick	35	29	70
Fort Meade	20	26	70
Fort Ritchie	35	32	70
Lexington Park	20	22	70
MASSACHUSETTS			
Boston	30	49	85
Fort Devens	45	64	80
L.G. Hanscom Field	40	54	85
Otis AFB	30	38	90
Springfield	30	64	70
Westover AFB	30	64	75
MICHIGAN			
Detroit	20	61	75
Kincheloe AFB	70	102	80
K.I. Sawyer AFB	60	102	80
Selfridge AFB	20	59	75
Wurtsmith AFB	50	84	75

Location	Ground Snow Load[b] (psf)	Frost Penetration[a] (in)	Basic Wind Speed[a] (mph)
MINNESOTA			
Duluth	65	140	75
Minneapolis	50	125	80
MISSISSIPPI			
Biloxi	0	0	100
Columbus AFB	10	7	70
Jackson	5	5	75
Keesler AFB	0	0	100
Gulfport	0	5	110
Meridian	5	5	70
Mississippi AAP	0	0	100
MISSOURI			
Fort Leonard Wood	15	36	70
Kansas City	20	49	75
Lake City AAP	20	49	75
Richards Gebaur AFB	20	49	75
St. Louis	20	38	70
Whiteman AFB	20	46	70
MONTANA			
Helena	20	107	75
Malmstrom AFB	20	107	80
Missoula	25	77	70
NEBRASKA			
Cornhusker AAP	25	64	85
Hastings	25	64	85
Lincoln	25	64	85
Offutt AFB	25	73	80
Omaha	25	75	80
NEVADA			
Carson City	25	23	75
Fallon	10	23	75
Hawthorne	15	23	75
Las Vegas	5	0	75
Reno	20	23	80
Stead AFB	15	23	80
NEW HAMPSHIRE			
Hanover	55	98	70
Pease AFB	50	64	80
Portsmouth	50	64	85

Location	Ground Snow Load[b] (psf)	Frost Penetration[a] (in)	Basic Wind Speed [b] (mph)
NEW JERSEY			
Atlantic City	15	18	90
Bayonne	20	38	80
Cape May	15	20	100
Fort Monmouth	25	38	80
McGuire AFB	20	29	80
Picatinny Arsenal	35	32	75
NEW MEXICO			
Albuquerque	5	18	80
Cannon AFB	10	18	80
Holloman AFB	5	4	80
Kirtland AFB	10	18	80
Sacramento PK	20	Not Available	90
White Sands MR	5	4	80
NEW YORK			
Albany	30	82	70
Buffalo	40	59	70
Fort Drum	60	94	70
Griffis AFB	50	86	70
New York City	20	38	80
Niagara Falls IAP	30	59	70
Plattsburg AFB	40	107	70
Stewart AFB, Newburgh	35	54	70
Syracuse	45	73	70
Watervliet	30	86	70
West Point Mil Res	35	54	70
NORTH CAROLINA			
Fort Bragg	10	0	80
Charlotte	10	4	70
Cherry Point	10	5	100
Camp Lejeune	10	0	100
Cape Hatteras	5	5	115
Greensboro	15	8	70
Pope AFB	10	0	80
Seymour Johnson	10	4	90
Sunny Point Ocean Term	10	0	100
Wilmington	10	5	115
NORTH DAKOTA			
Bismarck	30	150	80
Fargo	35	153	90
Grand Forks AFB	40	166	80
Minot AFB	35	163	75

Location	Ground Snow Load[b] (psf)	Frost Penetration[a] (in)	Basic Wind Speed[b] (mph)
VERMONT			
Bennington	50	77	70
Burlington	40	107	70
Montpelier	70	107	70
St. Albans	40	107	70
VIRGINIA			
Fort Belvoir	20	26	70
Fort Eustis	10	9	85
Fort Myer	20	26	70
Langley AFB, Hampton	10	9	90
Norfolk	10	9	90
Petersburg/Fort Lee	15	14	75
Quantico	20	22	70
Radford AAP	25	22	70
Richmond	15	18	75
Virginia Beach Coast	10	10	100
Yorktown	10	9	85
WASHINGTON			
Bremerton	20	9	75
Fairchild AFB/Spokane	40	64	70
Fort Lewis	20	9	75
Larson AFB, Moses Lake	25	52	70
McChord AFB	20	9	75
Pasco	15	49	70
Seattle	15	9	70
Tacoma	20	8	90
Walla Walla	15	49	70
Yakima	25	52	70
WASHINGTON, D.C.			
Bolling AFB	20	26	70
Fort McNair	20	26	70
Walter Reed AMC	20	26	70
WEST VIRGINIA			
Charleston	20	22	70
Sugar Grove	30	38	70
WISCONSIN			
Badger AAP	35	98	85
Fort McCoy	40	114	85
Green Bay	40	94	90
Madison	35	75	80
Milwaukee	35	75	80

Location	Ground Snow Load[b] (pfs)	Frost Penetration[a] (in)	Basic Wind Speed[b] (mph)
WISCONSIN (continued)			
Osceola	55	135	80
WYOMING			
Cheyenne	15	59	85
Yellowstone	60	125	80

[a] Frost penetration values will be used to establish minimum design depth of building foundations below finish grade. These values are based on the deepest, i.e. worst case, frost penetrations away from buildings and may be reduced for foundation design according to information in Appendix D.

[b] 50 year mean recurrence interval.

[c] Determine all snow loads based on tabulated ground snow load. However, based on local practice, the final design snow load cannot be less than 30 psf.

[d] Determine all snow loads based on tabulated ground snow load. However, based on local practice, the final design snow load cannot be less than 25 psf.

Location	Ground Snow Load[b] (psf)	Frost Penetration[a] (in)	Basic Wind Speed[b] (mph)
ATLANTIC OCEAN AREA			
Ascension Island	0	0	70
Azores			
Lajes Field	0	0	100
Bermuda	0	0	110
CARIBBEAN SEA			
Bahama Islands			
Eleuthera Island	0	0	120
Grand Bahama Isle	0	0	120
Grand Turk Island	0	0	130
Great Exuma Island	0	0	120
Cuba			
Guantanamo NAS	0	0	75
Leeward Islands			
Antigua Island	0	0	120
Puerto Rico			
Boringuen Field	0	0	Not Available
Ramey AFB and Aguada	0	0	80
San Juan	0	0	100
Sabana Seca	0	0	100
Vieques Island	0	0	120
Roosevelt Roads	0	0	120
Trinidad Island			
Port of Spain	0	0	70
Trinidad NS	0	0	70
CENTRAL AMERICA			
Canal Zone			
Albrook AFB	0	0	70
Balboa	0	0	70
Coco Solo	0	0	70
Colon	0	0	70
Cristobal	0	0	70
France AFB	0	0	70
EUROPE			
England			
Birmingham	15	12	70
London	15	12	75
Mildenhall AB	15	12	80
Plymouth	10	12	70
Sculthorpe AB	15	12	75
Southport	10	12	80
South Shields	15	12	75
Spurn Head	15	12	75

Location	Ground Snow Load[b] (psf)	Frost Penetration[a] (in)	Basic Wind Speed[a] (mph)
EUROPE (continued)			
France			
Nancy	15	18	70
Paris/LeBourget	20	18	80
Rennes	15	18	85
Vichy	25	24	100
Germany			
Bremen	25	30	70
Munich-Reim	40	36	75
Rhein-Main AB	25	30	70
Stuttgart AB	45	36	70
Greece			
Athens	5	0	70
Souda Bay	5	0	70
Iceland			
Keflavik	30	24	100
Thorshofn	30	36	120
Northern sites	Not Available	May be permafrost	130
Italy			
Aviano AB	10	18	70
Brindisi	5	6	85
La Maddalena	Not Available	Not Available	70
Sigonella-Catania	Not Available	Not Available	75
Northern Ireland			
Londonderry, Ulster	15	12	105
Scotland			
Aberdeen	15	12	70
Edinburgh	15	12	75
Edzell	15	12	70
Glasgow/Renfrew Airfield	15	12	75
Lerwick, Shetland Islands	15	18	90
Prestwick	15	12	75
Stornoway	15	12	95
Thurso	15	12	105
Spain			
Madrid	10	6	70
Rota	5	0	70
San Pablo	5	6	90
Zaragoza	10	6	90
NORTH AMERICA			
Canada			
Argentia NAS, Newfoundland	47	36	90
Churchill, Manitoba	66	Permafrost	85
Cold Lake, Alberta	41	72	70

Location	Ground Snow Load[b] (psf)	Frost Penetration[a] (in)	Basic Wind Speed[b] (mph)
NORTH AMERICA (continued)			
Edmonton, Alberta	27	60	70
E. Harmon AFB, Newfoundland	86	60	70
Fort William, Ontario	73	60	70
Frobisher, N.W.T.	50	Permafrost	85
Goose Airport, Newfoundland	100	60	70
Ottawa, Ontario	60	48	70
St. John's, Newfoundland	72	36	90
Toronto, Ontario	40	36	70
Winnipeg, Manitoba	45	60	70
Greenland			
Narsarssuak AB	30	60	110
Simiutak AB	25	60	135
Sondrestrom AB	20	Permafrost	95
Thule AB	25	Permafrost	115
PACIFIC OCEAN AREA			
Australia			
H.E. Holt, NW Cape	0	0	110
Caroline Islands			
Koror, Palau Islands	0	0	80
Ponape	0	0	90
Johnston Island	0	0	95
Kwajalein Island	0	0	100
Mariana Islands			
Agana, Guam	0	0	135
Andersen AFB, Guam	0	0	135
Saipan	0	0	130
Tinian	0	0	130
Marcus Island	0	0	130
Midway Island	0	0	70
Okinawa			
Kadena AB	0	0	165
Naha AB	0	0	165
Philippine Islands			
Clark AFB	0	0	90
Sangley Point	0	0	90

Location	Ground Snow Load[b] (psf)	Frost Penetration[a] (in)	Basic Wind Speed[b] (mph)
PACIFIC OCEAN AREA (continued)			
Subic Bay	0	0	90
Samoa Islands			
Apia, Upolu Island	0	0	125
Tutuila, Tutuila Island	0	0	125
Volcano Islands			
Iwo Jima AB	0	0	185
Wake Island	0	0	70

[a] Frost penetration values will be used to establish minimum design depth of building foundations below finish grade. These values are based on the deepest, i.e. worst case, frost penetrations away from buildings and may be reduced for foundation design according to information in Appendix D.

[b] 50 year mean recurrence interval.

APPENDIX D

FROST PENETRATION

D-1. Frost penetration

The depth to which frost penetrates at a site depends on the climate, the type of soil, the moisture in the soil and the surface cover (e.g., pavement kept clear of snow vs. snow-covered turf). If the supporting soil is warmed by heat from a building, frost penetration is reduced considerably. The values in appendices B and C represent the depth of frost penetration to be expected if the ground is bare of vegetation and snow cover, the soil is non-frost susceptible (NFS), well-drained (i.e., dry) sand or gravel, and no building heat is available. Thus, these values represent the deepest (i.e., worst case) frost penetration expected in each area. Most building foundations can be at a shallower depth without suffering frost action. (However, other considerations besides frost penetration may affect foundation depth, such as erosion potential or moisture desiccation). For interior footings, which under service conditions are not normally susceptible to frost, the potential effects of frost heave during construction should be considered. Design values for heated and unheated buildings may be obtained by reducing the values in appendices B and C according to figure D-1. For buildings heated only infrequently, the curve in figure D-1 for unheated buildings should be used. The curves

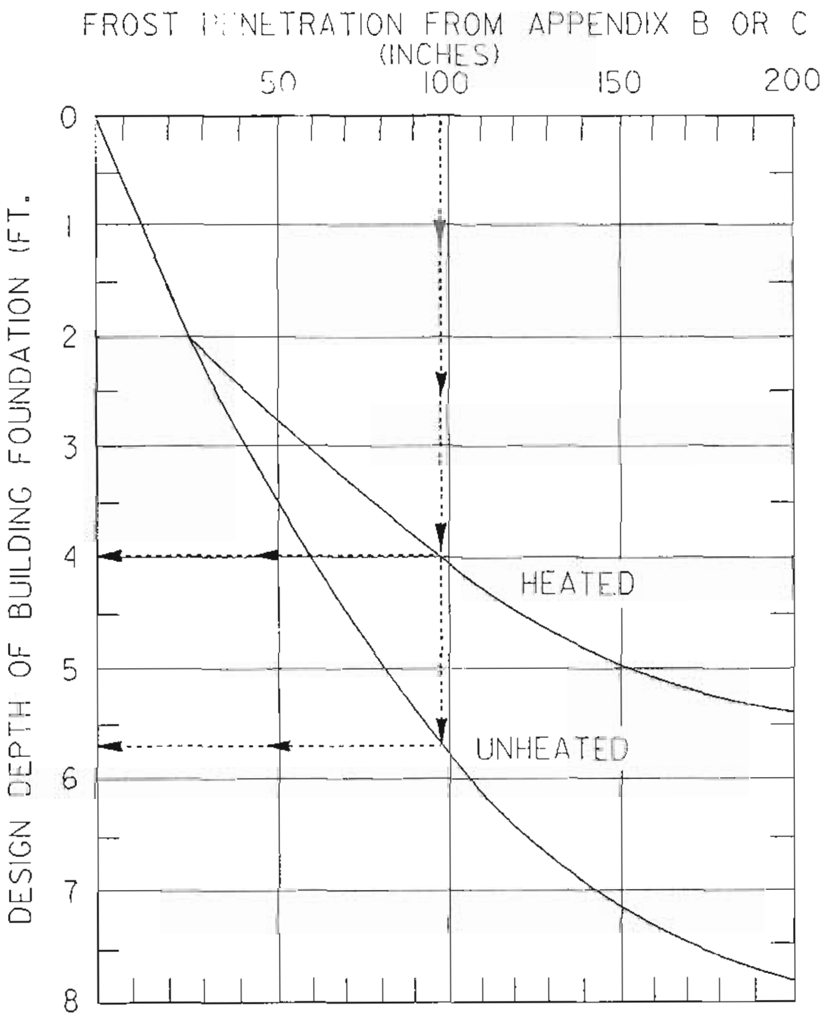


Figure D-1. Design depth of building foundation.

in figure D-l were established with an appreciation for the variability of soil and the understanding that some portions of the building may abut snow-covered turf while other portions abut paved areas kept clear of snow.

D-2. Example

What minimum depth is needed for footings of a hospital and an unheated vehicle storage building to be built in Bangor, Maine, to protect them from frost action? Solution: The tabulated frost penetration value for Bangor, Maine, is 98 inches (appendix B). Using the "heated" curve in figure D-l, footings for the hospital should be located 4 feet below the surface to protect them from frost action. Using the "unheated" curve, footings for the unheated garage should be located 6 feet below the surface.

D-3. Additional information

Additional information on which more refined estimates of frost penetration can be made, based on site-specific climatic information, the type of ground cover and soil conditions is contained in TM 5-852-6.

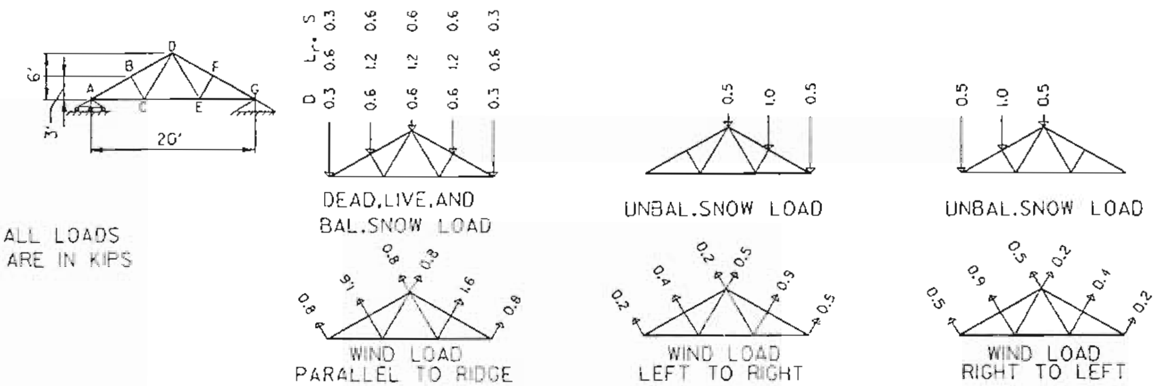
D-4. Frost protection

Foundations should be placed at or below the depths calculated above. The foundation may be placed at a shallower depth than calculated above if protected from frost action by insulation on the cold side. For more information on foundation insulation, see "Building Foundation Design Handbook" by Oak Ridge National Laboratory.

APPENDIX E

DESIGN EXAMPLES FOR LOAD COMBINATIONS

GIVEN: THE ROOF TRUSS SHOWN BELOW IS FOR A BUILDING LOCATED IN A ZONE WHERE THERE IS NO EARTHQUAKE. EIGHT LOADING CONDITIONS ARE INCLUDED.



PROBLEM: WHAT CRITICAL LOAD COMBINATION(S) PROVIDE THE MAXIMUM STRESSES IN MEMBERS AB AND BC?

SOLUTION:

ROOF LOADS AND CRITICAL LOAD COMBINATIONS-KIPS

MEMBER	DEAD LOAD		LIVE LOAD		SNOW LOAD BALANCED		SNOW LOAD UNBALANCED RIGHT SLOPE		SNOW LOAD UNBALANCED LEFT SLOPE		WIND LOAD PARALLEL W/RIDGE	WIND LOAD LT TO RT	WIND LOAD RT TO LT	MAX. FORCE (KIPS)	CRITICAL LOAD COMBINATION (ANSWER)
	1.00D	0.75D	$1.00L_r$	$0.75L_r$	1.00S	0.75S	1.00S	0.75S	1.00S	0.75S	1.00W	1.00W	1.00W		
AB	-1.8	-1.3	-3.6	-2.7	-1.8	-1.3	-1.0	-0.8	-1.9	-1.4	+4.0	+1.5	+1.9	-5.4 +2.2	$1.00D+1.00L_r$ 1.00D+1.00W
BC	-0.5	-0.4	-1.0	-0.7	-0.5	-0.4	0	0	-0.9	-0.6	+1.6	+0.4	+0.9	-1.5 +1.1	$1.00D+1.00L_r$ 1.00D+1.00W

LOAD COMBINATIONS-KIPS

COMBINATION * / MEMBER	AB	BC
1.00D	-1.8	-0.5
$1.00D+1.00L_r$	-5.4	-1.5
1.00D+1.00S	-3.7	-1.4
1.00D+1.00W	-0.3 +2.2	-0.1 +1.1
$0.75D+0.75L_r+0.75W$	-2.9	-0.8 +0.1
0.75D+0.75S+0.75W	-1.6 -0.9	-0.7 +0.8

SELECT MAX COMPRESSION FORCE

SELECT MAX TENSION FORCE

*CONFORMS WITH ANSI/ASCE 7-88 PARA. 2.3

SYMBOLS

D-DEAD LOAD

L_r-ROOF LIVE LOAD

S-SNOW LOAD

W-WIND LOAD

Figure E-1. Design example for load combinations.

APPENDIX F

DESIGN EXAMPLES FOR LIVE LOADS

F-1. Purpose and scope

This appendix contains illustrative examples using the live load criteria given in ASCE 7-88.

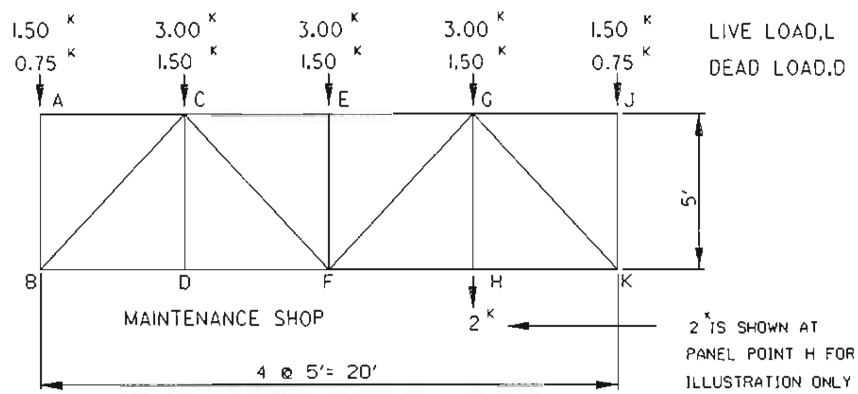
F-2. Abbreviations

The following abbreviations are used in the example problems:

a. Eq. - Equation
b. Para. - Paragraph

GIVEN: ACCESSIBLE ROOF TRUSS WITH DEAD AND LIVE LOADS SHOWN BELOW.

ASSUME THE TRUSS WILL CARRY A CONCENTRATED LOAD OF 2000 LBS AT ANY OF THE PANEL POINTS IN THE LOWER CHORD CONSISTENT WITH PARAGRAPH 4.3.1 IN ASCE 7-88..



EXPOSED ROOF TRUSS

PROBLEM: DETERMINE THE MAXIMUM FORCES FOR ONE POSSIBLE LOAD COMBINATION ON THE EXPOSED ROOF TRUSS.

SOLUTION:

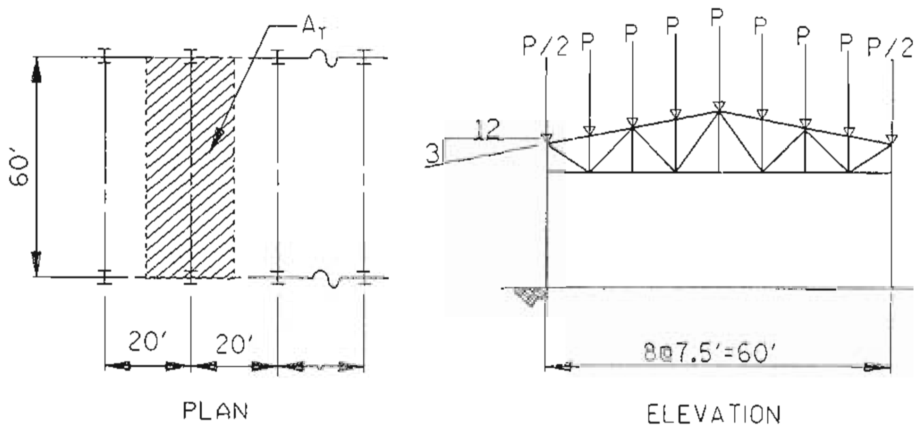
FORCE-KIPS							
MEMBER	D	L	2K@EITHER PANEL POINT *				MAX FORCE
			D	F	H		
AB	-0.75	-1.50	0	0	0		-2.25
CD	0	0	+2.00	0	0		+2.00
EF	-1.50	-3.00	0	0	0		-4.50
AC	0	0	0	0	0		0
CE	-3.00	-6.00	-1.00	-2.00	-1.00		-11.00
BD	+2.25	+4.50	+1.50	+1.00	+0.50		+8.25
DF	+2.25	+4.50	+1.50	+1.00	+0.50		+8.25
BC	-3.18	-6.36	-2.12	-1.41	-0.71		-11.66
CF	+1.06	+2.12	-0.71	+1.41	+0.71		+4.59

* FOR EACH MEMBER SELECT ONE FORCE ONLY FROM EITHER COLUMN D,F OR H AND COMBINE WITH (D+L) TO OBTAIN MAXIMUM FORCE.

Figure F-1. Design example for live loads - accessible roof truss.

GIVEN: INTERIOR ROOF TRUSS SHOWN BELOW.

PROBLEM: DETERMINE THE MINIMUM ROOF LIVE LOAD ON A PANEL POINT.



INTERIOR TRUSS

SOLUTION: REDUCED LIVE LOAD, L_r

$L_r = 20R_1 R_2 \geq 12$ — E0.2*

WHERE $A_t = 20 \times 60 = 1200$ FT^2

SINCE $A_t > 600$ FT^2 — PARA.4-11*

$R_1 = 0.6$

$F = 3$

SINCE $F < 4$ — PARA.4-11*

$R_2 = 1.0$

$L_r = 20 \times 0.6 \times 1.0 = 12$ PSF — E0.2*

LOAD ON PANEL POINT

$P = 12 \times 20 \times 7.5 = 1800$ LBS SAY 1.8^K

* REFERENCE: ASCE 7-88

Figure F-2. Design example for live loads - roof live load.

GIVEN: 25 FT. CRANE RUNWAY GIRDER SUPPORTING A 30 TON CAPACITY BRIDGE CRANE SHOWN BELOW.

PROBLEM: FIND THE DESIGN LOADS FOR THE RUNWAY GIRDER.

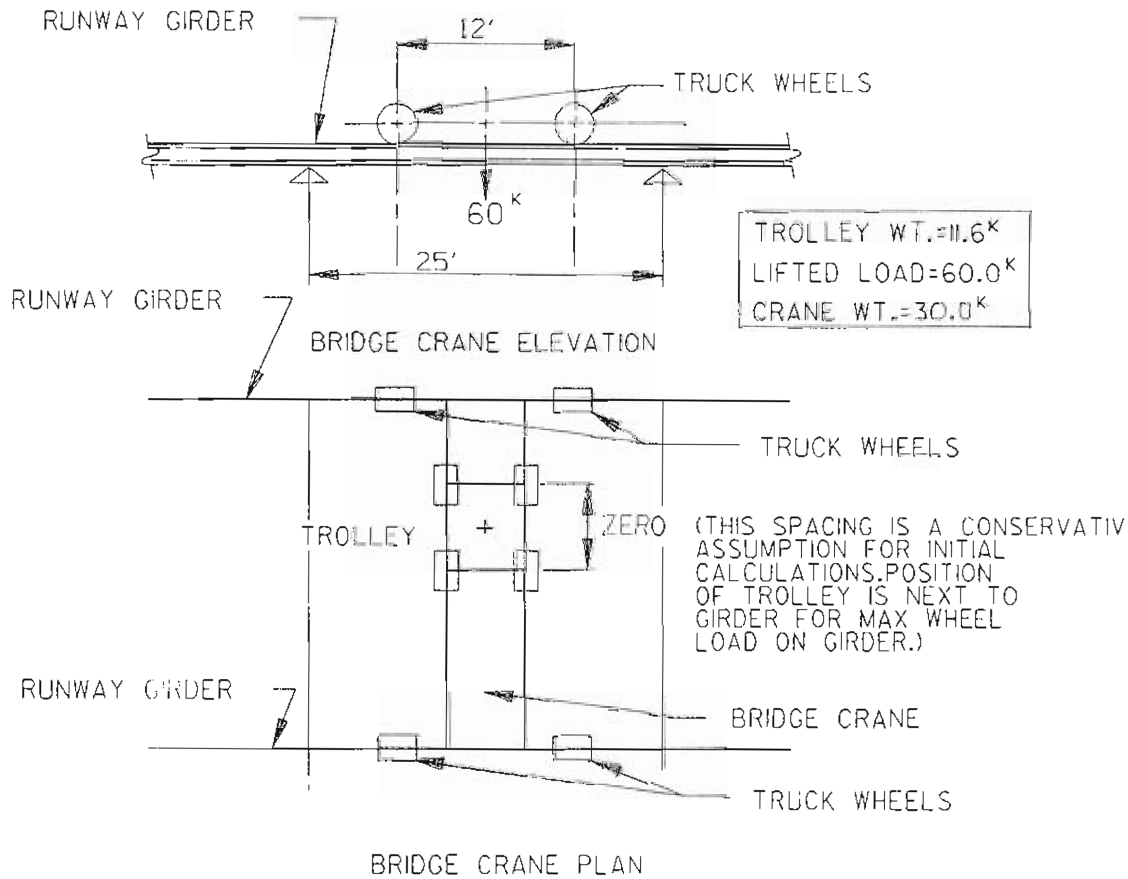


Figure F-3. Design example for live loads - crane runway. (Sheet 1 of 2)

SOLUTION:

VERTICAL LOAD FROM EACH TRUCK WHEEL — PARA. 4.7.3•

$1/2(60 + 11.6 + 1/2 \times 30) = 43.3^K$

25 PERCENT IMPACT $= 10.8^K$

54.1^K

LATERAL LOAD FROM EACH TRUCK WHEEL — PARA. 4.7.3•

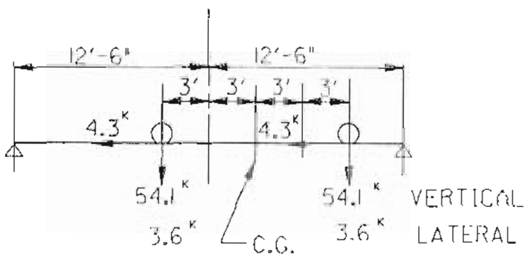
$1/4 \times 0.20(60 + 11.6) = 3.6^K$

LONG. LOAD FROM EACH TRUCK WHEEL — PARA. 4.7.3•

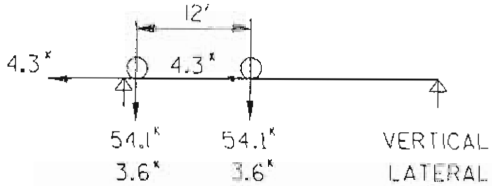
$1/2 \times 0.10(60 + 11.6 + 1/2 \times 30)$

$= 4.3^K$

• REFERENCE: ASCE 7-88



WHEEL LOAD LOCATION FOR MAXIMUM MOMENT ✳



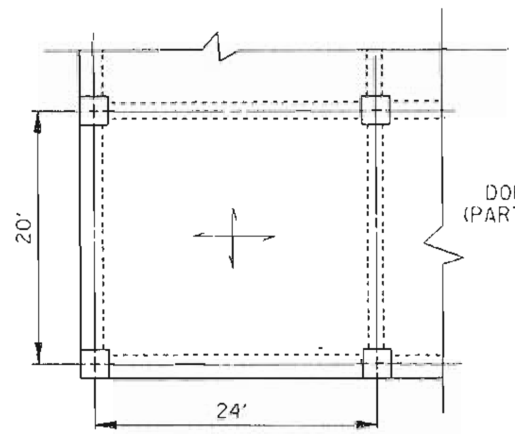
WHEEL LOAD LOCATION FOR MAXIMUM SHEAR ✳

✳ ALL LOADS APPLIED AT TOP OF CRANE RAIL.

Figure F-3. Design example for live loads - crane runway. (Sheet 2 of 2)

GIVEN: PARTIAL SECOND FLOOR FRAMING PLAN OF A TWO STORY DORMITORY IS SHOWN BELOW.

PROBLEM: DETERMINE THE REDUCED UNIFORM DESIGN LIVE LOAD.



SECOND FLOOR FRAMING PLAN

SOLUTION: REDUCED LIVE LOAD

$L=L_0(0.25+15/\sqrt{A_I})$ EQ.1*

WHERE $L_0=40$ PSF TABLE C3*

$A_I=1\times24\times20=480$ FT² PARA.4.8.1*

$L=40(0.25+15/\sqrt{480})=37.4$ PSF EQ.1*

CHECK

$L\geq0.50L_0$ PARA.4.8.1*

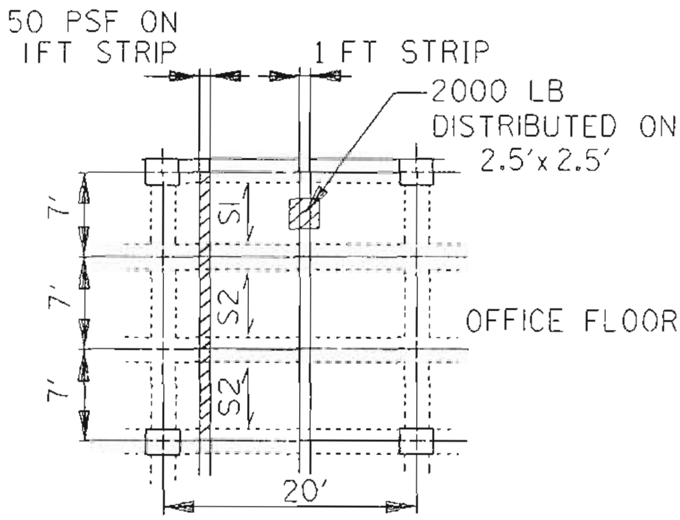
37.4 PSF>(0.5 x 40=20 PSF) O.K.

* REFERENCE: ASCE 7-88

Figure F-4. Design example for live loads - two-way concrete floor slab.

GIVEN: PARTIAL FLOOR FRAMING PLAN OF AN OFFICE IS SHOWN BELOW.

PROBLEM: DETERMINE THE UNFACTORED UNIFORM AND CONCENTRATED LIVE LOADS



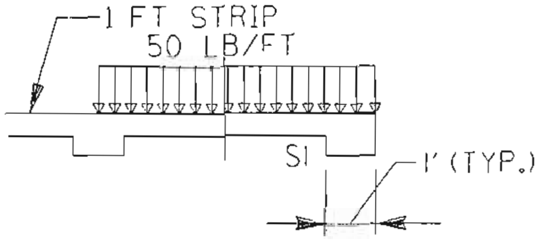
PARTIAL FLOOR FRAMING PLAN

Figure F-5. Design example for live loads - one-way concrete floor slab. (Sheet 1 of 2)

SOLUTION:

UNIFORM LIVE LOAD

50 LB/FT ON 1FT STRIP — TABLE 2*



POSITION FOR
MAXIMUM POSITIVE MOMENT

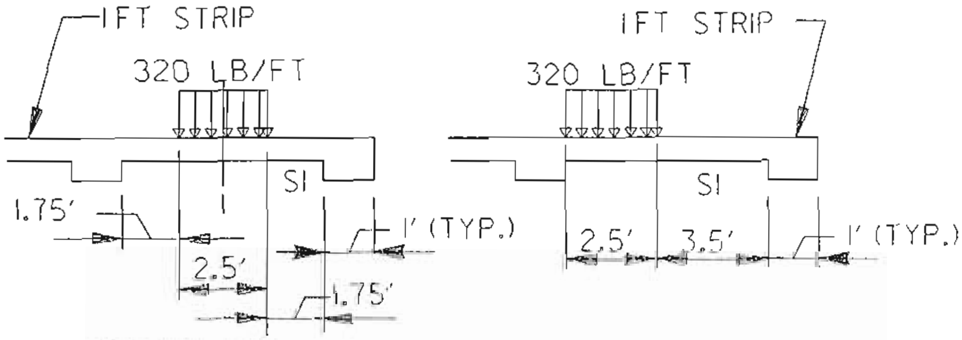
CONCENTRATED LIVE LOAD

2000 LB — TABLE 3*

DISTRIBUTED OVER 2.5'x2.5' — PARA.4.3*

2000/(2.5 x 2.5)=320 PSF OR 320 LB/FT

ON A ONE FT STRIP



POSITION FOR
MAXIMUM POSITIVE MOMENT

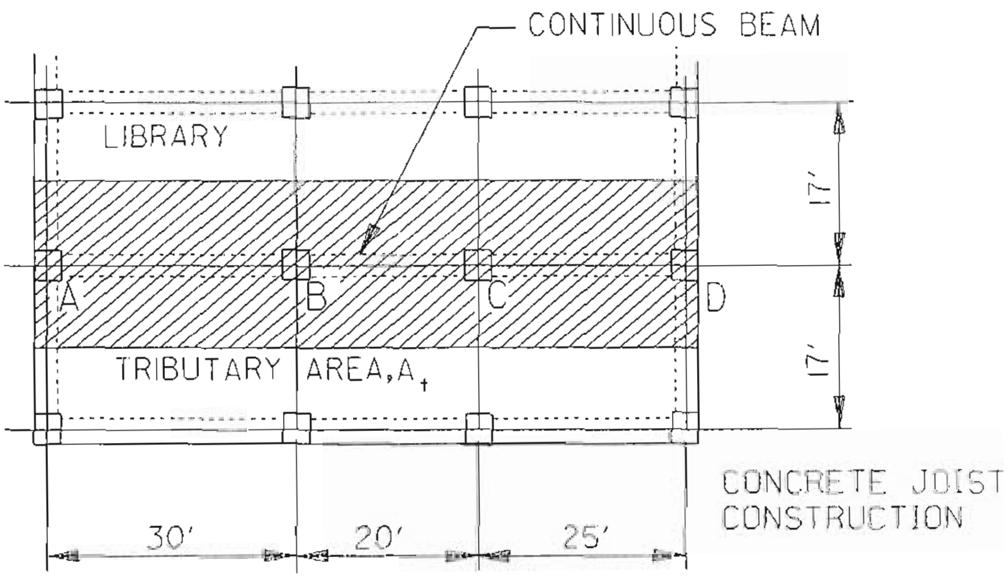
POSITION FOR
MAXIMUM SHEAR

*REFERENCE: ASCE 7-88

Figure F-5. Design example for live loads - one-way concrete floor slab. (Sheet 2 of 2)

GIVEN: A PARTIAL FLOOR FRAMING PLAN OF A LIBRARY READING ROOM IS SHOWN BELOW.

PROBLEM: DETERMINE (A)REDUCED LIVE LOAD,L,AND (B)LOADING FOR THE MAXIMUM AND MINIMUM LIVE LOAD MOMENT AT MIDPOINT OF SPAN BC AND THE MAXIMUM NEGATIVE MOMENT AT SUPPORT B OF THE CONTINUOUS BEAM.



PARTIAL FLOOR FRAMING PLAN

Figure F-6. Design example for live loads - continuous beam. (Sheet 1 of 3)

SOLUTION:

A.REDUCED LIVE LOAD

$L=L_0(0.25+15/\sqrt{A_I})$ EQ.1

WHERE $L_0=60$ PSF TABLE 2

BEAM AB $A_I=2\times$ TRIBUTARY AREA PARA.4.8.1

$=2\times 30\times 17=1020\ FT^2$

BEAM BC $A_I=2\times 20\times 17=680\ FT^2$ PARA.4.8.1

BEAM CD $A_I=2\times 25\times 17=850\ FT^2$ PARA.4.8.1

$L_{AB}=60(0.25+15/\sqrt{1020})=43.2$ PSF EQ.1

$L_{BC}=60(0.25+15/\sqrt{680})=49.5$ PSF EQ.1

$L_{CD}=60(0.25+15/\sqrt{850})=45.9$ PSF EQ.1

CHECK

$L\geq 0.50L_0$

$43.2>(0.50\times 60=30$ PSF) O.K. PARA.4.8.1

$W_{AB}=43.2\times 17=734$ LB/FT

$W_{BC}=49.5\times 17=842$ LB/FT

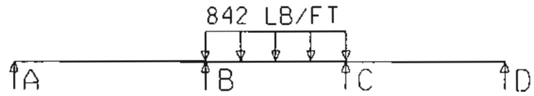
$W_{CD}=45.9\times 17=780$ LB/FT

* ALL REFERENCES IN THIS EXAMPLE ARE TO ASCE 7-88

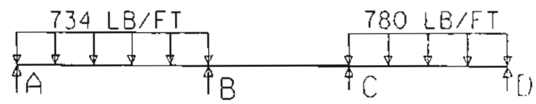
Figure F-6. Design example live loads - continuous beam. (Sheet 2 of 3)

B.LOADING

REMOVE LIVE LOAD FROM SELECTED SPANS TO PRODUCE UNFAVORABLE EFFECT. PARA.4-6

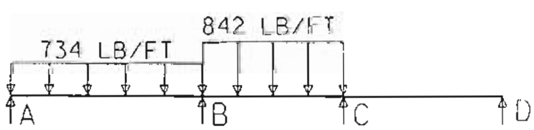


LOADING FOR MAXIMUM MOMENT @ MIDPOINT OF SPAN BC



LOADING FOR MINIMUM MOMENT @ MIDPOINT OF SPAN BC *

* NOTE: THIS IS ALSO THE LOADING FOR THE MAXIMUM POSITIVE MOMENTS IN SPANS AB AND CD.

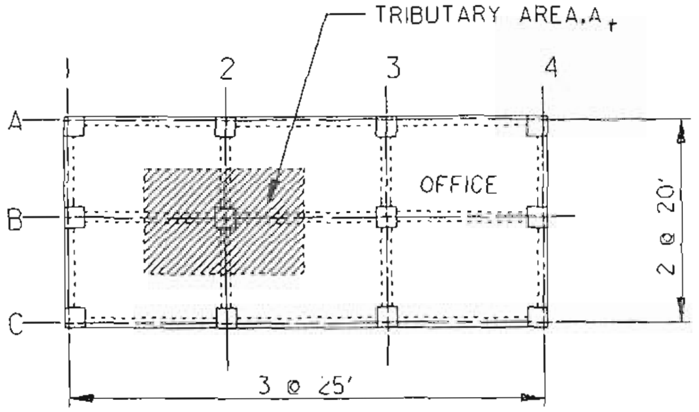


LOADING FOR MAXIMUM NEGATIVE MOMENT @ SUPPORT B

Figure F-6. Design example live loads - continuous beam. (sheet 3 of 3)

GIVEN: TYPICAL FLOOR FRAMING PLAN OF A THREE STORY ADMINISTRATIVE BUILDING SHOWN BELOW.

PROBLEM: DETERMINE THE FLOOR LIVE LOAD ON COLUMN B2 LOCATED ON THE FIRST FLOOR.



TYPICAL FLOOR FRAMING PLAN

SOLUTION:

REDUCED LIVE LOAD

$L = L_0(0.25 + 15/\sqrt{A_I})$ — EQ.1*

WHERE $L_0 = 50$ PSF — TABLE 2*

$A_I = 4A_t$ FOR 2ND AND 3RD FLOOR — PARA.4.8.1*

$A_I = 4(2 \times 20 \times 25)$

$A_I = 4000$ SQ FT

$L = 50(0.25 + 15/\sqrt{4000}) = 24.4$ PSF

CHECK

$L \geq 0.4L_0$ — PARA.4.8.1*

24.4 PSF>(0.4 x 50=20 PSF) O.K.

FLOOR LIVE LOAD ON COLUMN B2

$24.4(2 \times 20 \times 25) = 24.4^k$

* REFERENCE ASCE 7-88

Figure F-7. Design example for live loads - column.

APPENDIX G

DESIGN EXAMPLES FOR WIND LOADS

G-1. Purpose and scope

This appendix contains illustrative examples using the wind load criteria given in ASCE 7-88.

G-2. Abbreviations

The following abbreviations arc used in the example problems:

a. Eq. - Equation
b. Para. - Paragraph
c. Fig. - Figure
d. Tab. - Table
e. U.N.O. - Unless noted otherwise.

GIVEN: ONE STORY INDUSTRIAL BUILDING SHOWN BELOW.

LOCATION:HUNTSVILLE.AL
WIND EXPOSURE CATEGORY C

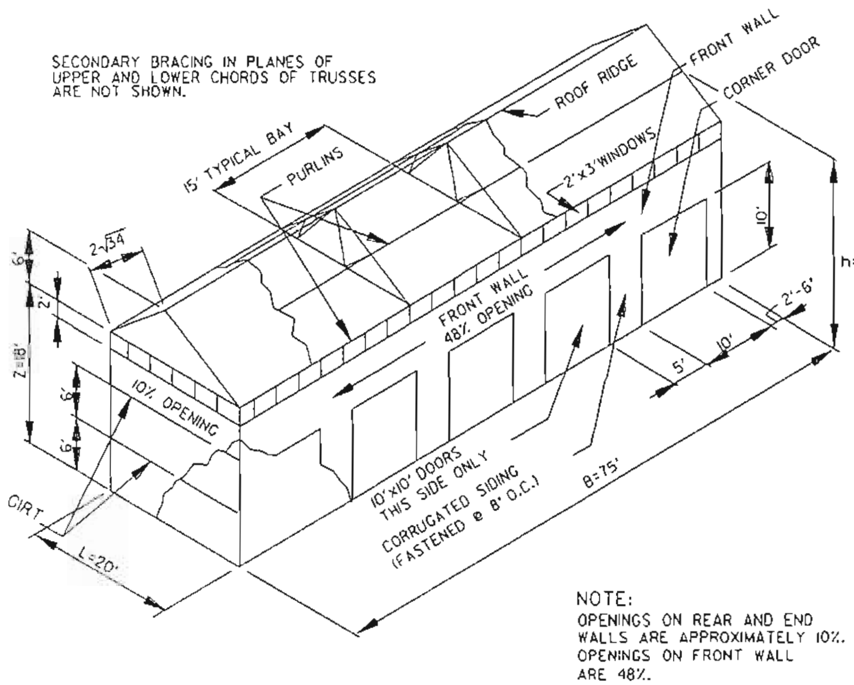


Figure G-1. Design example for wind loads - industrial building. (Sheet 1 of 11)

PROBLEM:

DETERMINE THE FOLLOWING RESULTING FROM WIND.

A. EXTERNAL PRESSURE ON THE BUILDING.
B. SHEAR FORCES ON WALLS.
C. MAXIMUM PRESSURE ON ROOF TRUSS.*
D. PRESSURE ON DOOR.
E. LOAD ON GIRT.
F. MAXIMUM TENSION ON WALL FASTENER.

* ASSUME DOORS ON FRONT WALL ARE OPEN IN DETERMINING THE MAXIMUM WIND PRESSURE ON THE ROOF TRUSS. SEE COMMENTARY IN ASCE 7-88 FOR DEFINITION OF OPENINGS.

Figure G-1. Design example for wind loads - industrial building (Sheet 2 of 11)

SOLUTION:

A. EXTERNAL WIND PRESSURE ON THE BUILDING

(1) $p = qG_hC_p - q_h(GC_{pi})$ — TABLE 4*

NOTE: NEGLECT INTERNAL PRESSURE TERM $-q_h(GC_{pi})$ WHEN ONLY EXTERNAL PRESSURES ARE CONSIDERED.

$q_z = 0.00256K_z(IV)^2$ — EQ.3

WHERE $K_z = 0.84$ AT Z=18′ — TABLE 6

$K_h = 0.88$ AT h=21′ — TABLE 6

I=1.00 — TABLE 6

V=70 MPH — APPENDIX B (THIS MANUAL)

$q_z = 0.00256 \times 0.84(1.0 \times 70)^2 = 10.5$ PSF

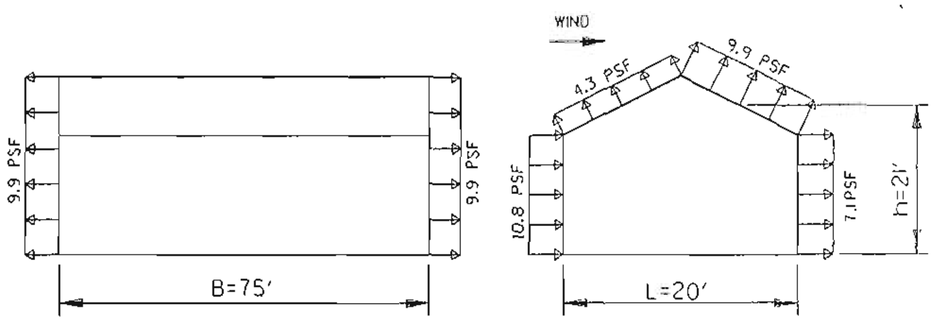
$q_h = 0.00256 \times 0.88(1.0 \times 70)^2 = 11.0$ PSF

$G_h = 1.29$ — TABLE 8

(2) WIND NORMAL TO RIDGE

ELEMENT	Cp	p=qGh Cp,PSF	CONDITION	
WINDWARD WALL	0.8	10.5x1.29x0.8=+10.8		
LEEWARD WALL	-0.5	11.0x1.29x(-0.5)=-7.1	L/B=20/75=0.3	FIG.2
WINDWARD ROOF	-0.3	11.0x1.29x(-0.3)=-4.3	h/L=21/20=1.1	FIG.2
LEEWARD ROOF	-0.7	11.0x1.29x(-0.7)=-9.9	θ=31°	
SIDE WALL	-0.7	11.0x1.29x(-0.7)=-9.9		

*NOTE: ALL REFERENCES IN THIS EXAMPLE ARE TO ASCE 7-88 U.N.O.



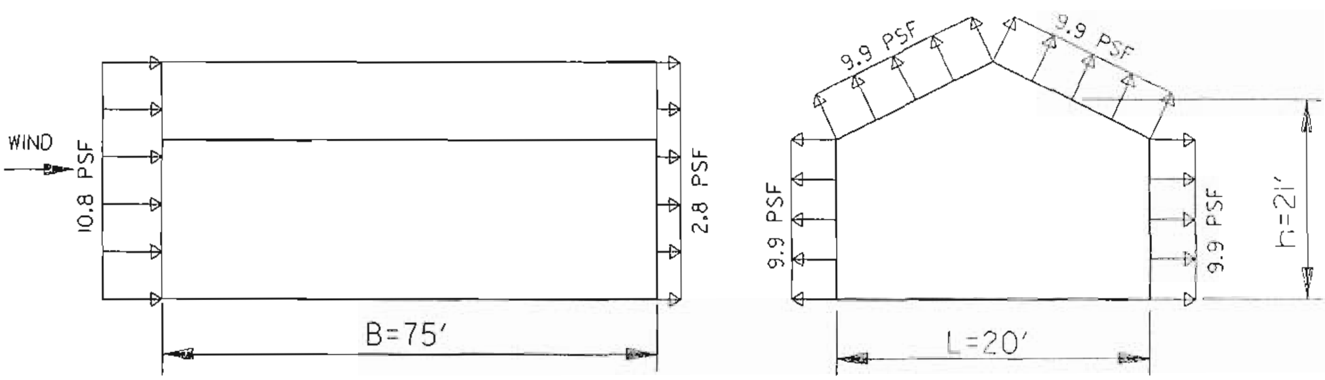
EXTERNAL WIND PRESSURE, p, ON BUILDING
WIND NORMAL TO RIDGE

Figure G-1. Design example for wind loads - industrial building. (Sheet 3 of 11)

(3)WIND PARALLEL WITH RIDGE

ELEMENT	Cp	$p=qG_hCp$,PSF	CONDITION *	
WINDWARD WALL	0.8	10.5x1.29x0.8=+10.8		
LEEWARD WALL	-0.2	11.0x1.29x(-0.2)=-2.8	B/L=75/20=3.8	FIG.2
ROOF	-0.7	11.0x1.29x(-0.7)=-9.9	h/L=21/20=1.1	FIG.2
SIDE WALL	-0.7	11.0x1.29x(-0.7)=-9.9		

*USE B/L FOR L/B AND h/B FOR h/L AS SHOWN IN FIG.2,WHEN WIND DIRECTION IS PARALLEL WITH RIDGE.

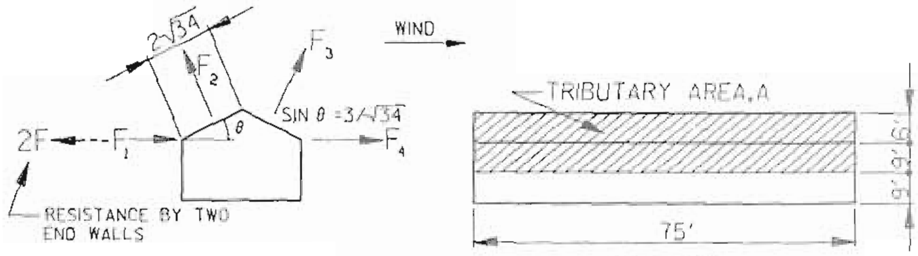


EXTERNAL WIND PRESSURE,p,ON BUILDING
WIND PARALLEL TO RIDGE

Figure G-1. Design example for wind loads - industrial building. (Sheet 4 of 11)

B. WIND SHEAR FORCES ON WALLS

(1) FORCE, F, ON ONE END WALL



END WALL

SIDE WALL

$2F = F_1 - F_2 \sin\theta + F_3 \sin\theta + F_4$

WHERE $F_1 = 10.8 \times 75 \times 9 = 7290$ LB

$F_2 \sin\theta = 4.3 \times 75 \times 2\sqrt{34} \times 3/\sqrt{34}$

$= 1935$ LB

$F_3 \sin\theta = 9.9 \times 75 \times 2\sqrt{34} \times 3/\sqrt{34}$

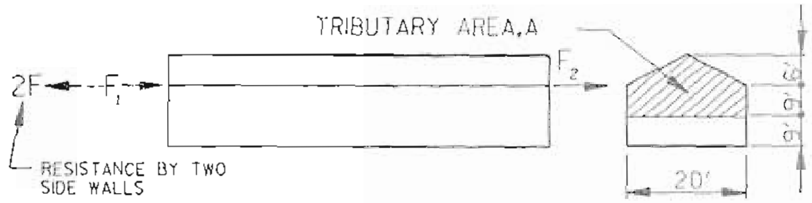
$= 4455$ LB

$F_4 = 7.1 \times 75 \times 9 = 4793$ LB

$2F = 7290 - 1935 + 4455 + 4793$

$F = 7301$ LB SAY 7.3^K

(2) FORCE, F, ON ONE SIDE WALL



SIDE WALL

END WALL

$2F = F_1 + F_2$

WHERE $F_1 = 10.8 \times 20 \times 12 = 2592$ LB

$F_2 = 2.8 \times 20 \times 12 = 672$ LB

$2F = 3592 + 672$

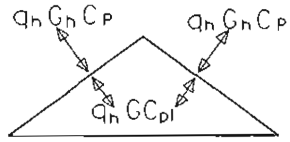
$F = 1632$ LB SAY 1.6^K

Figure G-1. Design example for wind loads - industrial building. (Sheet 5 of 11)

C. MAX WIND PRESSURE ON ROOF TRUSS (WIND PARALLEL WITH RIDGE)

EXT. PRESSURE → ← INTERNAL PRESSURE

(1) $p = q_h G_h C_p - q_h (GC_{pi})$ — TABLE 4



(2) EXTERNAL PRESSURE, $q_h G_h C_p$

$q_h G_h C_p = -9.9$ PSF — SEE A(3) OF THIS EXAMPLE

(3) INTERNAL PRESSURE, $q_h GC_{pi}$

$q_h = 11.0$ PSF — SEE A(1) OF THIS EXAMPLE

SINCE, FOR OPENINGS,
48% - 10% = 38% > 10% — SEE SKETCH OF INDUSTRIAL BLDG.
AND 10% < 20%
SELECT $GC_{pi} = +0.75$, OR -0.25 — TABLE 9
ASSUME WORST CASE. OPENINGS ON WINDWARD WALL ARE OPEN. OTHER WALL OPENINGS ARE CLOSED. ACCORDINGLY INTERNAL PRESSURE IS POSITIVE. SELECT $GC_{pi} = +0.75$.

$q_h GC_p = 11.0 \times 0.75 = +8.3$ PSF

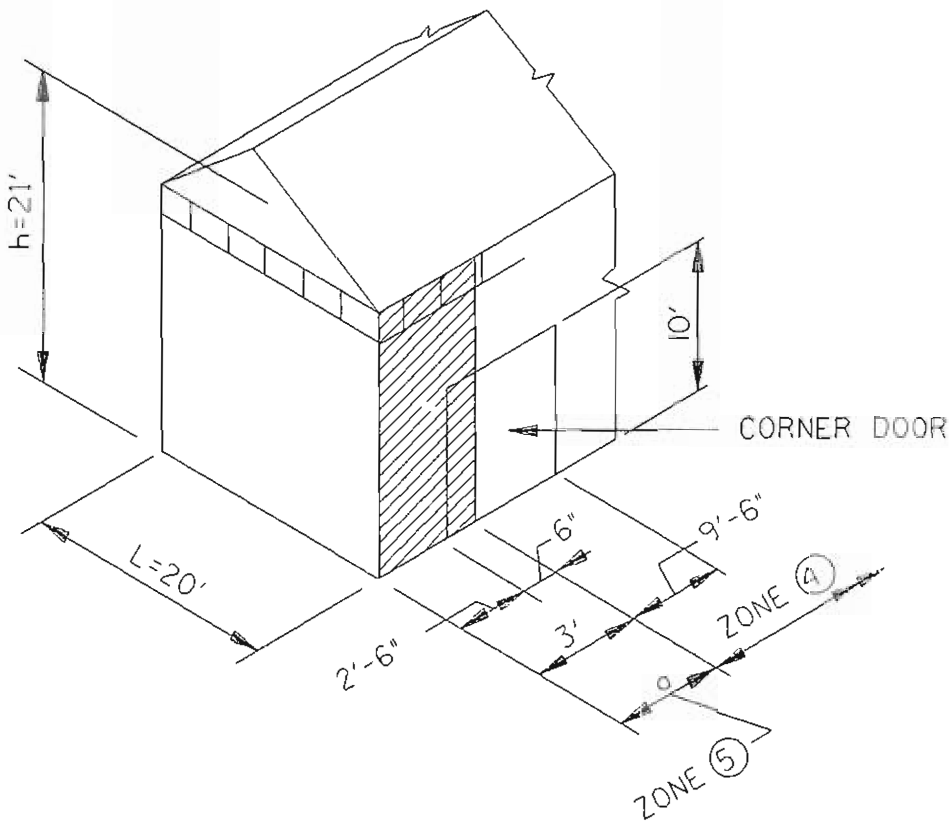
(4) MAX PRESSURE, p

$p = q_h G_h C_p - q_h (GC_{pi})$ — TABLE 4
$p = -9.9 - (+8.3) = -18.2$ PSF

MAXIMUM PRESSURE, p, ON ROOF TRUSS

Figure G-1. Design example for wind loads - industrial building. (Sheet 6 of 11)

D. PRESSURE ON DOOR (ASSUME WORST CASE. OPENINGS ON REAR WALL ARE OPEN. ALL OTHER OPENINGS ARE CLOSED.)



(1) WIDTH a FOR ZONE (5)
THE SMALLER OF
0.10L x W = 0.1 x 20 FT = 2 FT
0.4h = 0.4 x 21 FT = 8.4 FT
BUT NOT LESS THAN THE LARGER OF
0.04L = 0.04 x 20 = 0.8 FT
OR 3 FT ← GOVERNS

FIG. 3

Figure G-1. Design example for wind loads - industrial building. (Sheet 7 of 11)

(2)PRESSURE,p,ON ZONE (5)

$p = q_h(GC_p) - q_h(GC_{pi})$ TABLE 4

WHERE q_h=11.0 PSF SEE A(1) OF THIS EXAMPLE

A=10x10=100 FT^2 *

GC_p (FIG.3)	GC_{pi} (TAB. 9)	p(PSF)	DIRECTION
-1.5	+0.75	11.0 (-1.5)-11.0x0.75=-24.8	OUTWARD
+1.15	-0.25	11.0x1.15-11.0 (-0.25)=+15.4	INWARD

* ASSUMES DOOR IS STRENGTHED IN BOTH DIRECTIONS. FOR ONE DIRECTION A=10x10/3=33 FT^2.SEE PARA.6.2

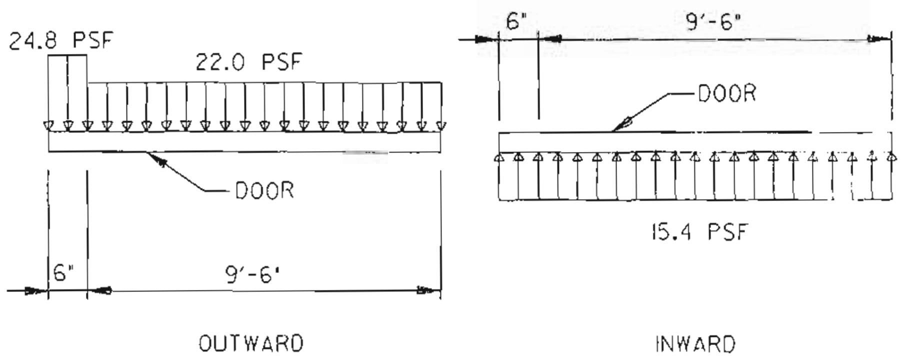
(3)PRESSURE,p,ON ZONE (4)

$p = q_h(GC_p) - q_h(GC_{pi})$

WHERE q_h=11.0 PSF

A=100 FT^2

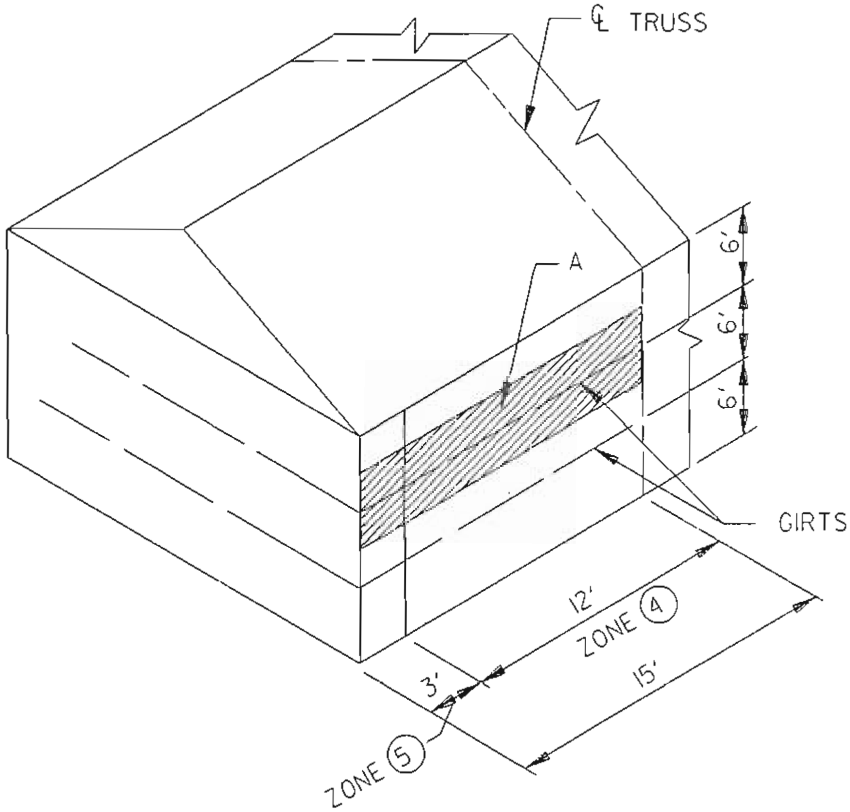
GC_p (FIG.3)	GC_{pi} (TAB.9)	p(PSF)	DIRECTION
-1.25	+0.75	1.0 (-1.25)-11.0x0.75=-22.0	OUTWARD
+1.15	-0.25	1.0x1.15-11.0 (-0.25)=+15.4	INWARD



WIND PRESSURE,p,ON CORNER DOOR

Figure G-1. Design example for wind loads - industrial building. (Sheet 8 of 11)

E. LOAD ON GIRT (NOTE: ASSUME WORST CASE FOR INTERNAL PRESSURE COEFFICIENTS.)



(I) PRESSURE, p, ON ZONE 5

$p = q_h(GC_p) - q_h(GC_{pi})$ TABLE 4

WHERE q_h = 11.0 PSF SEE A(I) OF THIS EXAMPLE

A = 6×15 = 90 FT^2

GC_p (FIG.3)	GC_{pi} (TAB.9)	p(PSF)	DIRECTION
-1.50	+0.75	11.0 (-1.50)-11.0×0.75=-24.8	OUTWARD
+1.20	-0.25	11.0×1.2-11.0 (-0.25)=+16.0	INWARD

Figure G-1. Design example for wind loads - industrial building. (Sheet 9 of 11)

(2) PRESSURE, p, ON ZONE (4)

$$p = q_h(GC_p) - q_h(GC_{pi})$$

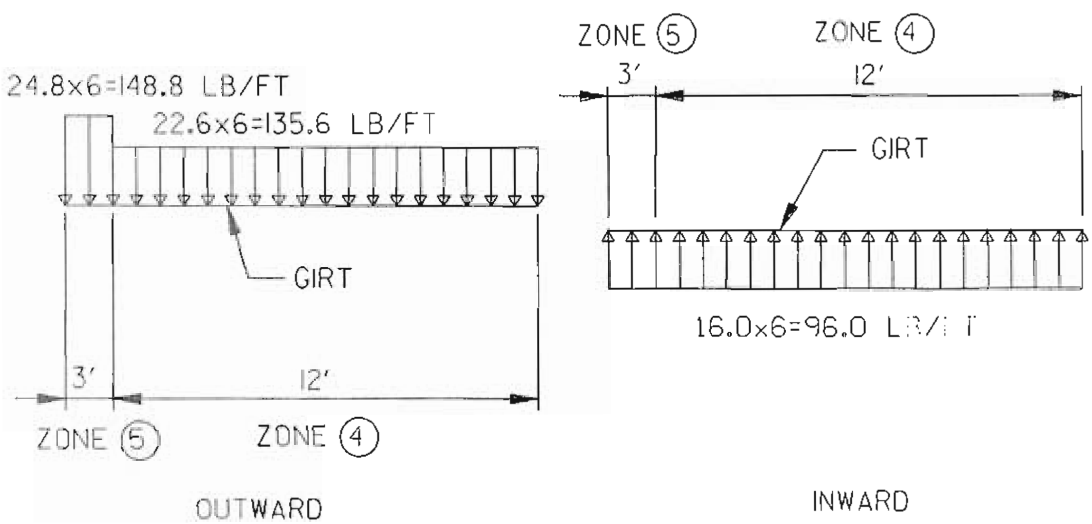
WHERE $q_h = 11.0$ PSF

TABLE 4

SEE A(1) OF THIS EXAMPLE

$A = 6 \times 15 = 90\ FT^2$

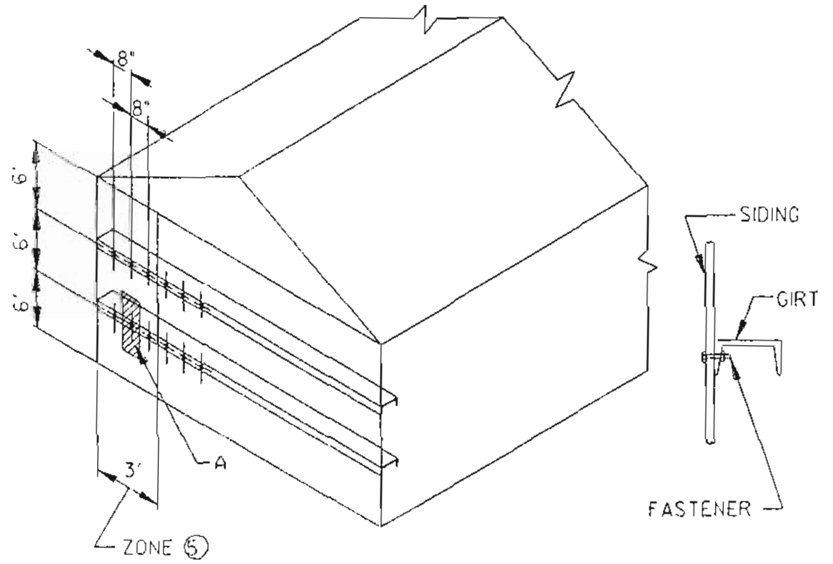
GC_p (FIG.3)	GC_{pi} (TAB.9)	p(PSF)	DIRECTION
-1.30	+0.75	11.0 (-1.30)-11.0x0.75=-22.6	OUTWARD
+1.20	-0.25	11.0x1.2-11.0 (-0.25)=+16.0	INWARD



LOAD ON GIRT

Figure G-1. Design example for wind loads - industrial building. (Sheet 10 of 11)

F. MAXIMUM TENSION ON WALL FASTENER



FASTENERS ARE IN MAXIMUM TENSION WHEN NEGATIVE EXTERNAL PRESSURE (SUCTION) IS CREATED IN ZONE ⑤

(1) TRIBUTARY AREA, A

LENGTH OF AREA = 6 FT
WIDTH OF AREA = 8 IN
BUT SHOULD NOT BE LESS THAN 1/3 THE LENGTH OF AREA OR 2 FT. ACCORDINGLY
$A = 6 \times 2 = 12\ FT^2$ — PARA. 6.2

(2) PRESSURE, p, ON ZONE ⑤

$p = q_h(GC_p) - q_h(GC_{pi})$ — TABLE 4
WHERE $q_h = 11.0$ PSF — SEE A(I) OF THIS EXAMPLE

$A = 12\ FT^2$ — WORST CASE

GC_p (FIG.3)	GC_{pi} (TAB.9)	p(PSF)	DIRECTION
-1.95	+0.75	11.0 (-1.95) - 11.0 × 0.75 = -29.7	OUTWARD

NEGATIVE EXT. PRESSURES — INTERNAL PRESSURE

(3) TENSION FORCE, T, ON FASTENER

$T = 29.7 \times 6 \times 8/12 = 118.8$ LB

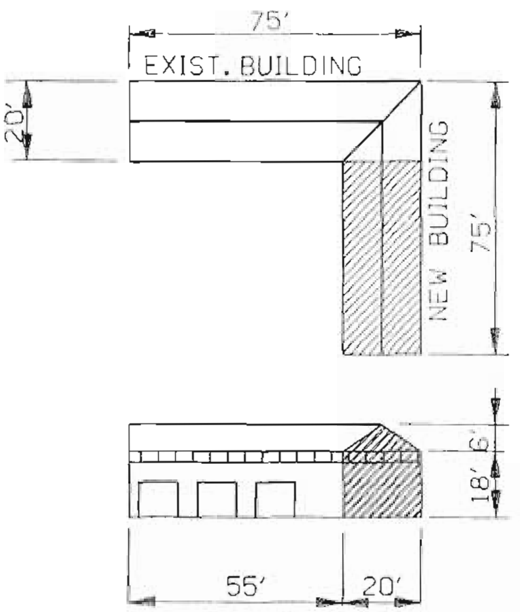
Figure G-1. Design example for wind loads - industrial building. (Sheet 11 of 11)

GIVEN: INDUSTRIAL BLDG WITH IRREGULAR PLAN CONFIGURATION.

THE SAME BUILDING IN EXAMPLE G-1 IS EXPANDED AS SHOWN IN FIGURE BELOW.

PROBLEM: DETERMINE THE FOLLOWING RESULTING FROM WIND.

A.EXTERNAL PRESSURE ON THE BUILDING
B.MAXIMUM PRESSURE ON ROOF TRUSS
C.PRESSURE ON DOOR
D.LOAD ON GIRT
E.MAXIMUM TENSION ON WALL FASTENER
F.SHEAR FORCES ON WALLS



INDUSTRIAL BUILDING

Figure G-2. Design example for wind loads - industrial building with irregular plan configuration. (Sheet 1 of 4)

SOLUTION:

A. EXTERNAL PRESSURE ON BUILDING.	SAME AS EXAMPLE G-1.
B. MAXIMUM PRESSURE ON ROOF TRUSS.	SAME AS EXAMPLE G-1.
C. PRESSURE ON DOOR.	SAME AS EXAMPLE G-1.
D. LOAD ON GIRT.	SAME AS EXAMPLE G-1.
E. MAX TENSION ON WALL FASTENER.	SAME AS EXAMPLE G-1.
G. SHEAR FORCES ON WALLS (APPROXIMATE)	

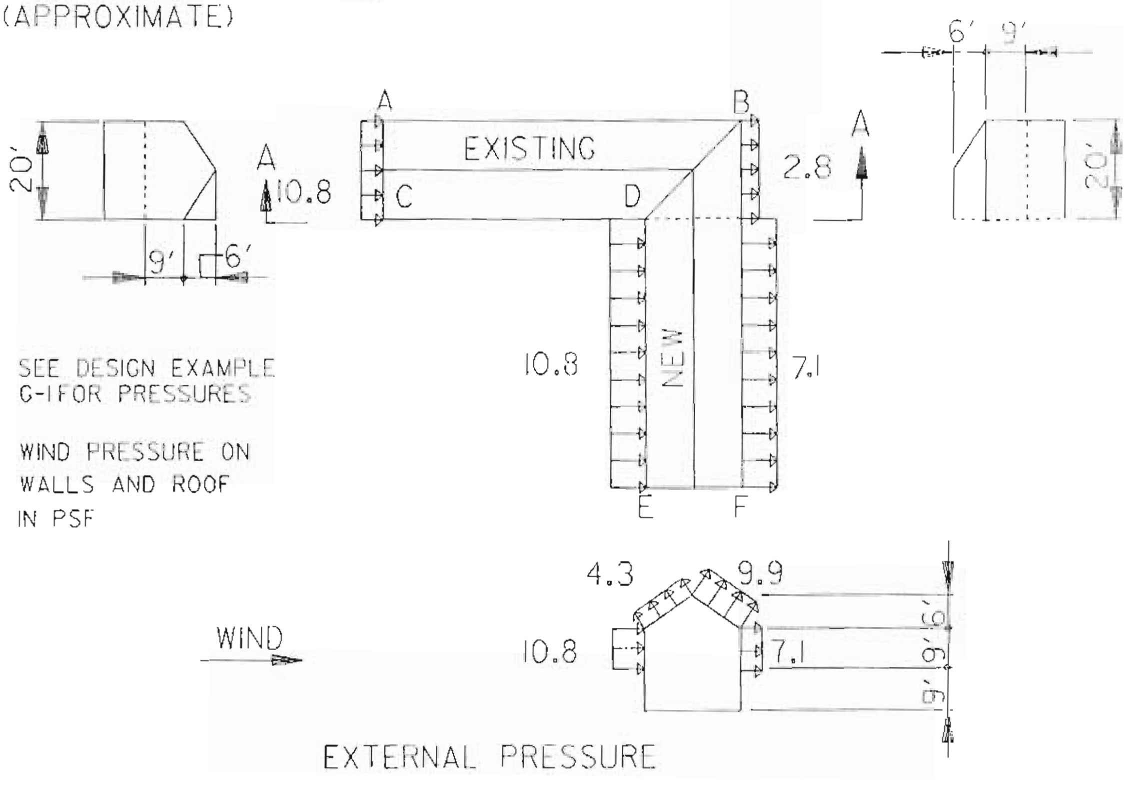
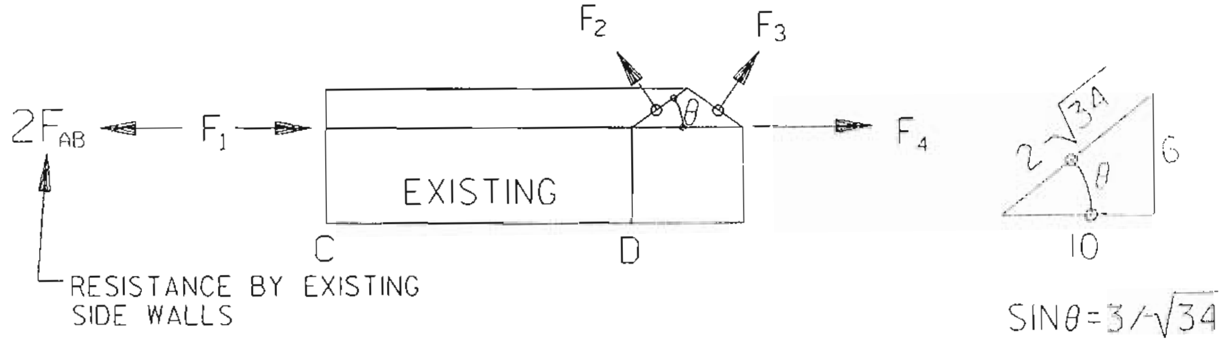


Figure G-2. Design example for wind loads - industrial building with irregular plan configuration. (Sheet 2 of 4)

1. SHEAR ON EXISTING WALL AB AND CD RESULTING FROM LOAD ON EXISTING BLDG.



SECTION A-A
EXISTING BUILDING

$2F_{AB} = 2F_{CD} = F_1 - F_2 \sin\theta + F_3 \sin\theta + F_4$

$F_{AB} = F_{CD} = (F_1 - F_2 \sin\theta + F_3 \sin\theta + F_4)/2$

WHERE $F_1 = 10.8 \times 20 \times 12 = 2592$ LB

$F_2 \sin\theta = (4.3 \times 2\sqrt{34} \times 10 \times 1/2)3/\sqrt{34}$

$= 129$ LB

$F_3 \sin\theta = 9.9[(10+20)1/2 \times 2\sqrt{34}]3/\sqrt{34}$

$= 891$ LB

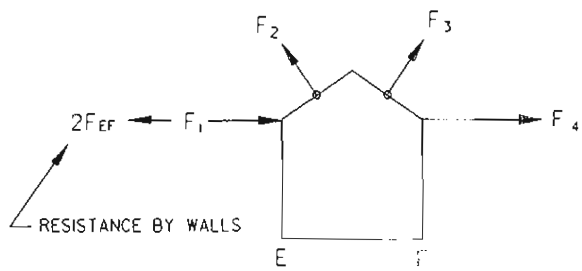
$F_4 = 2.8 \times 20 \times 9 = 504$ LB

$F_{AB} = (2592 - 129 + 891 + 504)/2 = 1929$ LB, SAY 1.9^K

$F_{CD} = 1.9^K$

Figure G-2. Design example for wind loads - industrial building with irregular plan configuration. (Sheet 3 of 4)

2.SHEAR ON NEW WALL EF AND EXISTING WALL CD RESULTING FROM LOAD ON NEW BLDG.



NEW BUILDING

$2F_{EF} = 2F_{CD} = F_1 - F_2 \, SIN\theta + F_3 \, SIN\theta + F_4$

F_{EF} = WHERE: $F_{CD} = (F_1 - F_2 \, SIN\theta + F_3 \, SIN\theta + F_4)/2$

$F_1 = 10.8 \times 55 \times 9) = 5346$ LB

$F_2 \, SIN\theta = (4.3 \times 55 \times 2\sqrt{34}) \; 3/\sqrt{34} = 1419$ LBS

$F_3 \, SIN\theta = (9.9 \times 55 \times 2\sqrt{34}) \; 3/\sqrt{34} == 3267$ LBS

$F_4 = 7.1 \times 55 \times 9 = 3515$ LBS

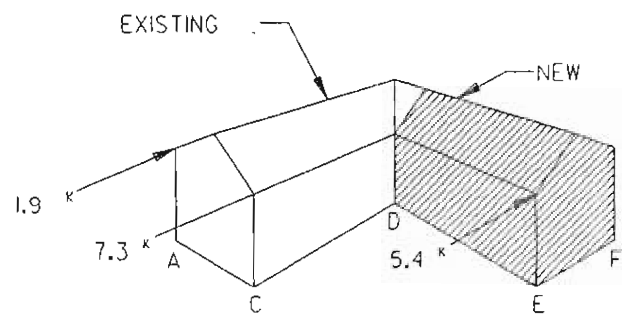
$F_{EF} = (5346 - 1419 + 3267 + 3515)/2 = 5354$ LBS

SAY 5.4^K

$F_{CD} = 5.4^K$

3.TOTAL SHEAR FORCE ON EXISTING WALL CD FROM WIND LOAD ON EXISTING AND NEW BUILDING.

TOTAL $F_{CD} = 1.9^K + 5.4^K = 7.3^K$



SHEAR FORCE ON WALLS

Figure G-2. Design example for wind loads - industrial building with irregular plan configuration. (Sheet 4 of 4)

GIVEN: THREE STORY BUILDING,h 60≤FT, SHOWN BELOW.

PROBLEM: THE MAIN WIND FORCE RESISTING SYSTEM OF THE THREE STORY ADMINISTRATIVE BUILDING SHOWN BELOW IS TO BE DESIGNED.DETERMINE THE DESIGN WIND PRESSURES ON THE BUILDING.

MISSISSIPPI ARMY AMMUNITION PLANT,MS.
WIND EXPOSURE CATEGORY C
BUILDING CATEGORY I

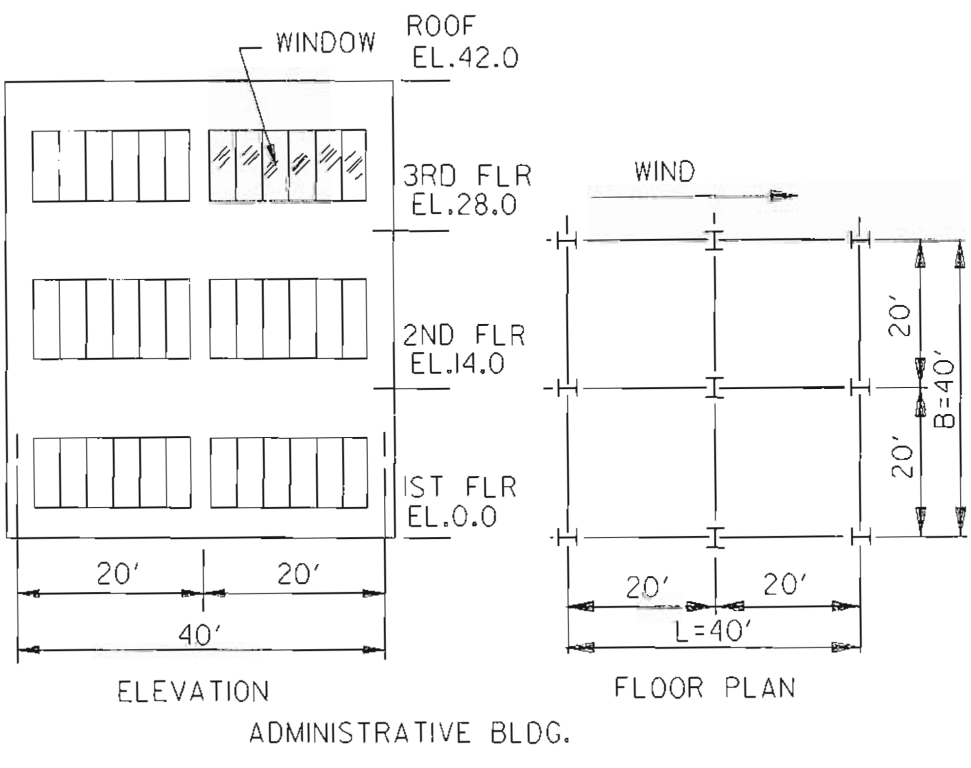


Figure G-3. Design example for wind loads - three-story building (height less than or equal to 60 feet). (Sheet 1 of 3)

SOLUTION: DESIGN WIND PRESSURE ON BUILDING

(I) WINDWARD WALL

$p = q_z G_h C_p - q_h (GC_{pi})$ — TABLE 4*

NOTE: NEGLECT INTERNAL PRESSURE TERM $-q_h (GC_{pi})$ WHEN ONLY EXTERNAL PRESSURES ARE CONSIDERED.

WHERE $q_z = 0.00256 K_z (IV)^2$ — EQ.3

$I = 1.05$ — TABLE 5

$V = 100$ MPH — APPENDIX B (THIS MANUAL)

$q_z = 0.00256 K_z (1.05 \times 100)^2$ — EQ.3

$= 28.22 K_z$

$G_h = 1.23$ AT $h = 42'$ — TABLE 8

$C_p = 0.8$ — FIG.2

* ALL REFERENCES IN THIS EXAMPLE ARE TO ASCE 7-88 U.N.O.

PRESSURE ON WINDWARD WALL, p					
Z	K_z	$28.22K_z$ q_z (1)	G_h (2)	C_p (3)	p (1)x(2)x(3)
42	1.07	30.2	1.23	0.8	29.7
35	1.02	28.8	1.23	0.8	28.3
21	0.88	24.8	1.23	0.8	24.4
7	0.80	22.6	1.23	0.8	22.2

Figure G-3. Design example for wind loads - three-story building (height less than or equal to 60 feet). (Sheet 2 of 3)

(2) LEEWARD WALL

$p = q_z G_h C_p - q_h (GC_{pi})$ TABLE 4

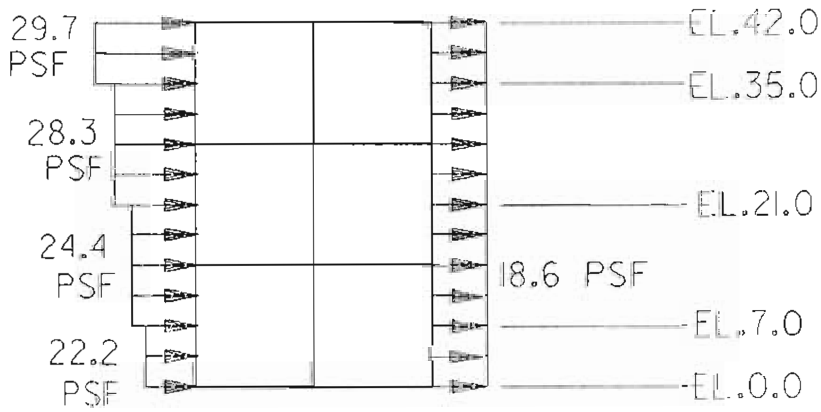
NOTE: NEGLECT INTERNAL PRESSURE TERM $-q_h (GC_{pi})$ WHEN ONLY EXTERNAL PRESSURES ARE CONSIDERED.

WHERE $q_h = 30.2$ PSF AT $h = Z = 42'$ TABLE ABOVE

$G_h = 1.23$ TABLE ABOVE

$C_p = -0.5$ (WHEN $L/B = 1$) FIG.2

$p = 30.2 \times 1.23 \times -0.5 = -18.6$ PSF



DESIGN WIND PRESSURE ON BUILDING

Figure G-3. Design example for wind loads - three-story building (height less than or equal to 60 feet). (Sheet 3 of 3)

GIVEN: FIVE STORY BUILDING, h>60 FT, SHOWN BELOW.

PROBLEM: DETERMINE THE DESIGN WIND PRESSURE ON THE FILLER WALL ON THE FIFTH FLOOR OF THE FIVE STORY ADMINISTRATION BUILDING SHOWN BELOW.

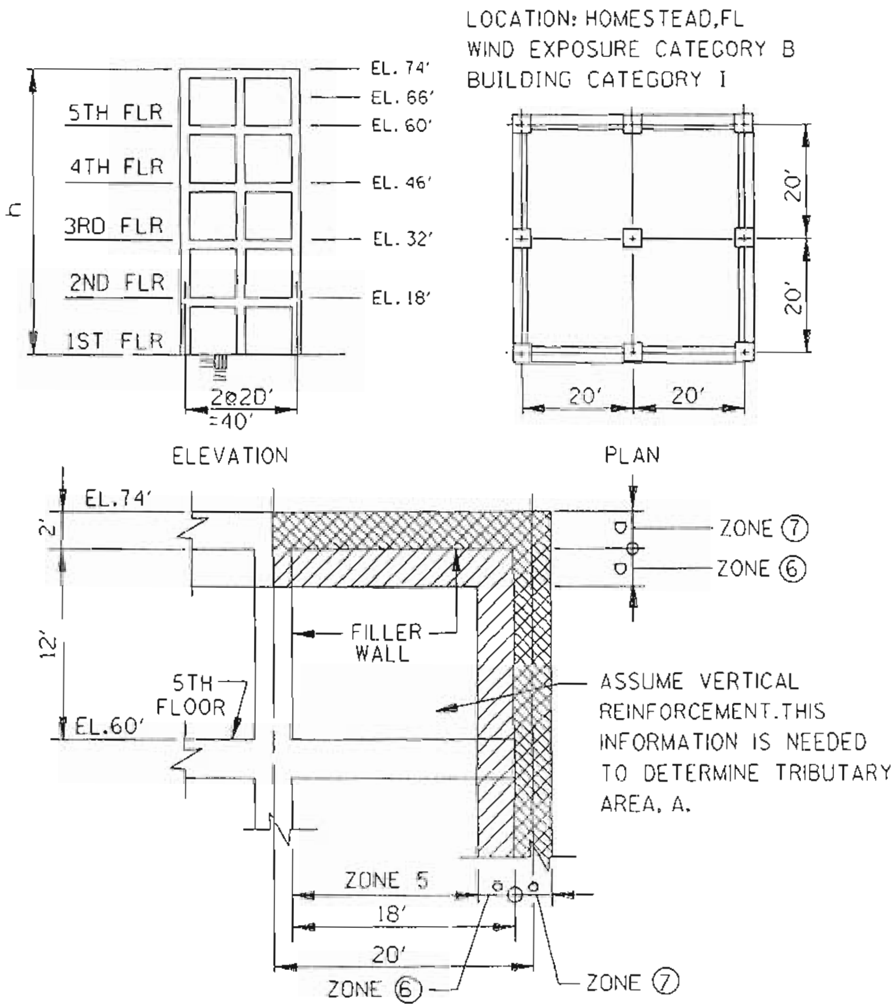


Figure G-4. Design example for wind loads - five-story building (height greater than 60 feet). (Sheet 1 of 3)

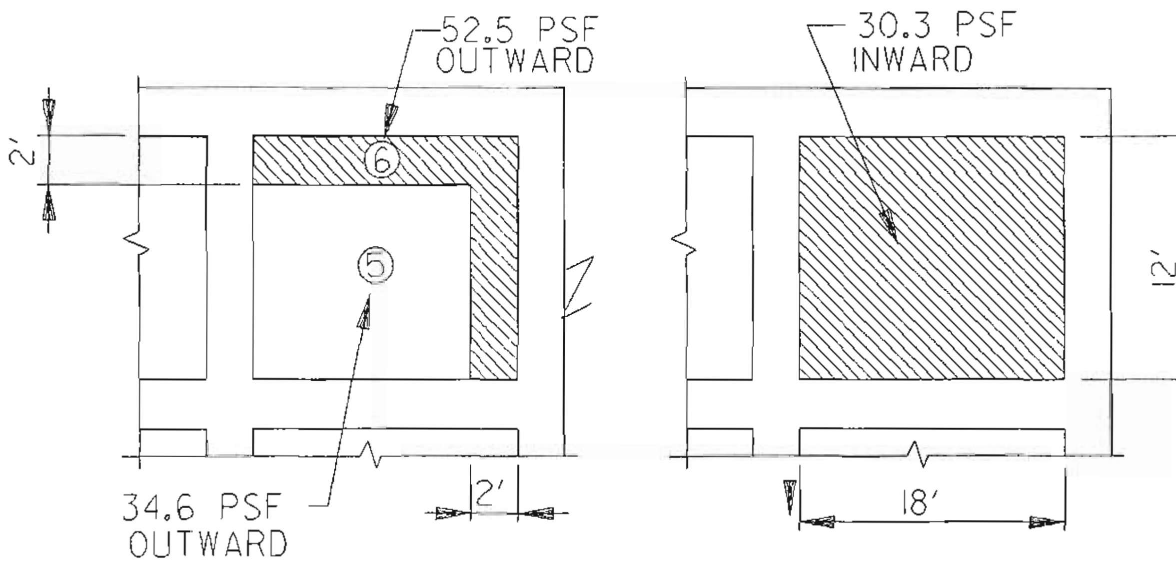
SOLUTION:

DETERMINE WIDTH, a	*ALL REFERENCES IN THIS EXAMPLE ARE TO ASCE 7-88 U.N.O.
SELECT SMALLER	
0.05x40=2 FT ←—— GOVERNS	FIG.4*
0.5x74=37 FT	NOTATIONS
DESIGN WIND PRESSURE, p, ON FILLER WALL	
$p=q(GC_p)-q_z(GC_{pi})$	TABLE 4
$q_z=0.00256K_z(IV)^2$	EQ.3
WHERE $K_z=0.71$ AT Z=66 FT	TABLE 6
I=1.05	TABLE 5
V=110 MPH	APPENDIX B (THIS MANUAL
$q_z=0.00256\times0.71(1.05\times110)^2=24.2$ PSF	EQ.3
$q_h=0.00256K_h(IV)^2$	EQ.3
WHERE $K_h=0.75$ AT h=74 FT	TABLE 6
$q_h=0.00256\times0.75(1.05\times110)^2=25.6$ PSF	EQ.3
ZONE 6	
$A=12\times12/3=48$ FT^2	PARA.6.2
$GC_p=+1.00,-1.80$	FIG.4
$GC_{pi}=\pm0.25$	TABLE 9
p=24.2(+1.00)-24.2(-0.25) =+30.3 PSF	TABLE 4
p=25.6(-1.80)-24.2(+0.25) =-52.1 PSF	TABLE 4

Figure G-4. Design example for wind loads - five-story building (height greater than 60 feet). (Sheet 2 of 3)

ZONE 5

$A=12 \times 12/3=48\ FT^2$	PARA.6.2
$GC_p=+1.00,-1.10$	FIG.4
$GC_{pi}=\pm 0.25$	TABLE 9
$p=24.2[1.00-(-0.25)]=30.3\ PSF$	TABLE 4
$p=25.6[-1.10-0.25]=-34.6\ PSF$	TABLE 4



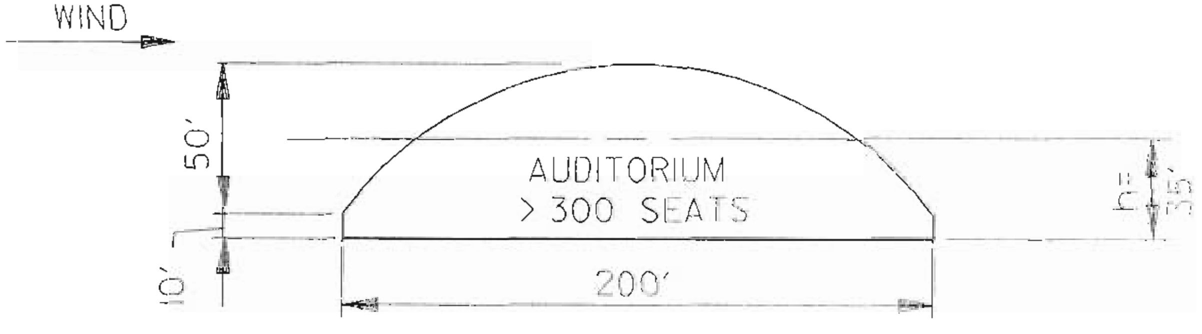
DESIGN WIND PRESSURE ON FILLER WALL

Figure G-4. Design example for wind loads - five-story building (height greater than 60 feet). (Sheet 3 of 3)

GIVEN: ARCHED ROOF SHOWN BELOW.

PROBLEM: DETERMINE THE DESIGN WIND PRESSURE ON THE ARCHED ROOF SHOWN BELOW FOR THE MAIN WIND-FORCE RESISTING SYSTEM.

LOCATION:ROBINS AFB.GA.
WIND EXPOSURE CATEGORY C
BUILDING CATEGORY II



ARCHED ROOF

Figure G-5. Design example for wind loads - arched roof. (Sheet 1 of 3)

DESIGN WIND PRESSURE

*NOTE: ALL REFERENCES IN THIS EXAMPLE ARE TO ASCE 7-88 U.N.O.

$p = q_h G_h C_p - q_h (GC_{pi})$	TABLE 4
NOTE: NEGLECT INTERNAL PRESSURE TERM $-q_h (GC_{pi})$ WHEN ONLY EXTERNAL PRESSURES ARE CONSIDERED.	
WHERE $q_h = 0.00256 K_z (IV)^2$	EQ.3
$K_z = 1.02$ WHERE Z=35 FT	TABLE 6
$I = 1.07$	TABLE 5
$V = 75$ MPH	APPENDIX B
$q_h = 0.00256 \times 1.02 (1.07 \times 75)^2$	EQ.3
$= 16.8$ PSF	
$G_h = 1.25$ AT h=35 FT	TABLE 8
RISE, $r = 50/200 = 0.25$	
THEREFORE $0.2 \leq r < 0.3$	TABLE 10
WINDWARD QUARTER, C_p	
$C_p = (1.5r - 0.3) = (1.5 \times 0.25 - 0.3)$	TABLE 10
$= +0.075$	
ALSO $C_p = (6r - 2.1) = (6 \times 0.25 - 2.1)$	TABLE 10
$= -0.6$	
CENTER HALF, C_p	
$C_p = (-0.7 - r) = -0.7 - 0.25 = -0.95$	TABLE 10
LEEWARD QUARTER, C_p	
$C_p = -0.5$	TABLE 10

Figure G-5. Design example for wind loads - arched roof. (Sheet 2 of 3)

DESIGN WIND PRESSURE-WINDWARD QUARTER

P=16.8x1.25x+0.075=+1.6 PSF TABLE 4

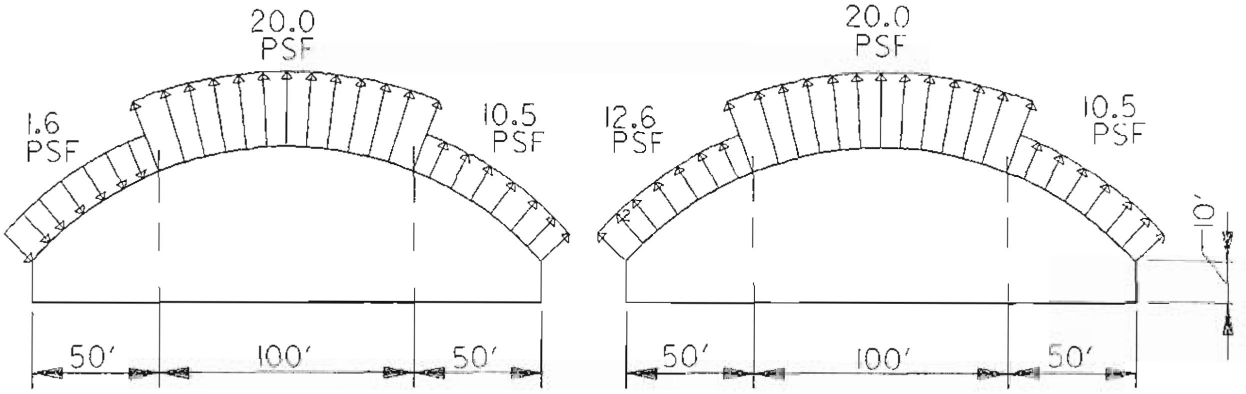
P=16.8x1.25x-0.6=-12.6 PSF TABLE 4

DESIGN WIND PRESSURE-CENTER HALF

P=16.8x1.25x-0.95=-20.0 TABLE 4

DESIGN WIND PRESSURE-LEEWARD QUARTER

P=16.8x1.25x-0.50=-10.5 TABLE 4



DESIGN PRESSURE ON ARCHED ROOF

Figure G-5. Design example for wind loads - arched roof. (Sheet 3 of 3)

<u>GIVEN:</u> MONOSLOPE ROOF SUBJECTED TO FORCE, F, SHOWN BELOW.

<u>PROBLEM:</u> AN OPEN SIDED STRUCTURE SHOWN BELOW, IS BEING DESIGNED AS PART OF AN OPEN STORAGE FACILITY. FOR DESIGNING THE ROOF COMPONENTS, DETERMINE WIND FORCE, F .

LOCATION: HICKMAN AFB, HONOLULU, HAWAII
WIND EXPOSURE CATEGORY D
BUILDING CATEGORY I

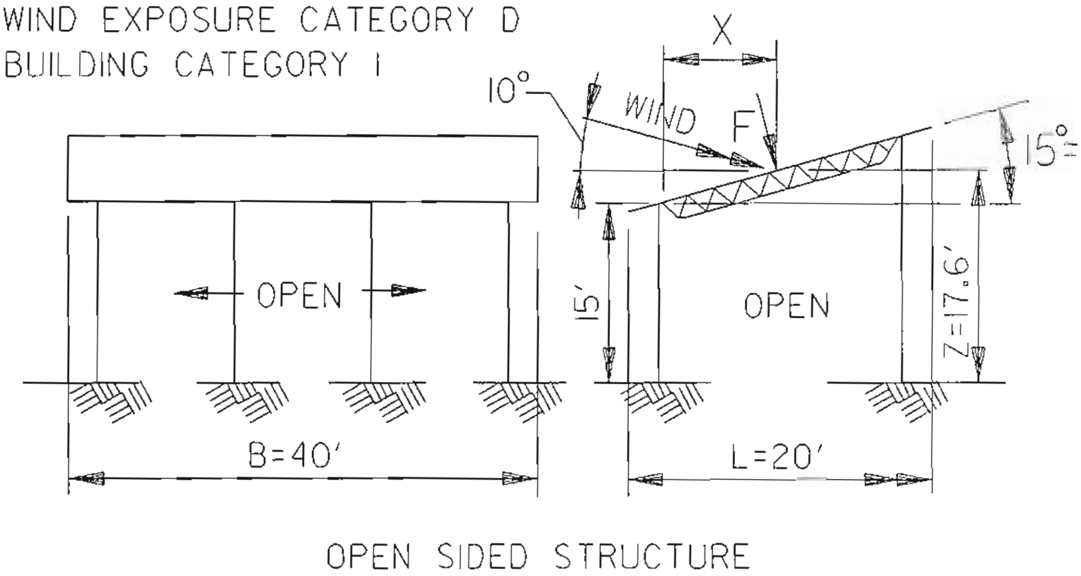


Figure G-6. Design example for wind loads - monoslope roof subjected to wind force. (Sheet 1 of 2)

SOLUTION:

WIND FORCE ON ROOF

$F = q_z G_z C_f A_f$	TABLE 4*
WHERE $q_z = 0.00256K_z(IV)^2$	EQ.3
$K_z = 1.24$ AT h=17.6 FT	TABLE 6
I=1.00	TABLE 5
V=80 MPH	APPENDIX B (THIS MANUAL)
$q_z = 0.00256 \times 1.24(1.00 \times 80)^2$	EQ.3
=20.3 PSF	
$G_z = 1.14$ AT h=17.6 FT	TABLE 8
B/L=40/20=2.0	TABLE 11 NOTE 2
$\theta = 15^\circ \pm 10^\circ$	
USE $15^\circ + 10^\circ = 25^\circ$ WORST CASE	
$C_f = 1.1$	
X/L=0.4	
X=0.4L=0.4(20)=8.0 FT	
$A_f = 40 \times 20/\cos 15^\circ = 828.2$ SF	
$F = 20.3 \times 1.14 \times 1.1 \times 828.2 = 21.1^K$	TABLE 4

IF WIND IS BLOWING FROM LEFT $F = +21.1^K$ AS SHOWN
IF WIND IS BLOWING FROM RIGHT $F = -21.1^K$
AND X IS MEASURED 8 FT FROM THE RIGHT EDGE

*ALL REFERENCES ARE TO ASCE 7-88 U.N.O.

Figure G-6. Design example for wind loads - monoslope roof subjected to wind force. (Sheet 2 of 2)

GIVEN: MONOSLOPE ROOF SUBJECTED TO WIND PRESSURE.

PROBLEM: TRANSLATE THE WIND FORCE,F,IN EXAMPLE G-6 INTO WIND PRESSURE,p.

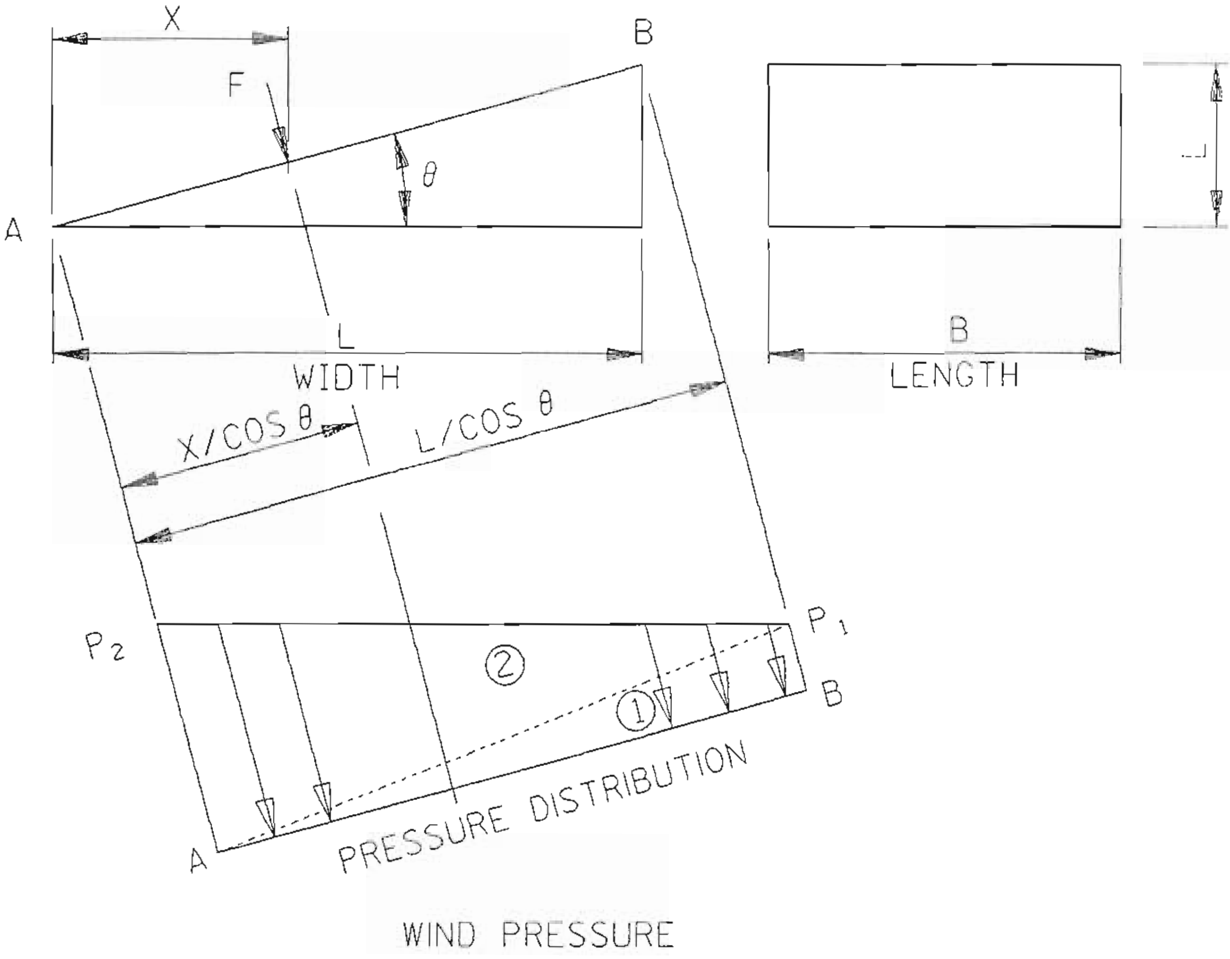


Figure G-7. Design example for wind loads - monoslope roof subjected to wind pressure. (Sheet 1 of 3)

EQUIVALENT FORCE NORMAL TO ROOF -LB/FT

EQ.1 $\frac{1}{2} P_2 (L/\cos\theta) + \frac{1}{2} P_1 (L/\cos\theta) = F/B$

EQUIVALENT MOMENT ABOUT POINT A -LB

EQ.2 $\frac{1}{2} P_2 (L/\cos\theta)[L/(3\cos\theta)] +$

$\frac{1}{2} P_1 (L/\cos\theta)[2L/(3\cos\theta)] = (F/B)(X/\cos\theta)$

SOLVING EQ.1 AND 2

$P_2 = (2F\cos\theta/BL)(2-3X/L)$

$P_1 = (2F\cos\theta/BL)(3X/L-1)$

THE FOLLOWING VALUES WERE OBTAINED FROM EXAMPLE G-6.

F=21,100 LB

L=20 FT

B=40 FT

X=8 FT

$\theta = 15^\circ$

USING THE FORMULAS

$P_2 = (2F\cos\theta/BL)(2-3X/L)$

$P_2 = [(2\times21{,}100\cos 15^\circ)(40\times20)][2-(3\times8/20)] = 40.8$ PSF

$P_1 = (2F\cos\theta/BL)(3X/L-1)$

$P_1 = [(2\times21{,}100\cos 15^\circ)/(40\times20)][(3\times8/20)-1] = 10.2$ PSF

Figure G-7. Design example for wind loads - monoslope roof subjected to wind pressure. (Sheet 2 of 3)

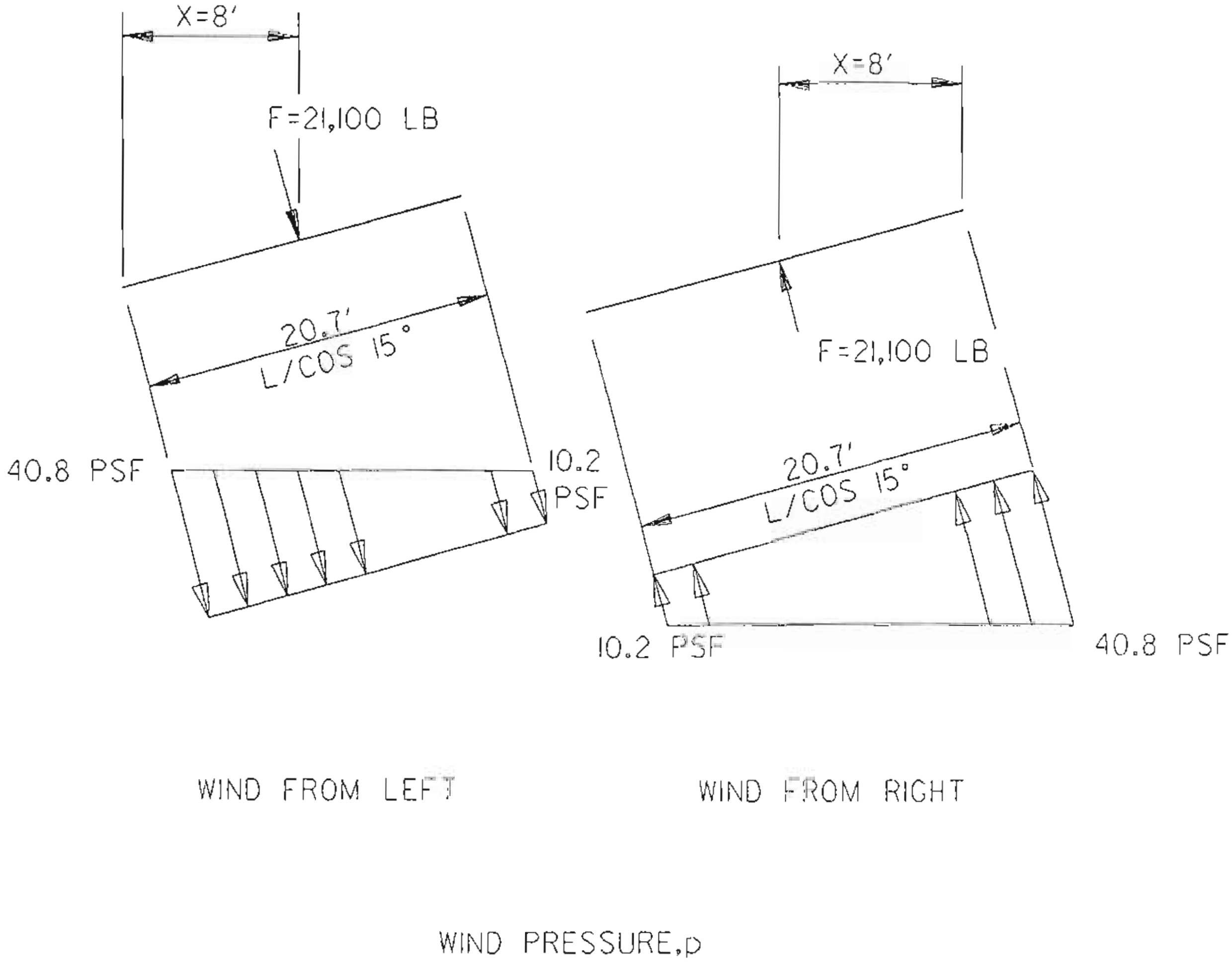


Figure G-7. Design example for wind loads - monoslope roof subjected to wind pressure. (Sheet 3 of 3)

GIVEN: CIRCULAR TANK ON BUILDING ROOF SHOWN BELOW.

PROBLEM: DETERMINE THE WIND LOAD ON A CIRCULAR WATER TANK BELOW. THE TANK IS LOCATED ON THE ROOF OF A MULTISTORY HOSPITAL.

HEIGHT, h=10 FT
DIAMETER, d=10 FT
LOCATION: FORT LEWIS, WA.
WIND EXPOSURE CATEGORY C
BUILDING CATEGORY III

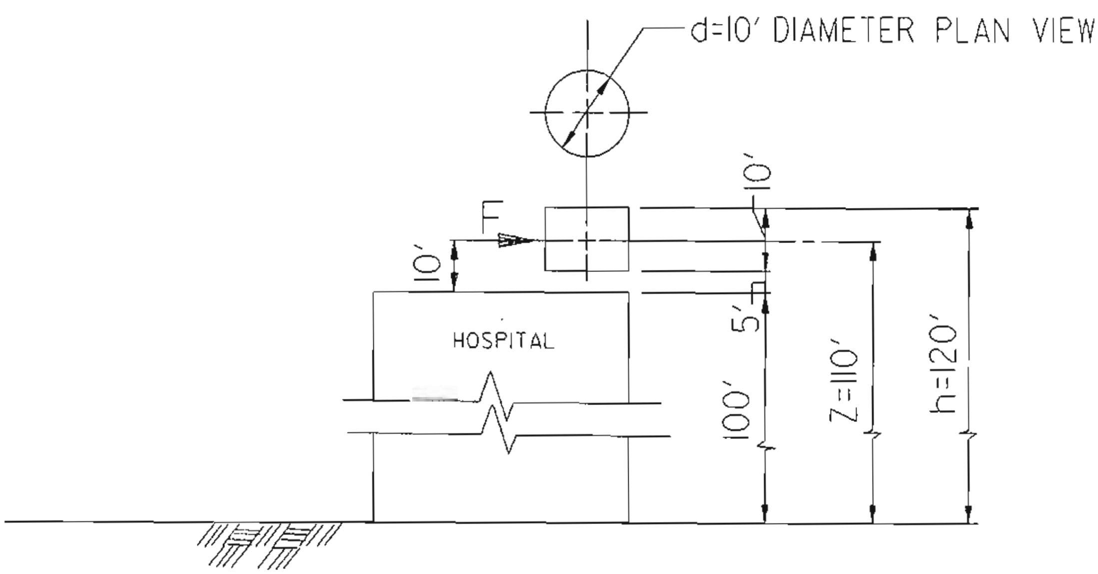


Figure G-8. Design example for wind loads - circular tank on building roof. (Sheet 1 of 2)

SOLUTION:

WIND PRESSURE

$F = q_z G_h C_f A_f$	TABLE 4*
WHERE $q_z = 0.00256\ K_z (IV)^2$	EQ.3
$Z = 110'$	
$I = 1.07$	TABLE 5
$V = 75$ MPH	APPENDIX B (THIS MANUAL)
$K_z = 1.42$ WHERE $Z = 110$ FT	TABLE 6
$q_z = (0.00256)(1.42)(1.07 \times 75)^2$	EQ.3
$q_z = 23.4$ PSF	
$G_h = 1.15$ WHERE $h = 120$ FT	TABLE 8
$d\sqrt{q_z} = 10\sqrt{23.4} = 48.4 > 2.5$	TABLE 12
$h/d = 1/1 = 1$	
$C_f = 0.5$	
$A_f = 10' \times 10' = 100\ FT^2$	
$F = 23.4 \times 1.15 \times 0.5 \times 100 = 1345\#$ OR 1.3^K	TABLE 4

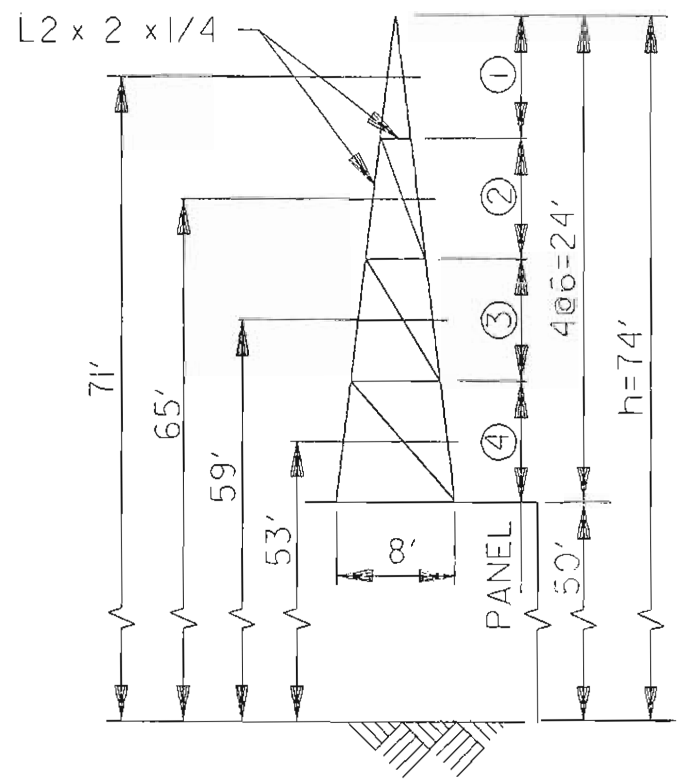
*ALL REFERENCES ARE TO ASCE 7-88 U.N.O.

Figure G-8. Design example for wind loads - circular tank on building roof. (Sheet 2 of 2)

GIVEN: THE TRIANGULAR STEEL TOWER SHOWN BELOW IS LOCATED ON THE ROOF OF A MULTISTORY BUILDING.

PROBLEM: DETERMINE THE WIND LOAD ON THE TOWER.

LOCATION: FORT WORTH, TEXAS
WIND EXPOSURE CATEGORY C
BUILDING CATEGORY I



TRIANGULAR STEEL TOWER

Figure G-9. Design example for wind loads - trussed tower on building roof. (Sheet 1 of 3)

SOLUTION:

PANEL	SOLID AREA A_f, FT^2 ①	GROSS AREA, FT^2 ②	ϵ ① ÷ ②
①	2.19	6.00	0.366
②	3.65	18.00	0.202
③	4.16	30.00	0.139
④	4.06	42.00	0.097

$F = q_z G_h C_f A_f$ — TABLE 4*

WHERE $q_z = 0.00256 K_z (IV)^2$ — E0.3

$I = 1.0$ — TABLE 5

$V = 70$ MPH — APPENDIX B (THIS MANUAL)

$q_z = 0.00256(1.0 \times 70)^2 K_z = 12.54 K_z$ — E0.3

$G_h = 1.19$ WHEN $h = 74'$ — TABLE 8

*ALL REFERENCES IN THIS EXAMPLE ARE TO ASCE 7-88 U.N.O.

FORCE COEFFICIENT, C_f

$C_f = (3.7 - 4.5\epsilon)$ — TABLE 15

$C_{f1} = (3.7 - 4.5 \times 0.366) = 2.05$

$C_{f2} = (3.7 - 4.5 \times 0.202) = 2.79$

$C_{f3} = (3.7 - 4.5 \times 0.139) = 3.07$

$C_{f4} = (3.7 - 4.5 \times 0.097) = 3.26$

Figure G-9. Design example for wind loads - trussed tower on building roof. (Sheet 2 of 3)

PANEL	Z FT	K_z TABLE 6	q_z PSF (1) (—12.54K_z)	G_h (2)	C_f (3)	A_f FT^2 (4)	F LBS (1)x(2)x(3)x(4)
①	71	1.25	15.68	1.19	2.05	2.20	83.8
②	65	1.22	15.30	1.20	2.79	3.65	185.4
③	59	1.18	14.80	1.20	3.07	4.16	224.9
④	53	1.15	14.42	1.20	3.26	4.06	227.1

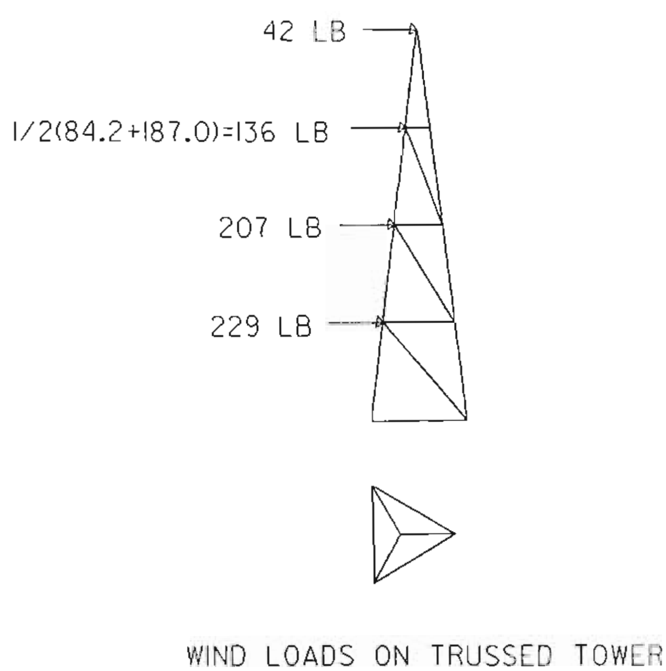


Figure G-9. Design example for wind loads - trussed tower on building roof. (Sheet 3 of 3)

APPENDIX H

DESIGN EXAMPLES FOR SNOW LOADS

H-1. Purpose and scope

This appendix contains illustrative examples using the snow load criteria given in ASCE 7-88.

H-2. Abbreviations

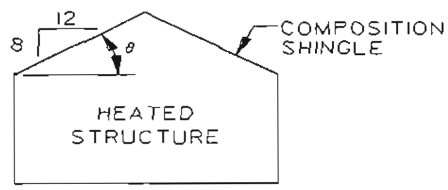
The following abbreviations are used in the example problems:

a. Eq. - Equation
b. Para. - Paragraph
c. Fig. - Figure
d. U.N.O. - Unless noted otherwise.

GIVEN: THE DORMITORY SHOWN BELOW IS SITED AMONG SEVERAL NEARBY PINE TREES.

PROBLEM: DETERMINE THE BALANCED AND UNBALANCED SNOW LOADS.

LOCATION: WESTOVER AFB, MA.
BUILDING CATEGORY I



θ = ARCTAN 8/12 = 34°

SOLUTION: FLAT ROOF SNOW LOAD

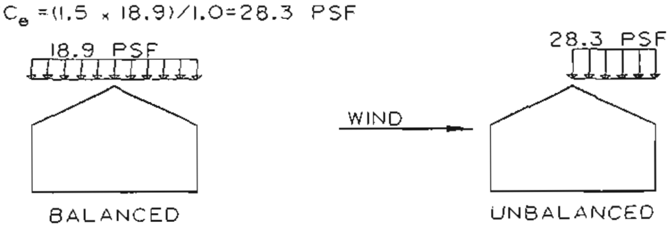
$p_f = 0.7 C_e C_t I p_g$	EQ.5a*
WHERE $C_e = 1.0$	TABLE 18
$C_t = 1.0$	TABLE 19
$I = 1.0$	TABLE 20
$p_g = 30$ PSF	APPENDIX B (THIS MANUAL)
$p_f = 0.7 \times 1.0 \times 1.0 \times 1.0 \times 30 = 21.0$ PSF	EQ.5a
(SINCE $\theta > 15°$, MIN. SNOW LOAD DOES NOT APPLY)	PARA.7.3.4

SLOPED ROOF SNOW LOAD

$p_s = C_s p_f$	EQ.6
WHERE $C_s = 0.9$	COMMENTARY SECTION 7
$p_s = 0.9 \times 21.0 = 18.9$ PSF	EQ.6

UNBALANCED SNOW LOAD

SINCE $15° < \theta < 70°$, UNBALANCED CONDITION APPLIES	FIG.9
$1.5 p_s / C_e = (1.5 \times 18.9)/1.0 = 28.3$ PSF	



ROOF SNOW LOAD

*ALL REFERENCES ARE TO ASCE 7-88 U.N.O.

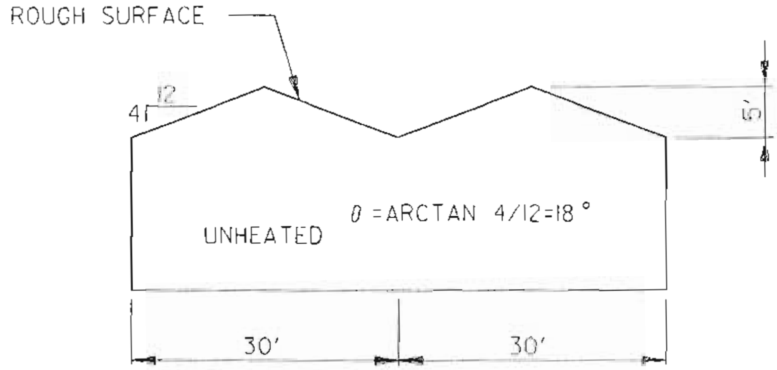
Figure H-1. Design example for snow loads - gable roof.

GIVEN: THE MULTIPLE GABLE WAREHOUSE SHOWN BELOW IS LOCATED IN A WINDY FIELD WITH A FEW BIRCH TREES PLANTED NEARBY.

PROBLEM: DETERMINE THE ROOF SNOW LOADS.

LOCATION: ANCHORAGE, ALASKA

OCCUPANCY CATEGORY I



*ALL REFERENCES IN THIS EXAMPLE ARE TO ASCE 7-88 U.N.O.

SOLUTION: FLAT ROOF SNOW LOAD

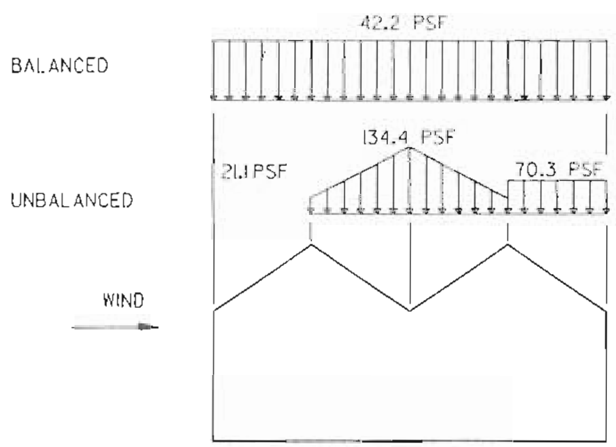
$p_f = 0.6 C_e C_t I p_g$	EQ.5b*
WHERE $C_e = 0.9$	TABLE 18
$C_t = 1.2$	TABLE 19
$I = 1.0$	TABLE 20
$p_g = 65$ PSF	APPENDIX B (THIS MANUAL)
$p_f = 0.6 \times 0.9 \times 1.2 \times 1.0 \times 65 = 42.2$ PSF	EQ.5b
(SINCE $\theta > 15°$, MIN. SNOW LOAD DOES NOT APPLY)	PARA.7.3.4

SLOPED ROOF SNOW LOAD

$p_s = C_s p_f$	EQ.6
WHERE $C_s = 1.00$	PARA.7.4.4 AND FIG.8
$p_s = 1.00 \times 42.2 = 42.2$ PSF	EQ.6

Figure H-2. Design example for snow loads - multiple gable roof. (Sheet 1 of 2)

UNBALANCED SNOW LOAD	
RIDGE $0.5P_f = 0.5 \times 42.2$ $= 21.1$ PSF	FIG.6-1 (THIS MANUAL)
VALLEY $3P_f / C_e = (3 \times 42.2)/0.9$ $= 140.7$ PSF	FIG.[illegible] (THIS MANUAL)
SNOW DENSITY $0.13(65)+14=22.4$ PCF	EQ.4
SNOW HEIGHT ABOVE RIDGE $21.2/22.4=1.0$ FT.	
CHECK SNOW HEIGHT ABOVE VALLEY $140.7/22.4=6.3>5.0+1.0=6.0$ USE 6.0 FT. HEIGHT (SAME ELEVATION AS SNOW ABOVE RIDGE)	PARA.7.6.3
VALLEY 6 FT. x 22.4 PCF=134.4 PSF	
UNBALANCED SNOW LOAD ON LEEWARD SLOPE $1.5P_s / C_e = (1.5 \times 42.2)/0.9 = 70.3$ PSF	FIG.6-1 (THIS MANUAL)

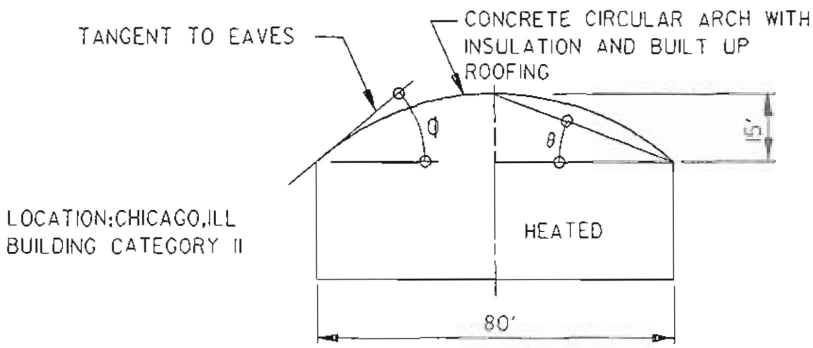


ROOF SNOW LOAD

Figure H-2. Design example for snow loads - multiple gable roof. (Sheet 2 of 2)

GIVEN: THE THEATER SHOWN BELOW HAS A CIRCULAR ARCHED ROOF. IT IS SITED IN A WINDY AREA WITH A FEW NEARBY CONIFEROUS TREES. IT IS THE TALLEST STRUCTURE IN A RECREATION COMPLEX.

PROBLEM: DETERMINE THE BALANCED AND UNBALANCED SNOW LOADS.



θ = ARCTAN 15/40=21°

*ALL REFERENCES ARE TO ASCE 7-88 U.N.O.

SOLUTION: FLAT ROOF SNOW LOAD

$p_f = 0.7 C_e C_t I p_g$	EQ.5a*
$C_e = 0.9$	TABLE 18
$C_t = 1.0$	TABLE 19
$I = 1.1$	TABLE 20
$p_g = 25$	APPENDIX B (THIS MANUAL)
$p_f = 0.7 \times 0.9 \times 1.0 \times 1.1 \times 25 = 17.3$ PSF	EQ.5a
(SINCE $\theta > 10°$, MIN p_f DOES NOT APPLY)	PARA.7.3.4

SLOPED ROOF SNOW LOAD

$p_s = C_s p_f$	EQ.6
WHERE $C_s = 1.0$	FIG.8a
$p_s = 1.0 \times 17.3 = 17.3$ PSF	

HOWEVER, USE 25 PSF FOR THE BALANCED LOAD PER FOOTNOTE 'd' OF APPENDIX B (THIS MANUAL)

Figure H-3. Design example for snow loads - arched roof. (Sheet 1 of 2)

UNBALANCED SNOW LOAD

SINCE EQUIVALENT SLOPE, θ, IS 21°	
$10° < \theta < 60°$	
UNBALANCED CONDITION APPLIES	PARA.7.6.2
SINCE SLOPE AT EAVES $(\phi)=41°$	GEOMETRY
USE CASE II	FIG.10

LOAD AT CROWN*

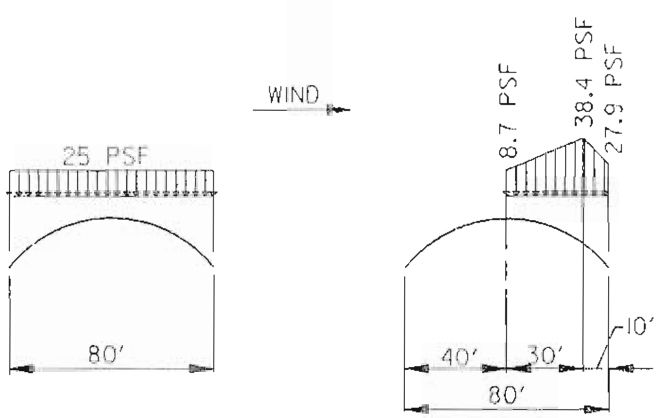
$0.5P_s = 0.5 \times 17.3 = 8.7$ PSF	FIG.10

LOAD AT 30° POINT (30 FT FROM CROWN)	GEOMETRY
$2P_s / C_e = (2 \times 17.3)/0.9 = 38.4$ PSF	FIG.10

LOAD AT EAVES*

$2P_s / C_e [1-(\phi-30°)/40°]$	FIG.10
$38.4[1-(41°-30°)/40°] = 27.9$ PSF	

*NOTE THAT 17.3 PSF, NOT 25 PSF, IS USED IN THE UNBALANCED SNOW LOAD CALCULATIONS.



BALANCED

UNBALANCED

DESIGN SNOW LOAD

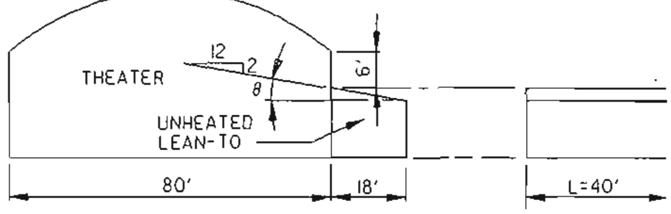
Figure H-3. Design example for snow loads - arched roof. (Sheet 2 of 2)

GIVEN: A LEAN-TO SHOWN BELOW IS ADDED TO THE THEATER IN DESIGN EXAMPLE H-3.

PROBLEM: DETERMINE SNOW LOAD ON THE ROOF OF THE LEAN-TO.

LOCATION: CHICAGO, ILL.
BUILDING CATEGORY I

θ=ARCTAN 2/12=9.5°



SOLUTION:

FLAT ROOF SNOW LOAD

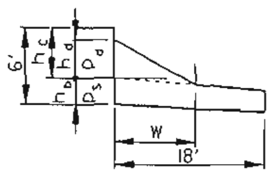
$p_f = 0.7 C_e C_t I p_g$	EQ.5a
WHERE $C_e = 0.9$	TABLE 18
$C_t = 1.2$	TABLE 19
$I = 1.0$	TABLE 20
$p_g = 25$ PSF	APPENDIX B (THIS MANUAL)
$P_f = 0.7 \times 0.9 \times 1.2 \times 1.0 \times 25 = 18.9$ PSF	EQ.5a

CHECK MINIMUM p_f WHERE $\theta < 15°$

WHEN $p_g > 20$ PSF,
MINIMUM $p_f = 20I = 20 \times 1.0 = 20.0$ PSF — PARA. 7.3.4
SINCE 18.9 PSF < 20.0 PSF,
USE 20.0 PSF
NOTE: AT THIS POINT DO NOT USE THE 25 PSF MINIMUM PER FOOTNOTE "d" OF APPENDIX B OF THIS MANUAL. COMPARE THE 25 PSF MINIMUM TO THE COMBINED LOAD AFTER IT IS CALCULATED.

SLOPED ROOF SNOW LOAD

$p_s = C_s p_f$	EQ.6
WHERE $C_s = 1.0$	FIG. 8b
$p_s = 1.0 \times 20.0 = 20.0$ PSF	EQ.6



DRIFT ON LEAN-TO

Figure H-4. Design example for snow loads - lean-to roof. (Sheet 1 of 2)

DRIFT SNOW LOAD

$\gamma = 0.13 \times 25 + 14 = 17$ PCF — EQ.4

$h_b = P_s/\gamma = 20/17 = 1.2$ FT

$h_c = 6 - h_b = 6 - 1.2 = 4.8$ FT — FIG.12

$h_c/h_b = 4.8/1.2 = 4.0 > 0.2$ — PARA.7.7.2

THEREFORE CONSIDER DRIFT LOAD

$l_u = 80$ FT. — FIG.12

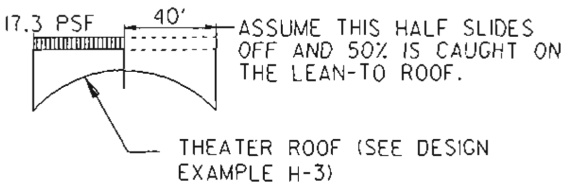
$h_d = 3.0$ FT. — FIG.13

$P_d = h_d \gamma = 3.0 \times 17 = 51.0$ PSF — PARA.7.7.2

WIDTH OF DRIFT

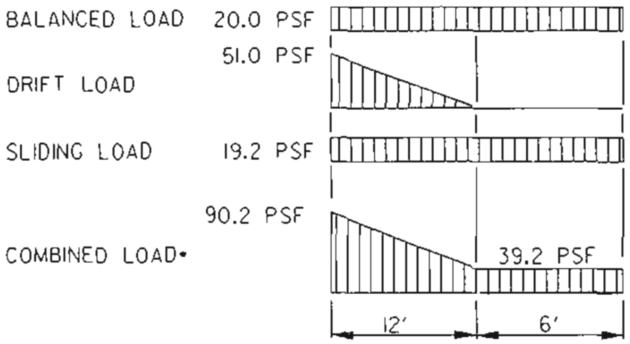
$W = 4h_d = 4 \times 3.0 = 12.0$ FT — PARA.7.7.2

SLIDING SNOW LOAD — PARA.7.9



SLIDING SNOW LOAD UNIFORMILY DISTRIBUTED ON LEAN-TO ROOF.

$(0.50 \times 17.3 \times 40)/18 = 19.2$ PSF



*NOTE: IF THE COMBINED LOAD WERE LESS THAN 25 PSF, IT WOULD BE INCREASED TO 25 PSF IN ACCORDANCE WITH FOOTNOTE "d" OF APPENDIX B.

SNOW LOAD ON LEAN-TO

Figure H-4. Design example for snow loads - lean-to roof. (Sheet 2 of 2)

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