1.0 INTRODUCTION TO STRUCTURAL ENGINEERING

1.1 GENERAL INTRODUCTION

Structural design is a systematic and iterative process that involves:

1) Identification of intended *use* and *occupancy* of a structure – by owner
2) Development of architectural plans and layout – by architect
3) Identification of structural framework – by engineer
4) Estimation of structural loads depending on use and occupancy
5) Analysis of the structure to determine member and connection design forces
6) Design of structural members and connections
7) Verification of design
8) Fabrication & Erection – by steel fabricator and contractor
9) Inspection and Approval – by state building official

Ideally, the owner and the architect, the architect and the engineer, and the engineer and the fabricator/contractor will collaborate and interact on a regular basis to conceive, develop, design, and build the structure in an efficient manner. The primary responsibilities of all these players are as follows:

- Owner - primary responsibility is deciding the use and occupancy, and approving the architectural plans of the building.
- Architect - primary responsibility is ensuring that the architectural plan of the building interior is appropriate for the intended use and the overall building is aesthetically pleasing.
- Engineer – primary responsibility is ensuring the safety and serviceability of the structure, i.e., designing the building to carry the loads safely and ___________.
- Fabricator – primary responsibility is ensuring that the designed members and connections are fabricated economically in the shop or field as required.
- Contractor/Erector - primary responsibility is ensuring that the members and connections are economically assembled in the field to build the structure.

- State Building Official – primary responsibility is ensuring that the built structure satisfies the appropriate building codes accepted by the Govt.

1.2 STRUCTURAL DESIGN

- Conceptually, from an engineering standpoint, the parameters that can be varied (somewhat) are: (1) the material of construction, and (2) the structural framing plan.
- The choices for material include: (a) *steel*, (b) reinforced concrete, and (c) steel-concrete composite construction.
- The choices for structural framing plan include moment resisting frames, braced frames, dual frames, shear wall frames, and so on. The engineer can also *innovate* a new structural framing plan for a particular structure if required.
- All viable material + framing plan alternatives must be considered and designed to compare the individual material + fabrication / erection costs to identify the most efficient and economical design for the structure.
- For each material + framing plan alternative considered, designing the structure consists of designing the individual structural components, i.e., the members and the connections, of the framing plan.
- This course ***CE470*** focuses on the design of individual structural *components*. The material of construction will limited be steel, and the structural framing plans will be limited to braced frames and moment resisting frames.

1.3 STRUCTURAL FRAMEWORK

- Figure 1 shows the structural plan and layout of a *four*-story office building to be located in Lansing. Figure 2 and 3 show the structural elevations of frames A-A and B-B, respectively, which are identified in Figure 1.

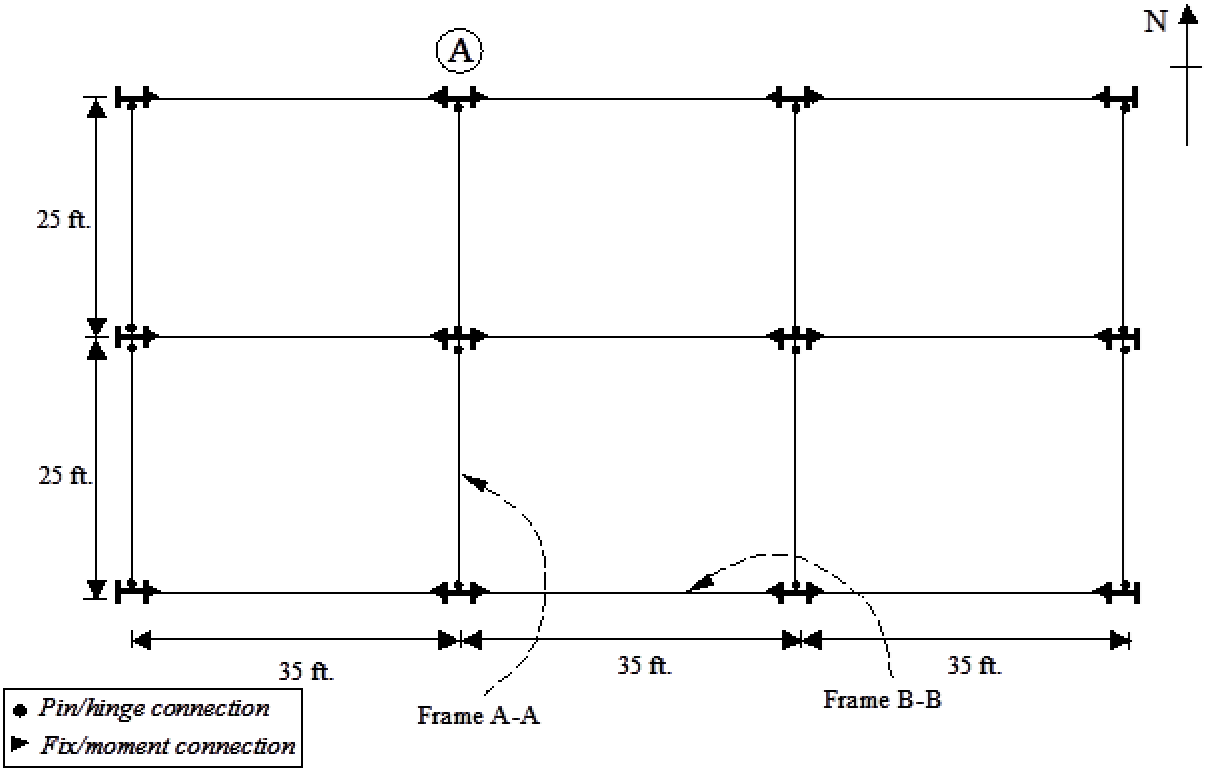


Figure 1. Structural floor plan and layout

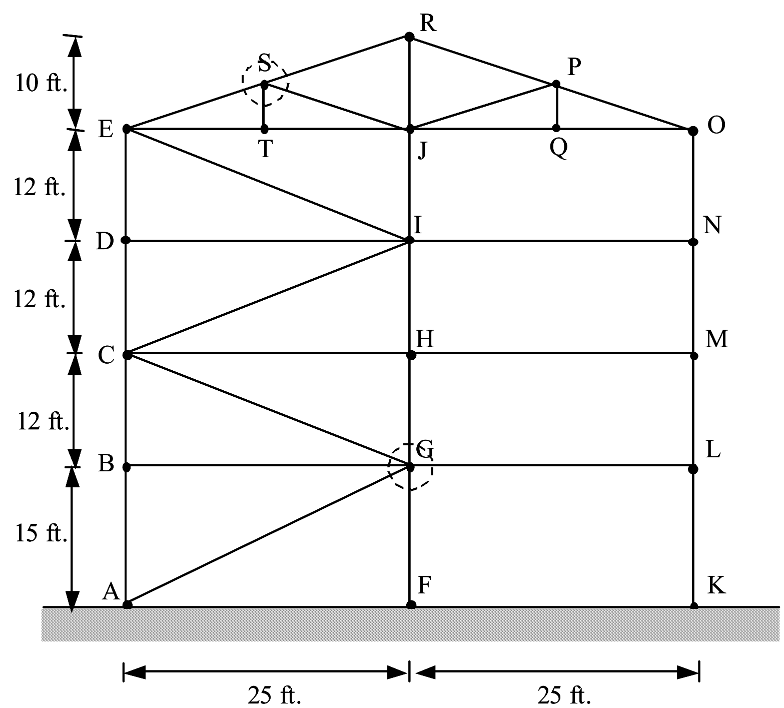


Figure 2. Structural elevation of frame A-A

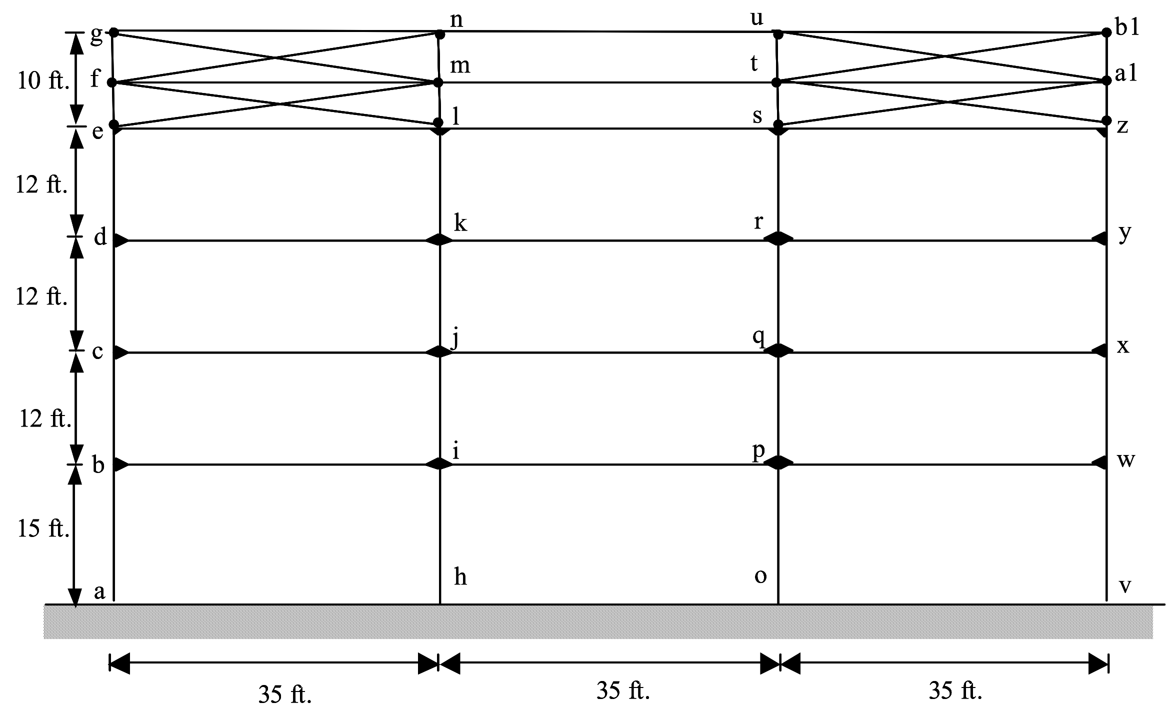


Figure 3. Structural elevation of frame B-B

- As shown in Figure 1, the building has two 25-ft. bays in the *north-south* direction and three 35 ft. bays in the *east-west* direction.
- There are *four* structural frames in the north-south direction. These frames have structural elevations similar to frame A-A shown in Figure 2.
- There are *three* structural frames in the east-west directions. These frames have structural elevations similar to frame B-B shown in Figure 3.
- The building has a *roof truss*, which is shown in Figures 2 and 3.
- Frame A-A is a braced frame, where all members are connected using *pin/hinge connections*. Diagonal bracing members are needed for stability.
- Frame B-B is a moment frame, where all members are connected using *fix/moment connections*. There is no need for diagonal bracing members.
- The north-south and east-west frames resist the *vertical gravity* loads together.
- The three moment frames in the east-west direction resist the *horizontal lateral loads* in the east-west direction.

- The four braced frames in the north-south direction resist the *horizontal lateral loads* in the north-south direction.

1.4 STRUCTURAL MEMBERS

Structural members are categorized based up on the internal forces in them. For example:

- Tension member –subjected to tensile axial force only
- Column or compression member –subjected to compressive axial force only
- Tension/Compression member –subjected to tensile/compressive axial forces
- Beam member –subjected to flexural loads, i.e., shear force and bending moment only. The axial force in a beam member is negligible.
- Beam-column member – member subjected to combined axial force and flexural loads (shear force, and bending moments)

In basic structural analysis ***(CE305)*** students have come across two types of structures, namely, *trusses and frames*. For example, Figure 2 shows a roof truss supported by a braced frame.

- All the members of a truss are connected using pin/hinge connections. All external forces are applied at the pins/hinges. As a result, all truss members are subjected to axial forces (tension or compression) only.
- In braced and moment frames, the horizontal members (beams) are subjected to flexural loads only.
- In braced frames, the vertical members (columns) are subjected to compressive axial forces only.
- In braced frames, the diagonal members (braces) are subjected to tension/compression axial forces only.
- In moment frames, the vertical members (beam-columns) are subjected to combined axial and flexural loads.

For practice, let us categorize the member shown in Figures 2 and 3.

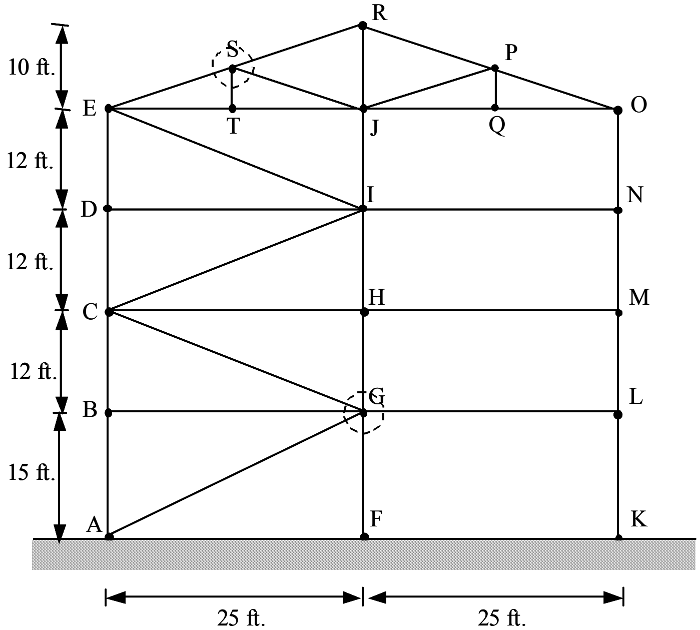


Figure 2. Structural elevation of frame A-A

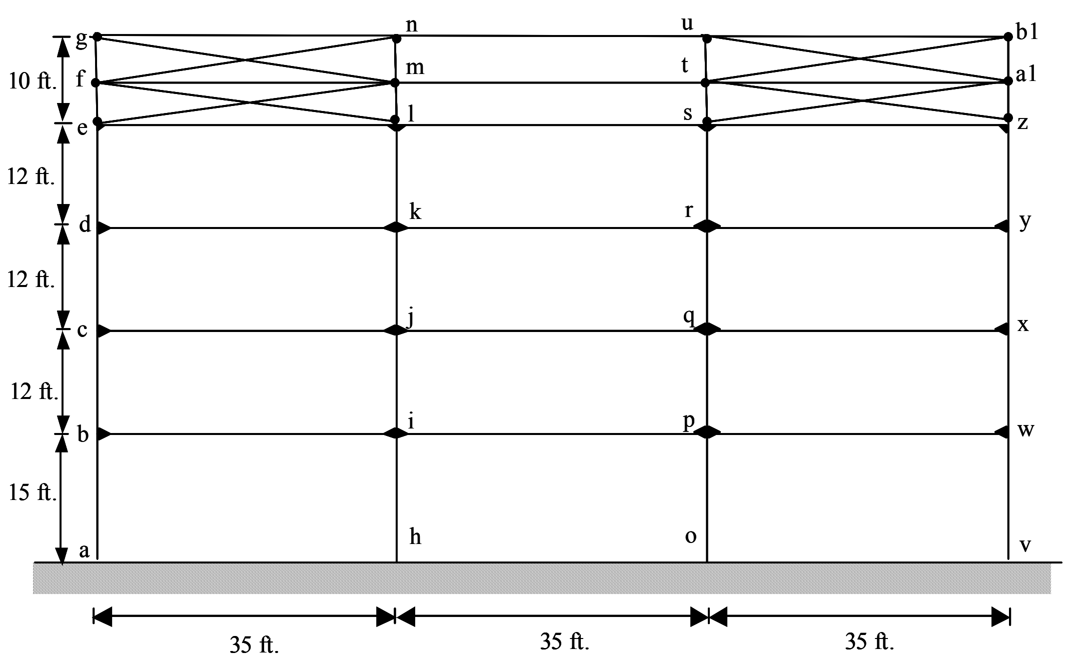


Figure 3. Structural elevation of frame B-B

1.5 STRUCTURAL CONNECTIONS

Members of a structural frame are connected together using connections. Prominent connection types include: (1) truss / bracing member connections; (2) simple shear connections; (3) fully-restrained moment connections; and (4) partially-restrained flexible moment connections.

- Truss / bracing member connections are used to connect two or more truss members together. Only the *axial forces* in the members have to be transferred through the connection for continuity.
- Simple shear connections are the *pin connections* used to connect beam to column members. Only the *shear forces* are transferred through the connection for continuity. The *bending moments* are not transferred through the connection.
- Moment connections are *fix connections* used to connect beam to column members. Both the shear forces and bending moments are transferred through the connections with very small deformations (*full restraint*).
- Partially restrained connections are *flexible connections* used to connect beam to column members. The shear forces are transferred fully through the connection. However, the bending moment is only transferred partially.

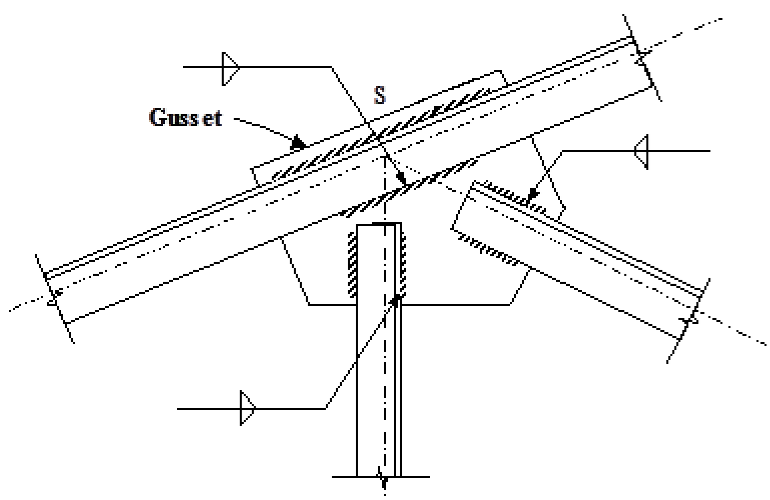


Figure 4. Truss connection at S in Frame A-A

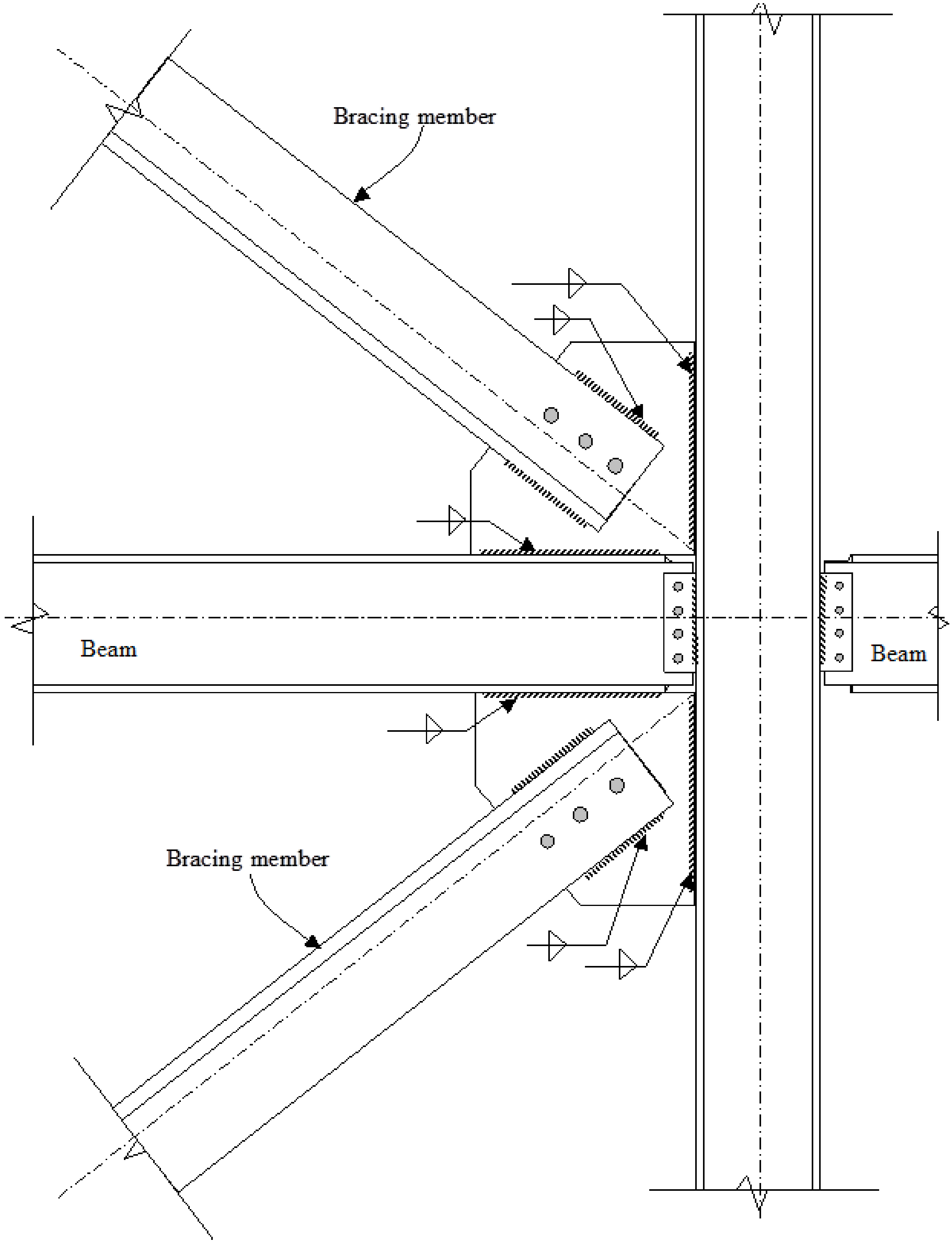


Figure 5. Bracing connection and Simple Shear Connection at **G** in Frame A-A.

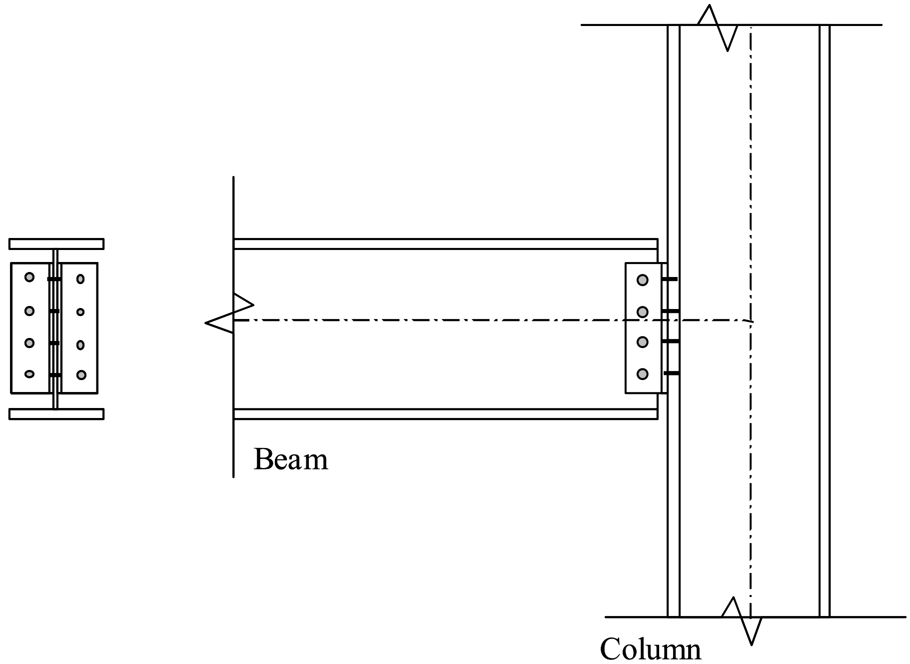


Figure 6. All-bolted double angle shear connection.

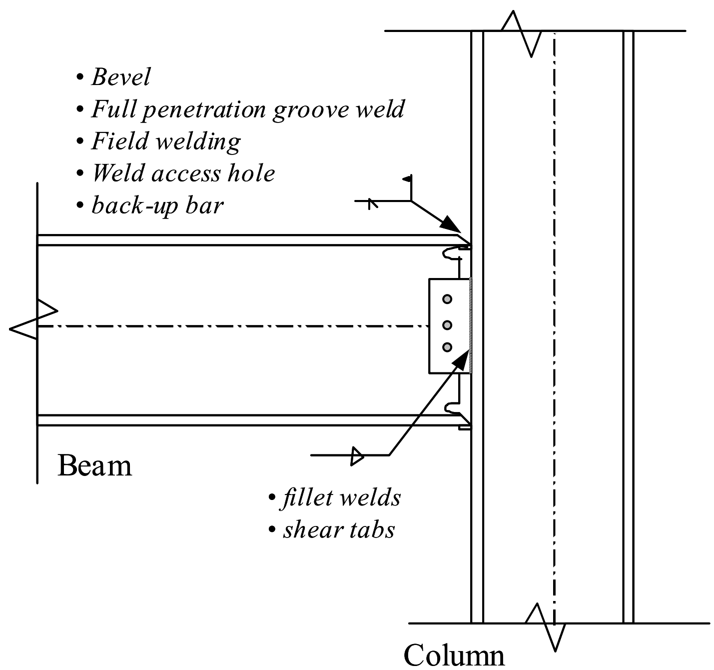
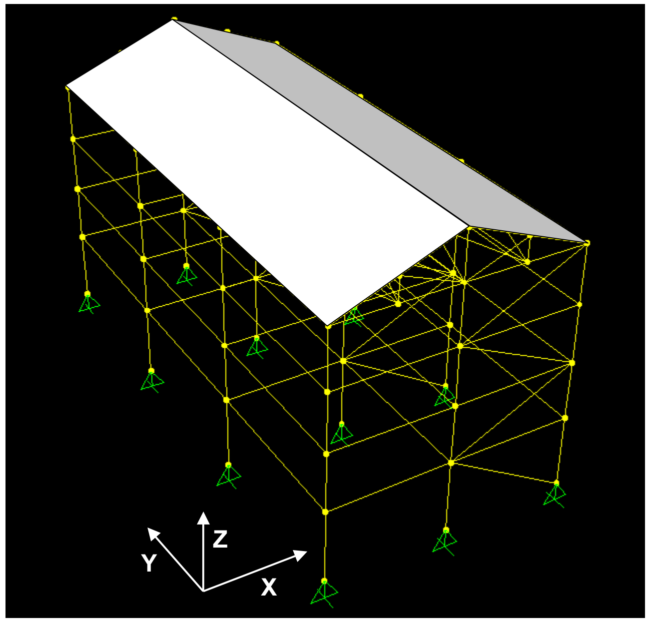


Figure 7. Directly welded flange fully restrained moment connection.

- Figure 4 shows an example truss connection. Figure 5 shows an example bracing connection. Figure 6 shows an example shear connection. Figure 7 shows an example moment connection.
- Connections are developed using bolts or welds.
- Bolts are used to connect two or more plate elements that are in the same plane. Bolt-holes are drilled in the plate elements. The *threaded* bolt shank passes through the holes, and the connection is secured using *nuts*.
- Bolts are usually made of *higher strength steel*.
- Welds can be used to connect plate elements that are in the same or different planes. A high voltage *electric arc* is developed between the two plate elements. The electric arc causes localized melting of the base metal (plate element) and the weld electrode. After cooling, all the molten metal (base and weld) solidifies into one *continuum*. Thus, developing a welded connection.
- In Figure 4, all the truss members are connected together by welding to a common *gusset* plate. The axial forces in the members are transferred through the *gusset* plates. This same connection can also be developed using bolts. *How?*
- In Figure 5, the bracing members are connected to *gusset* plates, which are also connected to the beam and column. The bracing member can be connected to the *gusset* plate using bolts or welds. However, the *gusset* plate has to be welded to the beam / column.
- In Figure 6, two angles are bolted to the web of the beam. The perpendicular legs of the angles are bolted to the flange of the column. Thus, an all-bolted double-angle shear connection is achieved. This all-bolted connection will be easier to assemble in the field as compared to welding. *How is this a shear connection?*
- In Figure 7, the beam flanges are *beveled* and welded directly to the flange of column using <u>full penetration</u> groove welds. This welding will have to be done in the *field* during erection and it will require the use of back-up bars. Weld-access holes and skilled welders are required to achieve a weld of acceptable quality.
- In Figure 7, the beam web is bolted to a shear tab (plate), which is fillet welded to the column in the shop. This shear tab connection transfers the shear from the beam to the column. *How is Figure 7 a moment connection?*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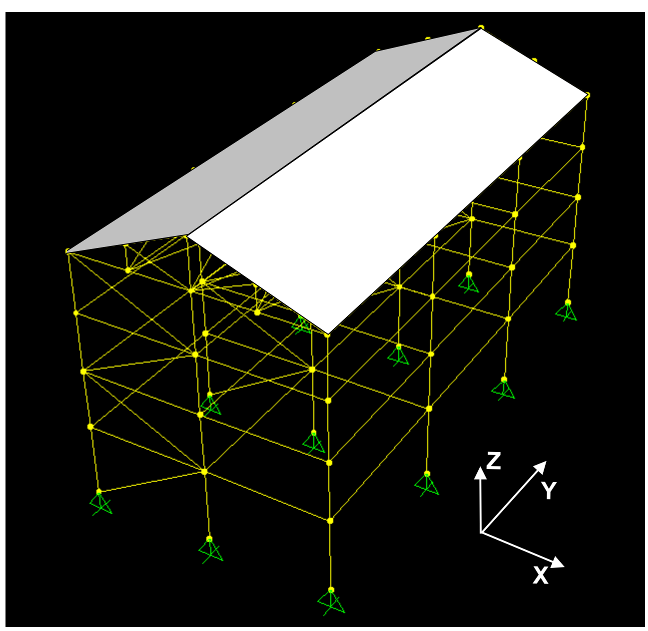
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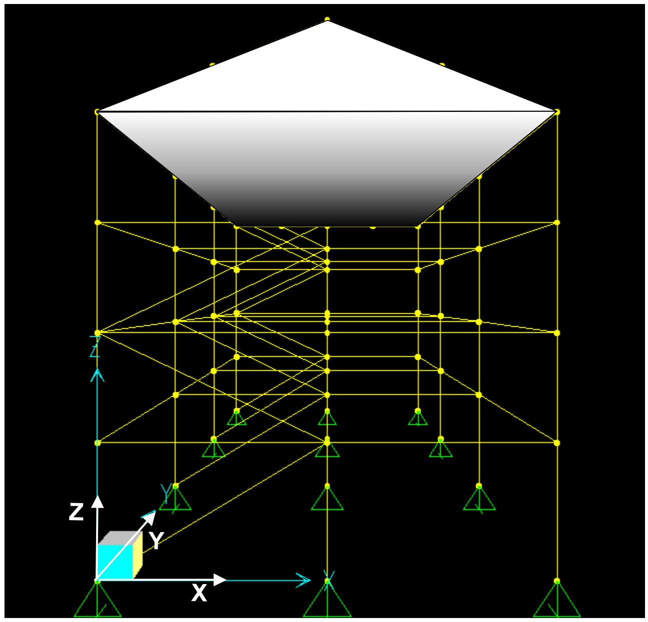
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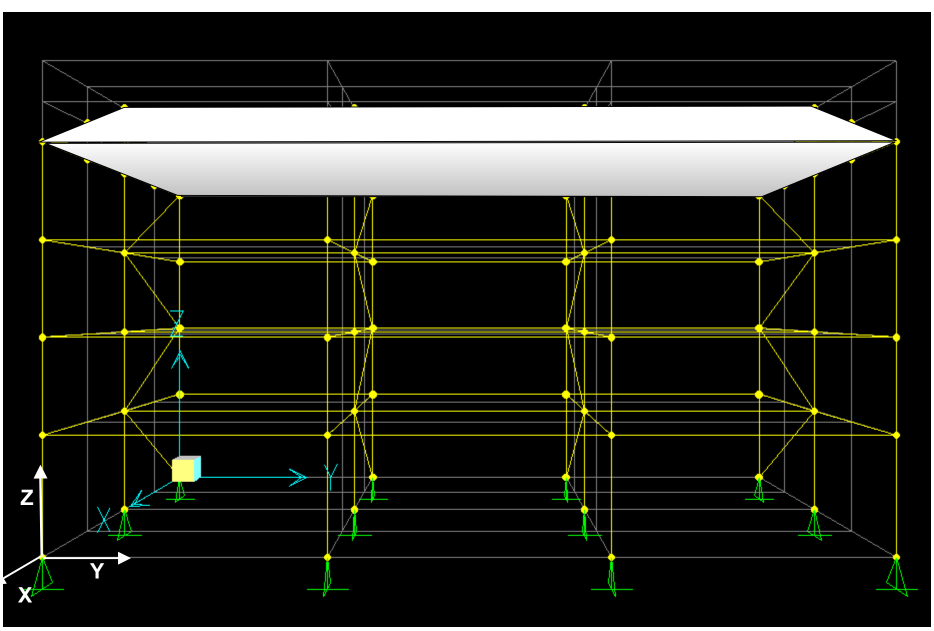
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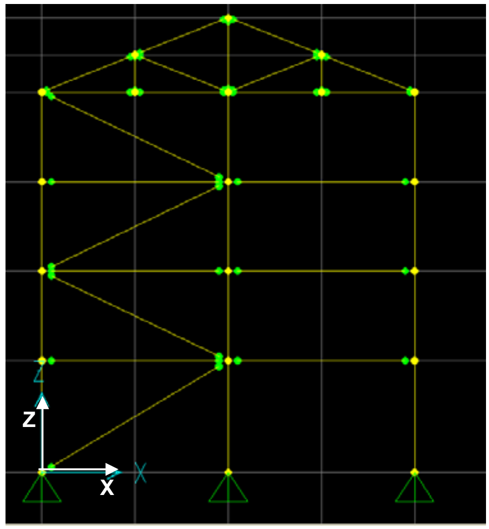
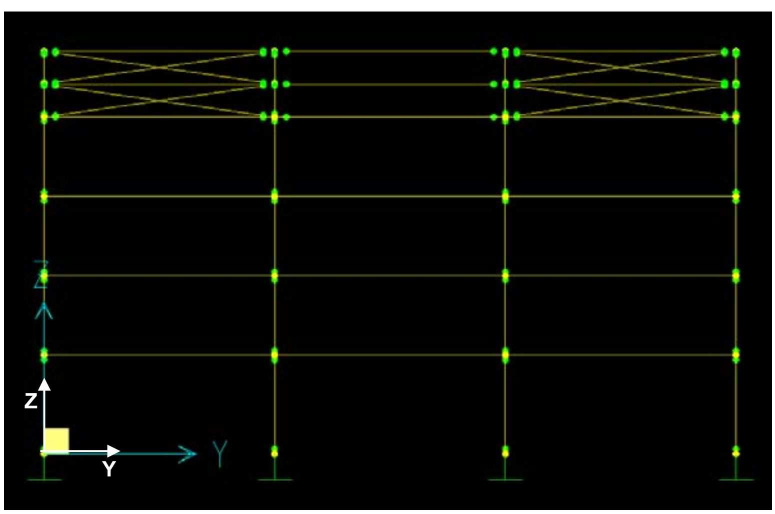
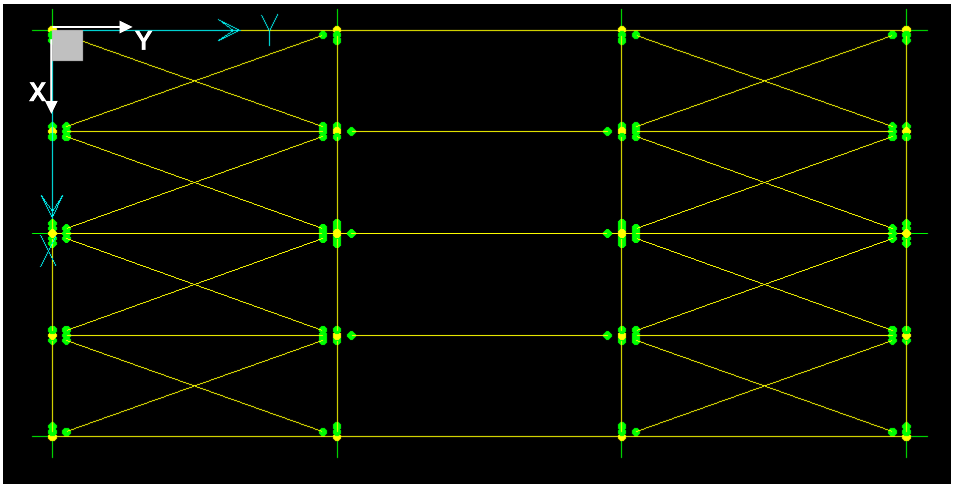


ELEVATION OF BRACED FRAME

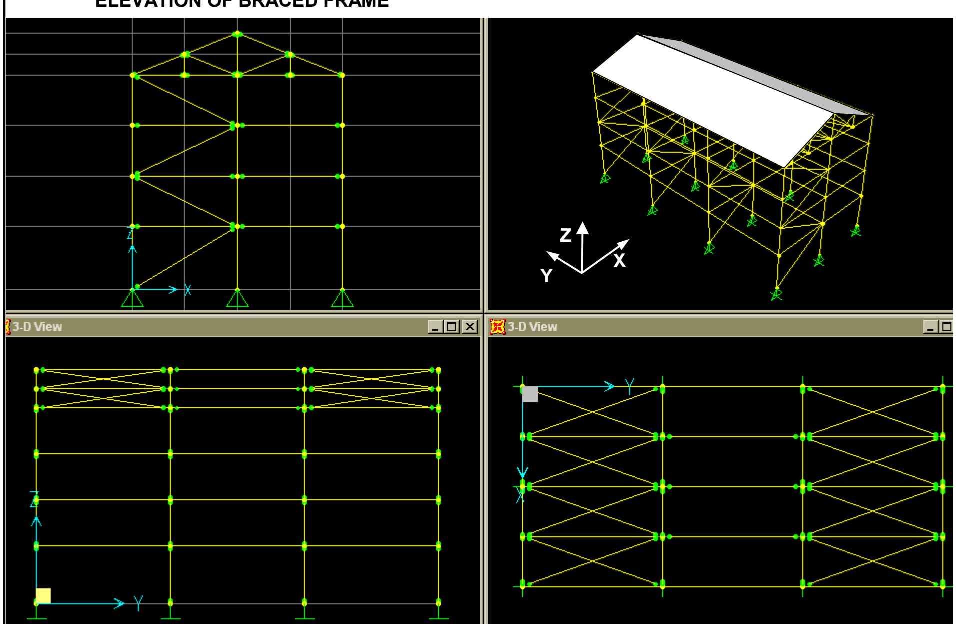


ELEVATION OF MOMENT FRAME

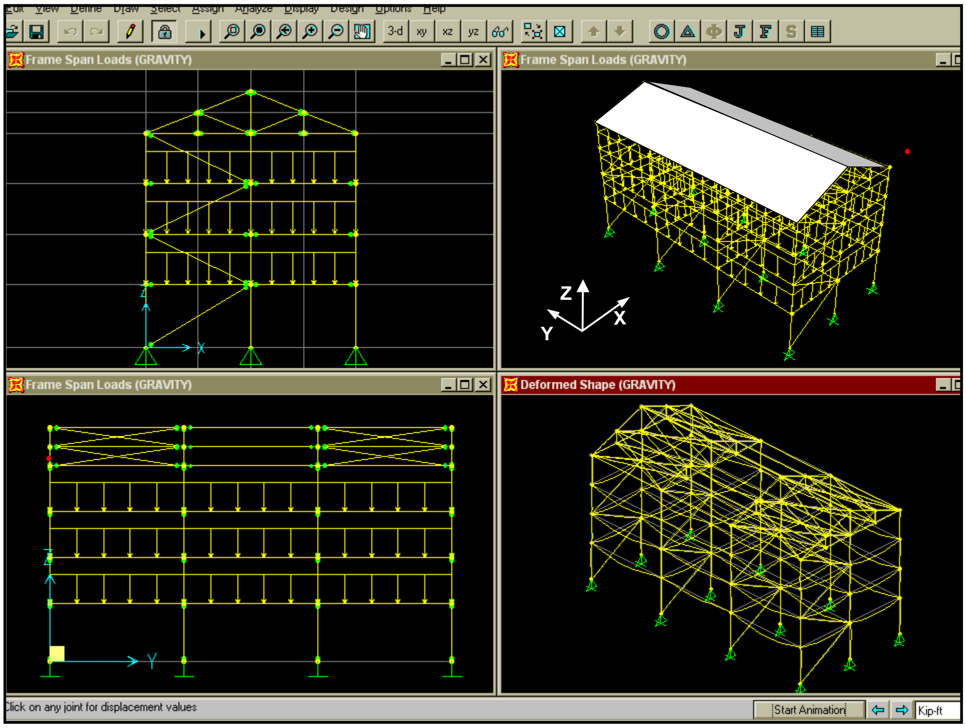


PLAN OF ROOF TRUSS

ELEVATION OF BRACED FRAME



ELEVATION OF MOMENT FRAME **PLAN OF ROOF TRUSS**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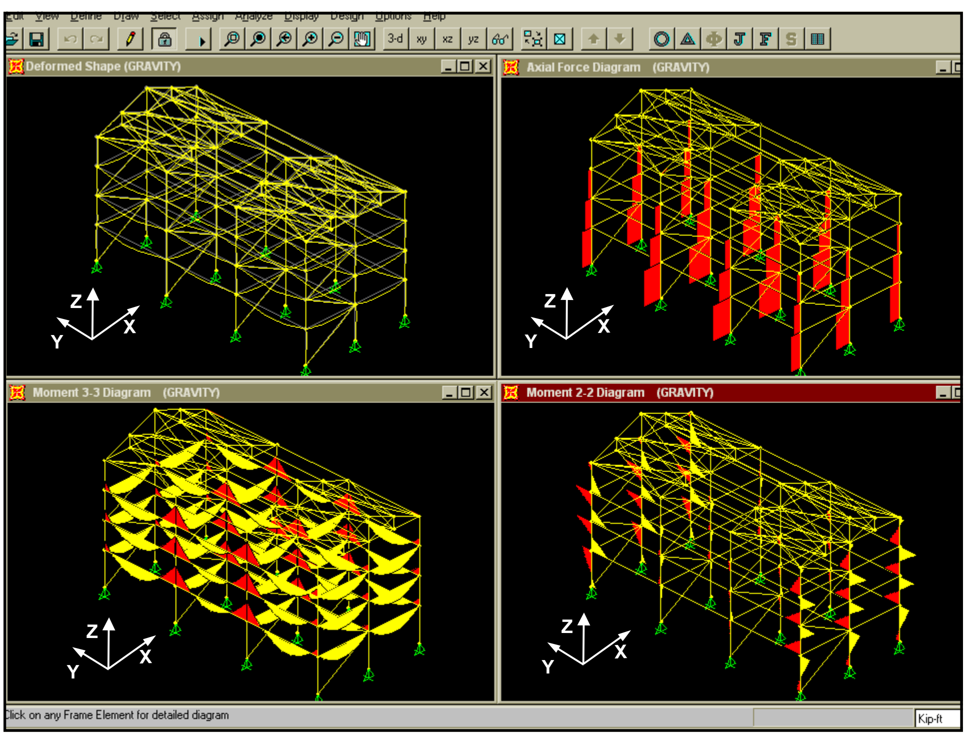
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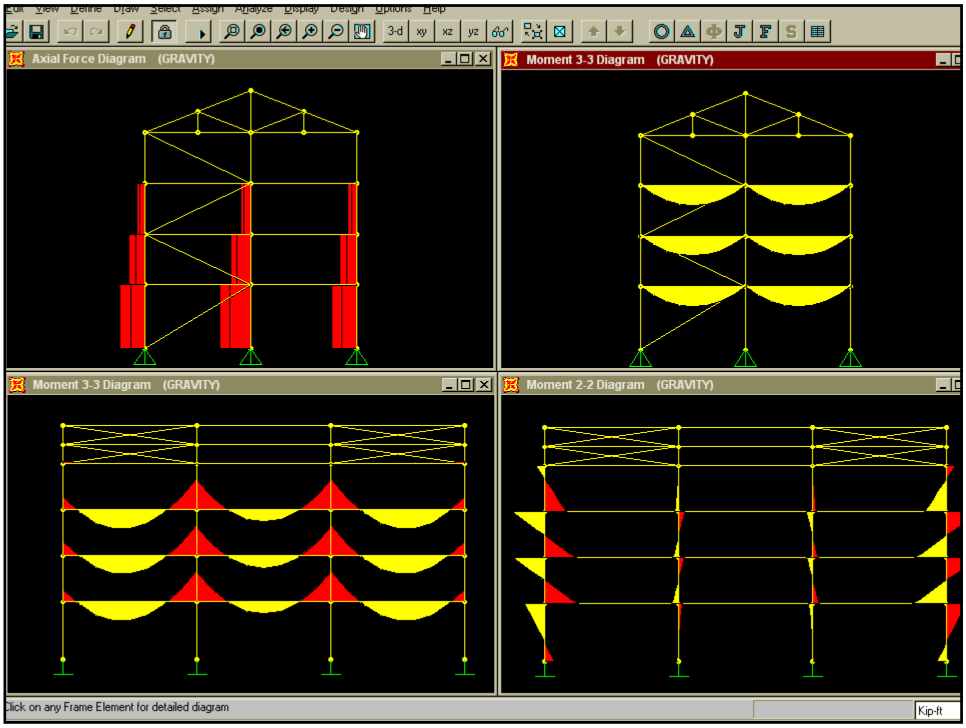
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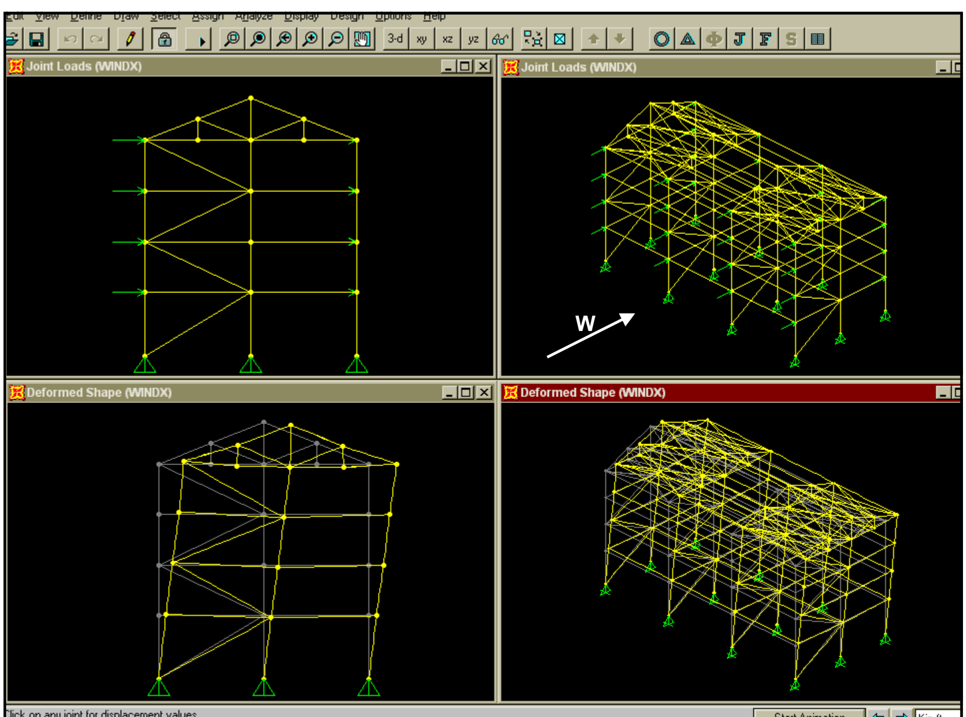
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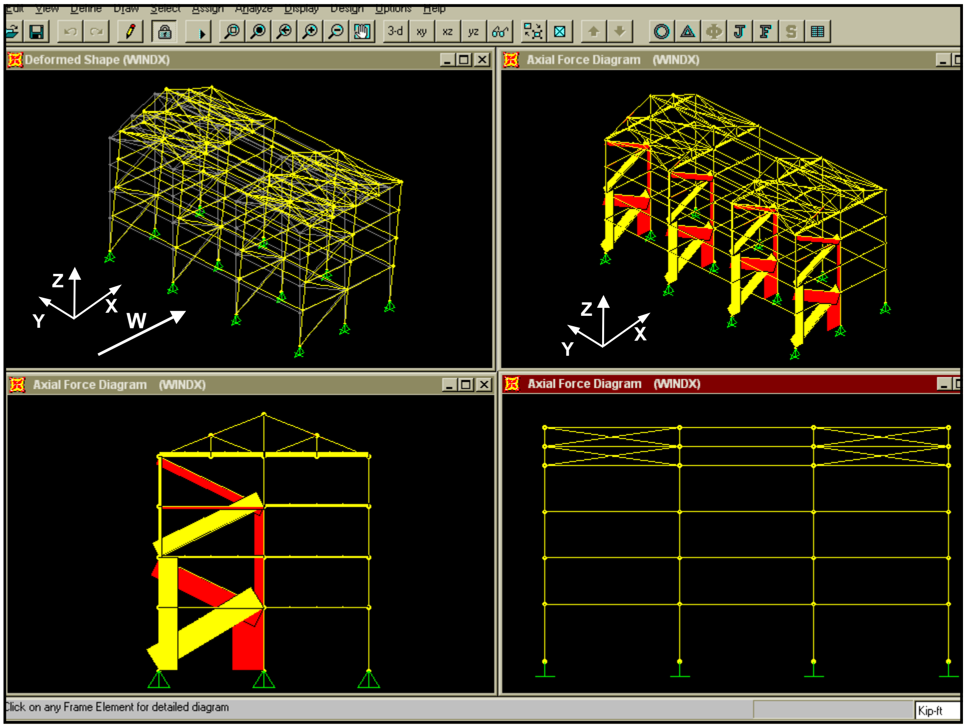
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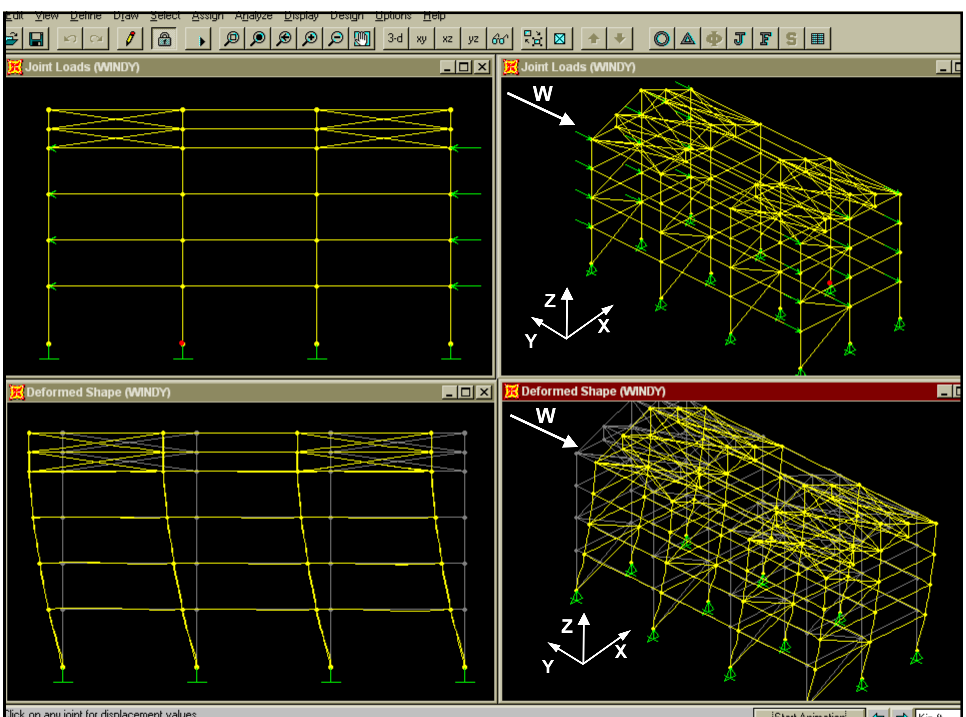
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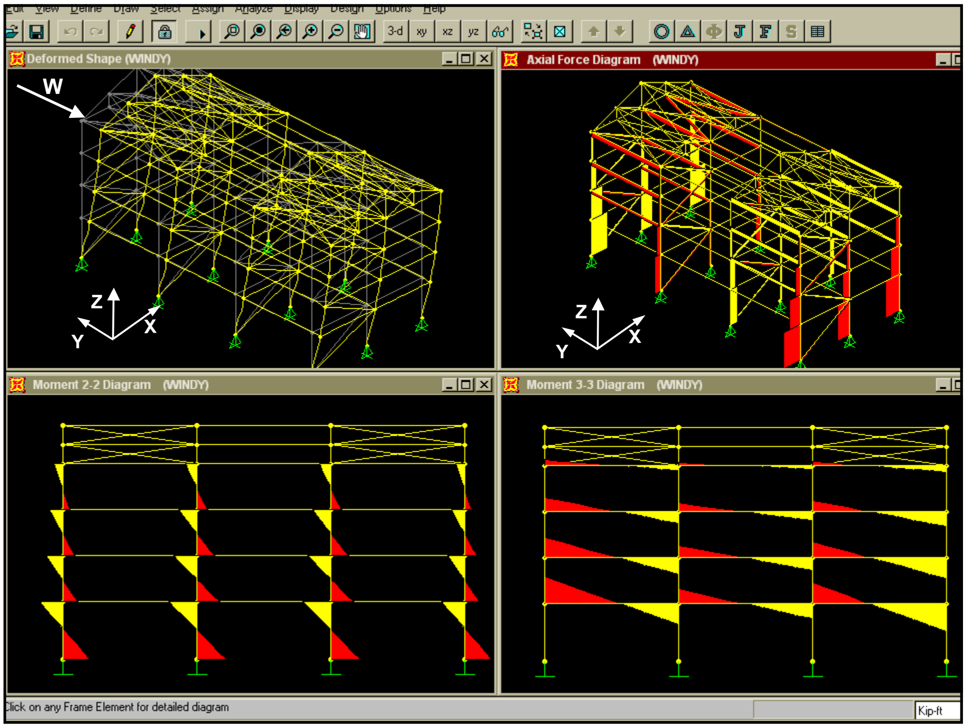
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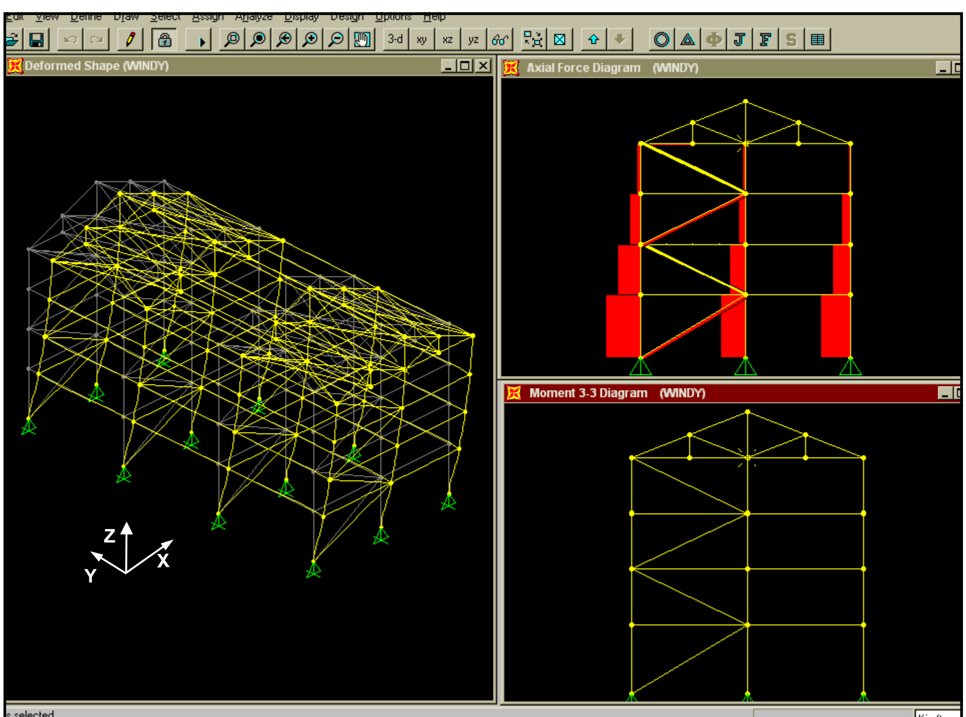
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1.6 Structural Loads

The building structure must be designed to carry or resist the loads that are applied to it over its design-life. The building structure will be subjected to loads that have been categorized as follows:

- Dead Loads (*D*): are permanent loads acting on the structure. These include the self-weight of structural and non-structural components. They are usually *gravity* loads.
- Live Loads (*L*): are non-permanent loads acting on the structure due to its use and occupancy. The magnitude and location of live loads changes frequently over the design life. Hence, they cannot be estimated with the same accuracy as dead loads.
- Wind Loads (*W*): are in the form of *pressure* or *suction* on the exterior surfaces of the building. They cause horizontal lateral loads (forces) on the structure, which can be critical for tall buildings. Wind loads also cause *uplift* of light roof systems.
- Snow Loads (*S*): are vertical gravity loads due to snow, which are subjected to variability due to seasons and drift.
- Roof Live Load (L_r): are live loads on the roof caused during the design life by planters, people, or by workers, equipment, and materials during maintenance.
- Values of structural loads are given in the publication ASCE/SEI 7-10: *Minimum Design Loads for Buildings and Other Structures.* The first phase of structural design consists of estimating the loads acting on the structure. This is done using the load values and combinations presented in ASCE/SEI 7-10 as explained in the following sub-sections.

1.6.1 Step I. Categorization of Buildings

- Categories I, II, III, and IV. See Table 1.5-1 below and in ASCE/SEI 7-10.

Table 1.5-1 Risk Category of Buildings and Other Structures for Flood, Wind, Snow, Earthquake, and Ice Loads

Use or Occupancy of Buildings and Structures	Risk Category
Buildings and other structures that represent a low risk to human life in the event of failure	I
All buildings and other structures except those listed in Risk Categories I, III, and IV	II
Buildings and other structures, the failure of which could pose a substantial risk to human life. Buildings and other structures, not included in Risk Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure. Buildings and other structures not included in Risk Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing toxic or explosive substances where their quantity exceeds a threshold quantity established by the authority having jurisdiction and is sufficient to pose a threat to the public if released.	III
Buildings and other structures designated as essential facilities. Buildings and other structures, the failure of which could pose a substantial hazard to the community. Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing sufficient quantities of highly toxic substances where the quantity exceeds a threshold quantity established by the authority having jurisdiction to be dangerous to the public if released and is sufficient to pose a threat to the public if released.[a] Buildings and other structures required to maintain the functionality of other Risk Category IV structures.	IV

[a]Buildings and other structures containing toxic, highly toxic, or explosive substances shall be eligible for classification to a lower Risk Category if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.5.2 that a release of the substances is commensurate with the risk associated with that Risk Category.

1.6.2 Dead Loads (D)

Dead loads consist of the weight of all materials of construction incorporated into the building including but not limited to walls, floors, roofs, ceilings, stairways, built-in partitions, finishes, cladding and other similarly incorporated architectural and structural items, and fixed service equipment such as plumbing stacks and risers, electrical feeders, and heating, ventilating, and air conditioning systems.

In some cases, the structural dead load can be estimated satisfactorily from simple formulas based in the weights and sizes of similar structures. For example, the average weight of steel framed buildings is 60-75 lb/ft^2, and the average weight for reinforced concrete buildings is 110 - 130 lb/ft^2.

From an engineering standpoint, once the materials and sizes of the various components of the structure are determined, their weights can be found from tables that list their densities. See Tables 1.2 and 1.3, which are taken from Hibbeler, R.C. (1999)*, Structural Analysis,* 4th Edition.

Table 1–2 Minimum Densities for Design Loads from Materials*

	lb/ft^3	kN/m^3
Aluminum	170	26.7
Concrete, plain cinder	108	17.0
Concrete, plain stone	144	22.6
Concrete, reinforced cinder	111	17.4
Concrete, reinforced stone	150	23.6
Clay, dry	63	9.9
Clay, damp	110	17.3
Sand and gravel, dry, loose	100	15.7
Sand and gravel, wet	120	18.9
Masonry, lightweight solid concrete	105	16.5
Masonry, normal weight	135	21.2
Plywood	36	5.7
Steel, cold-drawn	492	77.3
Wood, Douglas Fir	34	5.3
Wood, Southern Pine	37	5.8
Wood, spruce	29	4.5

Table 1–3 Minimum Design Dead Loads*

Walls	psf	kN/m^2
4-in. (102 mm) clay brick	39	1.87
8-in. (203 mm) clay brick	79	3.78
12-in. (305 mm) clay brick	115	5.51
Frame Partitions and Walls		
Exterior stud walls with brick veneer	48	2.30
Windows, glass, frame and sash	8	0.38
Wood studs 2 × 4, (51 × 102) unplastered	4	0.19
Wood studs 2 × 4, (51 × 102) plastered one side	12	0.57
Wood studs 2 × 4, (51 × 102) plastered two sides	20	0.96
Floor Fill		
Cinder concrete, per inch (mm)	9	0.017
Lightweight concrete, plain, per inch (mm)	8	0.015
Stone concrete, per inch (mm)	12	0.023
Ceilings		
Acoustical fiberboard	1	0.05
Plaster on tile or concrete	5	0.24
Suspended metal lath and gypsum plaster	10	0.48
Asphalt shingles	2	0.10
Fiberboard, $\frac{1}{2}$-in. (13 mm)	0.75	0.04

*Reproduced with permission from American Society of Civil Engineers *Minimum Design Loads for Buildings and Other Structures*, ANSI/ASCE 7-95. Copies of this standard may be purchased from ASCE at 345 East 47th Street, New York, N.Y. 10017-2398.

1.6.3 Live Loads

- Building floors are usually subjected to uniform live loads or concentrated live loads. They have to be designed to safely support the *minimum uniformly distributed load* or the *minimum concentrated live load* values given in the ASCE/SEI 7-10 (see Table 4.1 below), whichever produces the maximum load effects in the structural members.

Table 4-1 Minimum Uniformly Distributed Live Loads, L_o, and Minimum Concentrated Live Loads

Occupancy or Use	Uniform psf (kN/m²)	Conc. lb (kN)
Apartments (see Residential)		
Access floor systems		
Office use	50 (2.4)	2,000 (8.9)
Computer use	100 (4.79)	2,000 (8.9)
Armories and drill rooms	150 (7.18)[a]	
Assembly areas and theaters		
Fixed seats (fastened to floor)	60 (2.87)[a]	
Lobbies	100 (4.79)[a]	
Movable seats	100 (4.79)[a]	
Platforms (assembly)	100 (4.79)[a]	
Stage floors	150 (7.18)[a]	
Balconies and decks	1.5 times the live load for the occupancy served. Not required to exceed 100 psf (4.79 kN/m²)	
Catwalks for maintenance access	40 (1.92)	300 (1.33)
Corridors		
First floor	100 (4.79)	
Other floors, same as occupancy served except as indicated		
Dining rooms and restaurants	100 (4.79)[a]	
Dwellings (see Residential)		
Elevator machine room grating (on area of 2 in. by 2 in. (50 mm by 50 mm))		300 (1.33)
Finish light floor plate construction (on area of 1 in. by 1 in. (25 mm by 25 mm))		200 (0.89)
Fire escapes	100 (4.79)	
On single-family dwellings only	40 (1.92)	
Fixed ladders	See Section 4.5	
Garages		
Passenger vehicles only	40 (1.92)[a,b,c]	
Trucks and buses	[c]	
Handrails, guardrails, and grab bars	See Section 4.5	
Helipads	60 (2.87)[d,e] Nonreducible	[e,f,g]
Hospitals		
Operating rooms, laboratories	60 (2.87)	1,000 (4.45)
Patient rooms	40 (1.92)	1,000 (4.45)
Corridors above first floor	80 (3.83)	1,000 (4.45)
Hotels (see Residential)		
Libraries		
Reading rooms	60 (2.87)	1,000 (4.45)
Stack rooms	150 (7.18)[a,h]	1,000 (4.45)
Corridors above first floor	80 (3.83)	1,000 (4.45)
Manufacturing		
Light	125 (6.00)[a]	2,000 (8.90)
Heavy	250 (11.97)[a]	3,000 (13.40)

Continued

Table 4-1 (*Continued*)

Occupancy or Use	Uniform psf (kN/m²)	Conc. lb (kN)
Office buildings		
File and computer rooms shall be designed for heavier loads based on anticipated occupancy		
Lobbies and first-floor corridors	100 (4.79)	2,000 (8.90)
Offices	50 (2.40)	2,000 (8.90)
Corridors above first floor	80 (3.83)	2,000 (8.90)
Penal institutions		
Cell blocks	40 (1.92)	
Corridors	100 (4.79)	
Recreational uses		
Bowling alleys, poolrooms, and similar uses	75 (3.59)[a]	
Dance halls and ballrooms	100 (4.79)[a]	
Gymnasiums	100 (4.79)[a]	
Reviewing stands, grandstands, and bleachers	100 (4.79)[a,k]	
Stadiums and arenas with fixed seats (fastened to the floor)	60 (2.87)[a,k]	
Residential		
One- and two-family dwellings		
Uninhabitable attics without storage	10 (0.48)[l]	
Uninhabitable attics with storage	20 (0.96)[m]	
Habitable attics and sleeping areas	30 (1.44)	
All other areas except stairs	40 (1.92)	
All other residential occupancies		
Private rooms and corridors serving them	40 (1.92)	
Public rooms[a] and corridors serving them	100 (4.79)	
Roofs		
Ordinary flat, pitched, and curved roofs	20 (0.96)[n]	
Roofs used for roof gardens	100 (4.79)	
Roofs used for assembly purposes	Same as occupancy served	
Roofs used for other occupancies	[o]	[o]
Awnings and canopies		
Fabric construction supported by a skeleton structure	5 (0.24) nonreducible	300 (1.33) applied to skeleton structure
Screen enclosure support frame	5 (0.24) nonreducible and applied to the roof frame members only, not the screen	200 (0.89) applied to supporting roof frame members only
All other construction	20 (0.96)	
Primary roof members, exposed to a work floor		
Single panel point of lower chord of roof trusses or any point along primary structural members supporting roofs over manufacturing, storage warehouses, and repair garages		2,000 (8.9)
All other primary roof members		300 (1.33)
All roof surfaces subject to maintenance workers		300 (1.33)
Schools		
Classrooms	40 (1.92)	1,000 (4.45)
Corridors above first floor	80 (3.83)	1,000 (4.45)
First-floor corridors	100 (4.79)	1,000 (4.45)
Scuttles, skylight ribs, and accessible ceilings		200 (0.89)
Sidewalks, vehicular driveways, and yards subject to trucking	250 (11.97)[a,p]	8,000 (35.60)[q]
Stairs and exit ways	100 (4.79)	300[r]
One- and two-family dwellings only	40 (1.92)	300[r]

Table 4-1 (*Continued*)

Occupancy or Use	Uniform psf (kN/m²)	Conc. lb (kN)
Storage areas above ceilings	20 (0.96)	
Storage warehouses (shall be designed for heavier loads if required for anticipated storage)		
Light	125 (6.00)[a]	
Heavy	250 (11.97)[a]	
Stores		
Retail		
First floor	100 (4.79)	1,000 (4.45)
Upper floors	75 (3.59)	1,000 (4.45)
Wholesale, all floors	125 (6.00)[a]	1,000 (4.45)
Vehicle barriers	See Section 4.5	
Walkways and elevated platforms (other than exit ways)	60 (2.87)	
Yards and terraces, pedestrian	100 (4.79)[a]	

[a]Live load reduction for this use is not permitted by Section 4.7 unless specific exceptions apply.

[b]Floors in garages or portions of a building used for the storage of motor vehicles shall be designed for the uniformly distributed live loads of Table 4-1 or the following concentrated load: (1) for garages restricted to passenger vehicles accommodating not more than nine passengers, 3,000 lb (13.35 kN) acting on an area of 4.5 in. by 4.5 in. (114 mm by 114 mm); and (2) for mechanical parking structures without slab or deck that are used for storing passenger vehicles only, 2,250 lb (10 kN) per wheel.

[c]Design for trucks and buses shall be per AASHTO LRFD Bridge Design Specifications; however, provisions for fatigue and dynamic load allowance are not required to be applied.

[d]Uniform load shall be 40 psf (1.92 kN/m²) where the design basis helicopter has a maximum take-off weight of 3,000 lbs (13.35 kN) or less. This load shall not be reduced.

[e]Labeling of helicopter capacity shall be as required by the authority having jurisdiction.

[f]Two single concentrated loads, 8 ft (2.44 m) apart shall be applied on the landing area (representing the helicopter's two main landing gear, whether skid type or wheeled type), each having a magnitude of 0.75 times the maximum take-off weight of the helicopter and located to produce the maximum load effect on the structural elements under consideration. The concentrated loads shall be applied over an area of 8 in. by 8 in. (200 mm by 200 mm) and shall not be concurrent with other uniform or concentrated live loads.

[g]A single concentrated load of 3,000 lbs (13.35 kN) shall be applied over an area 4.5 in. by 4.5 in. (114 mm by 114 mm), located so as to produce the maximum load effects on the structural elements under consideration. The concentrated load need not be assumed to act concurrently with other uniform or concentrated live loads.

[h]The loading applies to stack room floors that support nonmobile, double-faced library book stacks subject to the following limitations: (1) The nominal book stack unit height shall not exceed 90 in. (2,290 mm); (2) the nominal shelf depth shall not exceed 12 in. (305 mm) for each face; and (3) parallel rows of double-faced book stacks shall be separated by aisles not less than 36 in. (914 mm) wide.

[k]In addition to the vertical live loads, the design shall include horizontal swaying forces applied to each row of the seats as follows: 24 lb per linear ft of seat applied in a direction parallel to each row of seats and 10 lb per linear ft of seat applied in a direction perpendicular to each row of seats. The parallel and perpendicular horizontal swaying forces need not be applied simultaneously.

[l]Uninhabitable attic areas without storage are those where the maximum clear height between the joist and rafter is less than 42 in. (1,067 mm), or where there are not two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 in. (1,067 mm) in height by 24 in. (610 mm) in width, or greater, within the plane of the trusses. This live load need not be assumed to act concurrently with any other live load requirement.

[m]Uninhabitable attic areas with storage are those where the maximum clear height between the joist and rafter is 42 in. (1,067 mm) or greater, or where there are two or more adjacent trusses with web configurations capable of accommodating an assumed rectangle 42 in. (1,067 mm) in height by 24 in. (610 mm) in width, or greater, within the plane of the trusses. At the trusses, the live load need only be applied to those portions of the bottom chords where both of the following conditions are met:

i. The attic area is accessible from an opening not less than 20 in. (508 mm) in width by 30 in. (762 mm) in length that is located where the clear height in the attic is a minimum of 30 in. (762 mm); and
ii. The slope of the truss bottom chord is no greater than 2 units vertical to 12 units horizontal (9.5% slope).

The remaining portions of the bottom chords shall be designed for a uniformly distributed nonconcurrent live load of not less than 10 lb/ft² (0.48 kN/m²).

[n]Where uniform roof live loads are reduced to less than 20 lb/ft² (0.96 kN/m²) in accordance with Section 4.8.1 and are applied to the design of structural members arranged so as to create continuity, the reduced roof live load shall be applied to adjacent spans or to alternate spans, whichever produces the greatest unfavorable load effect.

[o]Roofs used for other occupancies shall be designed for appropriate loads as approved by the authority having jurisdiction.

[p]Other uniform loads in accordance with an approved method, which contains provisions for truck loadings, shall also be considered where appropriate.

[q]The concentrated wheel load shall be applied on an area of 4.5 in. by 4.5 in. (114 mm by 114 mm).

[r]Minimum concentrated load on stair treads (on area of 2 in. by 2 in. [50 mm by 50 mm]) is to be applied nonconcurrent with the uniform load.

- The minimum uniformly distributed live loads (L_o) given in Table 4.1 above can be reduced for buildings with *very large floor areas,* because it is unlikely that the prescribed live load will occur simultaneously throughout the entire structure.

- Equation (1.1) can be used to calculate the reduce uniformly distributed live load (L) for members with $K_{LL}A_T \geq 400\ ft^2$:

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) \qquad (1.1)$$

where, A_T is the tributary area in ft^2 and K_{LL} is the live load element factor as follows:

K_{LL} is equal to 4.0 for interior columns and exterior columns without cantilever slabs. K_{LL} is equal to 3.0 for edge columns with cantilever slabs.

K_{LL} is equal to 2.0 for corner columns with cantilever slabs, edge beams without cantilever slabs, and interior beams.

K_{LL} is equal to 1.0 for all other members not identified above.

EXCEPTION: Equation 1.1(a) can be used instead of Equation 1.1 for members of one and two-family structures supporting more than one floor load.

$$L = 0.7 \times (L_{o1} + L_{o2} + ...) \qquad (1.1(a))$$

L_{o1}, L_{o2} … unreduced floor live load applicable to each of the supported story level irrespective of the tributary area.

L ≥ the largest unreduced floor live load on a given story level acting alone

- Some limitations to the live load reduction are as follows:

L cannot be less than $0.5L_o$ for members supporting one floor and L cannot be less that $0.4L_o$ for members supporting two or more floors.

Live loads that exceed 100 lb/ft^2 shall not be reduced except the live loads for members supporting two or more floors may be reduced by 20%.

Live loads shall not be reduced for passenger vehicle garages except the live loads for members supporting two or more floors may be reduced by 20%.

Live loads shall not be reduced in assembly uses.

Example 1.1 Determine the magnitude and distribution of live loading on the north-south frame b_i - e_i - h_i

NOTES:

1) The live loading on the beams b_i - e_i and $e_i - h_i$ can be calculated using the reduced floor live load and the tributary area for the beams supporting the floors.
2) The live loads acting on beams $d_i - e_i$ and e_i - f_i can be used to determine the concentrated live load reactions on columns d_i, e_i, and f_i
   - Where, the live loads acting on the beams $d_i - e_i$ and e_i - f_i are calculated using the reduced floor live load and the corresponding tributary area for the beams
3) The concentrated live load acting on the columns can also be estimated directly using the reduced live load and the tributary area for the columns.
   - But, this method would be inconsistent because the live load carried by the beams b_i-e_i and e_i-h_i would be included twice.
   - Additionally, the live load reduction factor calculated directly for the columns will be different from the live load reduction factors calculated for the beams. Consider the Tables developed in this example.

- **Step I:** Determine relevant tributary and influence areas. Estimate live load reduction factors.

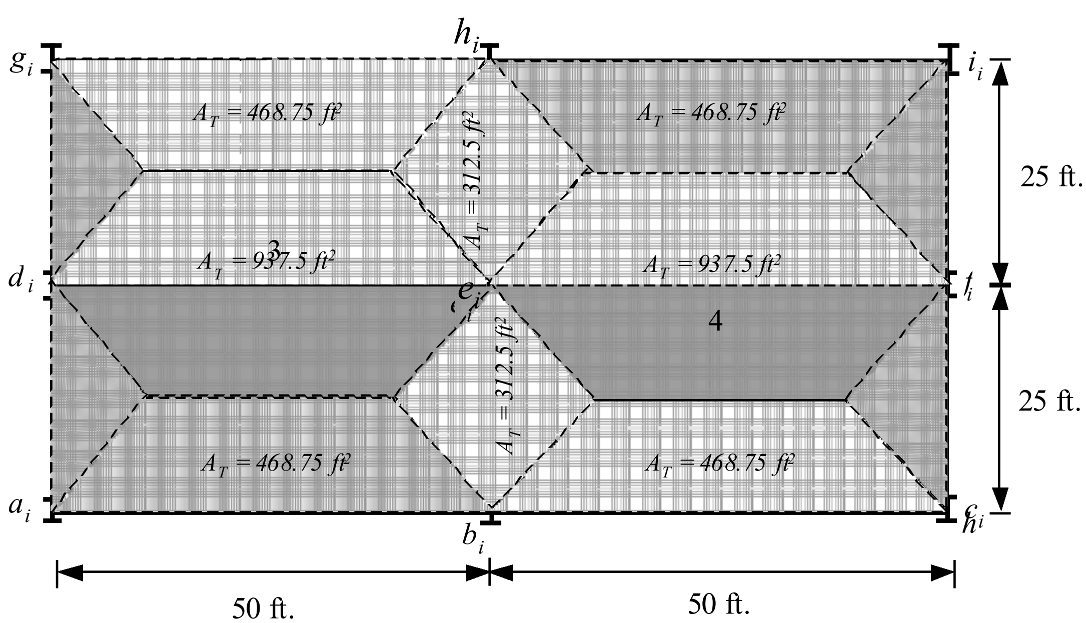
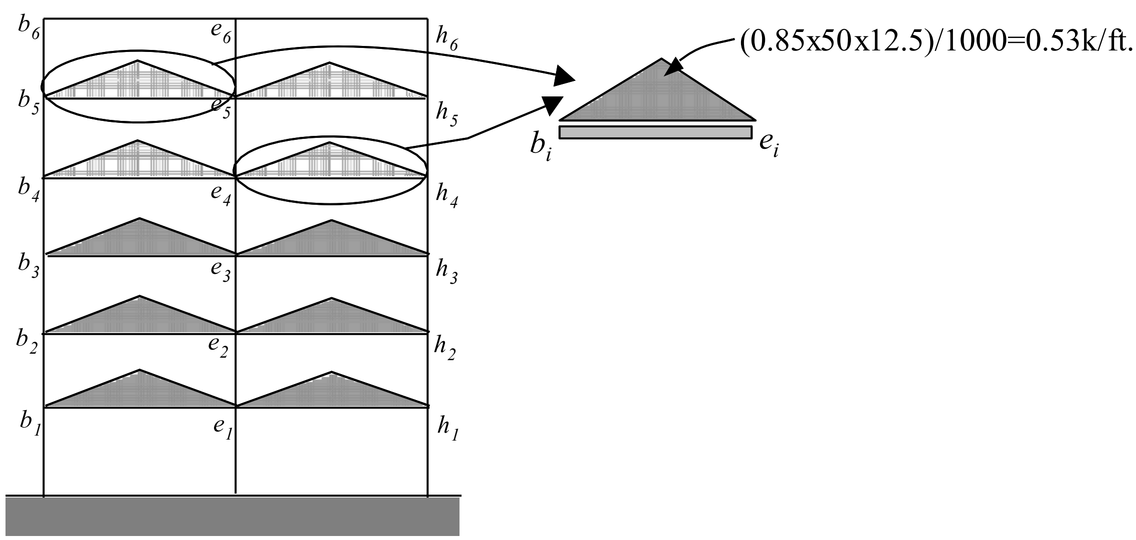


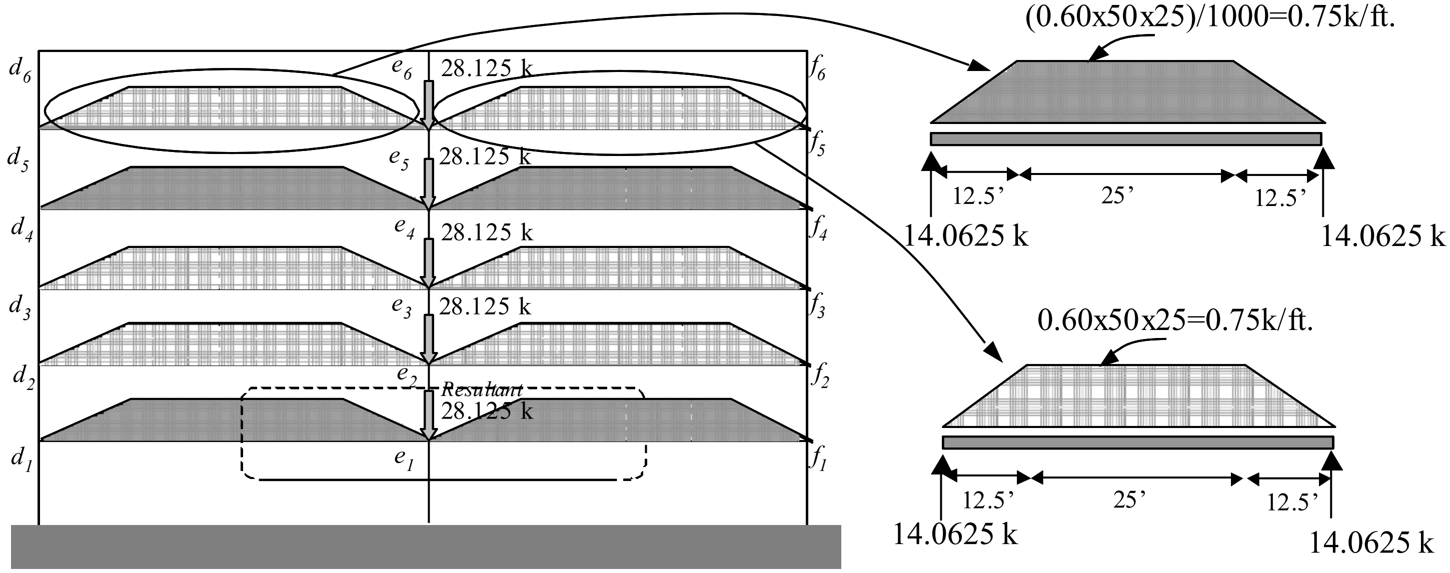
Table 1.1 Member tributary areas and minimum design live loading.

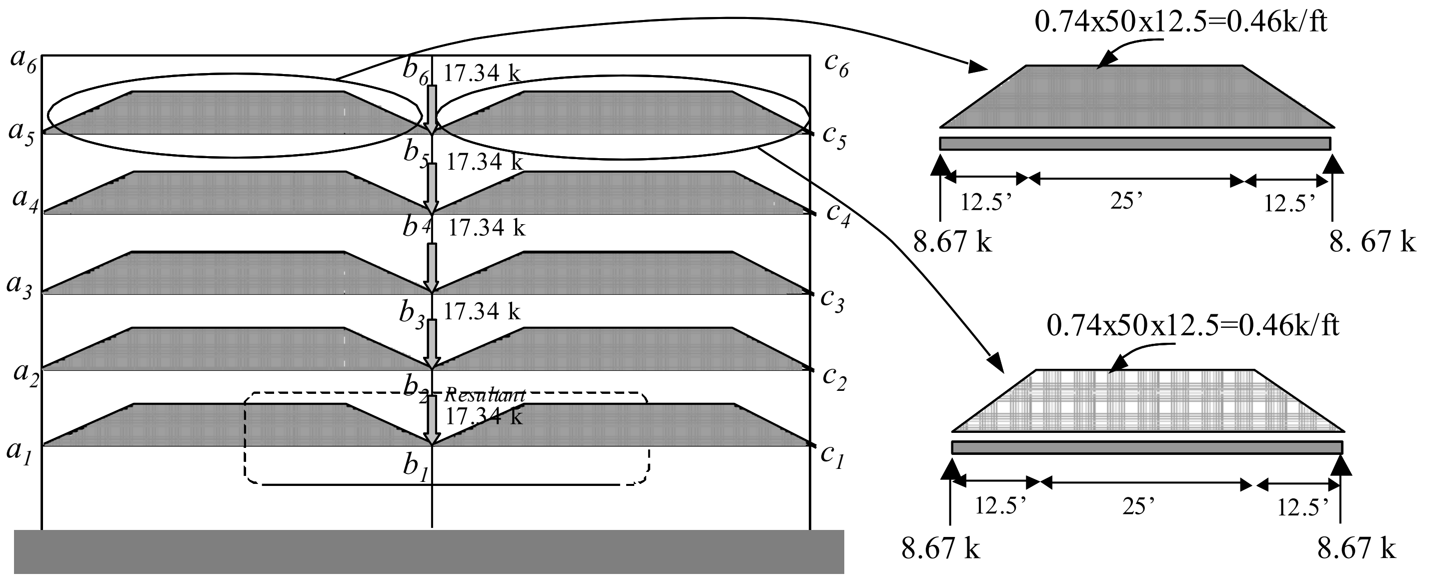
Beam Member	***Tributary area***	K_{LL}	$L/L_o = 0.25 + 15.0/(K_{LL}A_T)^{0.5}$	L/L_o **min.**
b_i-e_i e_i-h_i	A_T = ½ x 25.0 x 12.5 x 2 = 312.5 ft^2	2.0	0.85	0.5
d_i - e_i e_i - f_i	A_T = ½ x 12.5 x 25.0 x 2 + 25.0 x 25.0 = 937.5 ft^2	2.0	0.60	0.5
g_i - h_i h_i - i_i a_i - b_i b_i - c_i	A_T = ½ x (50+25) x 12.5 = 468.75 ft^2	2.0	0.74	0.5
a_i - d_i d_i - g_i c_i - f_i f_i - i_i	A_T = ½ x 25.0 x 12.5 = 156.25 ft^2	2.0	1.0	0.5

Step II. Estimate the distributed loads acting on the beams b_i - e_i and e_i - h_i



Step III: Estimate the concentrated live loads acting on the columns of frame b_i-e_i-h_i, which are produced by the live load distribution on the beams of the orthogonal frames d_i-e_i-f_i , a_i-b_i-c_i , *and* g_i-h_i-i_i

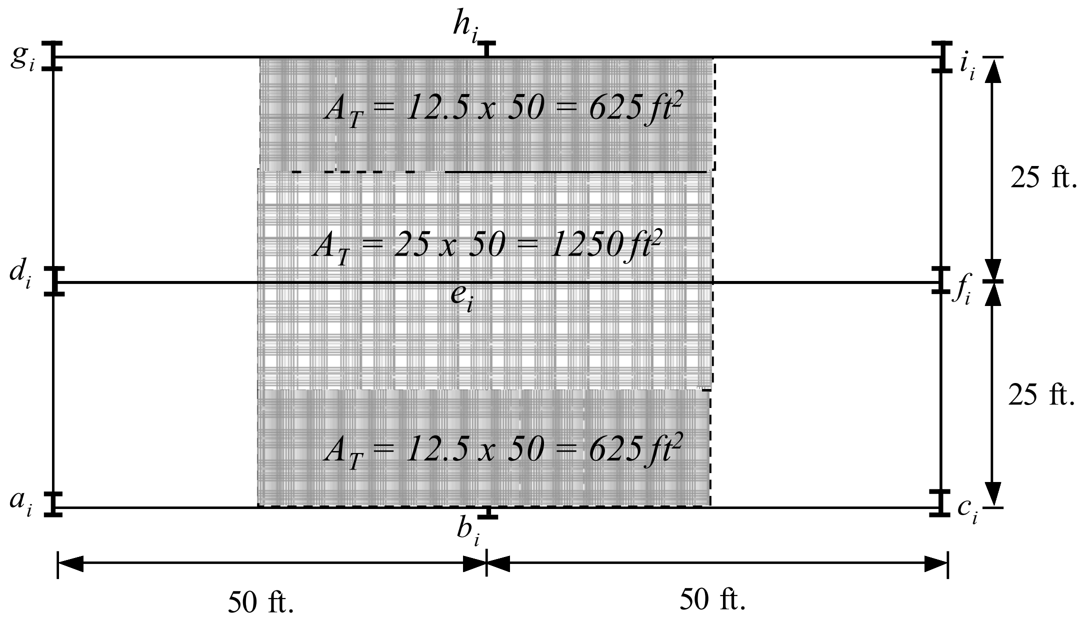




Thus, the concentrated live loads acting on columns b_i and h_i are 17.34 kips

The concentrated live loads acting on columns e_i are 28.125 k

Step IV: Check the estimated column live loadings with values that would be obtained directly for the columns



Column Member	*Tributary area*	K_{LL}	$L/L_o=0.25 + 15.0/(K_{LL}A_T)^{0.5}$	L/L_o *min.*
b_i h_i	$A_T = 12.5 \times 50 = 625\ ft^2$	4.0	0.55	0.4 0.5 for b_6, h_6
e_i	$A_T = 25 \times 50 = 1250\ ft^2$	4.0	0.46	0.4 0.5 for e_6

Live load acting on column b_i and h_i are = 0.55 x 50 psf x 625 = 17.18 kips

Live load acting on column e_i = 0.46 x 50 psf x 1250 = 28.75 kips

- Live load acting on column e_6 = 0.5 x 50 psf x 1250 = 31.25 kips

- **Note that the live loads calculated in Steps I, II, and III are consistent and to be used for design.**

- **The concentrated live load calculated in Step IV are just to check that the loads calculated in Steps I, II, and III are more than the loads calculated in Step IV.**

1.6.4 Roof Live Loads

Ordinary flat, pitched, and curved roofs shall be designed for the live loads specified in Equation 1.2 (from ASCE/SEI 7-10).

$$L_r = 20\ R_1\ R_2 \qquad \text{where, } 12 \leq L_r \leq 20 \qquad (1.2)$$

where,

L_r is the roof live load per square foot of horizontal projection in psf.

$$\mathbf{R_1} \quad \begin{aligned} &= 1 && \text{for } A_T \leq 200\ ft^2 \\ &= 1.2 - 0.001\ A_T && \text{for } 200 < A_T < 600\ ft^2 \\ &= 0.6 && \text{for } 600ft^2 \leq A_T \end{aligned}$$

$$\mathbf{R_2} \quad \begin{aligned} &= 1 && \text{for } F \leq 4 \\ &= 1.2 - 0.05\ F && \text{for } 4 < F < 12 \\ &= 0.6 && \text{for } 12 \leq F \end{aligned}$$

where, F = no. of inches of rise per foot for pitched roof. 1.6.5 Wind Loads

- Design wind loads for buildings can be based on: (a) simplified procedure; (b) analytical procedure; and (c) wind tunnel or small-scale procedure.
- Figure 26.1-1 (ASCE/SEI 7-10) outlines the process for determining wind loads.

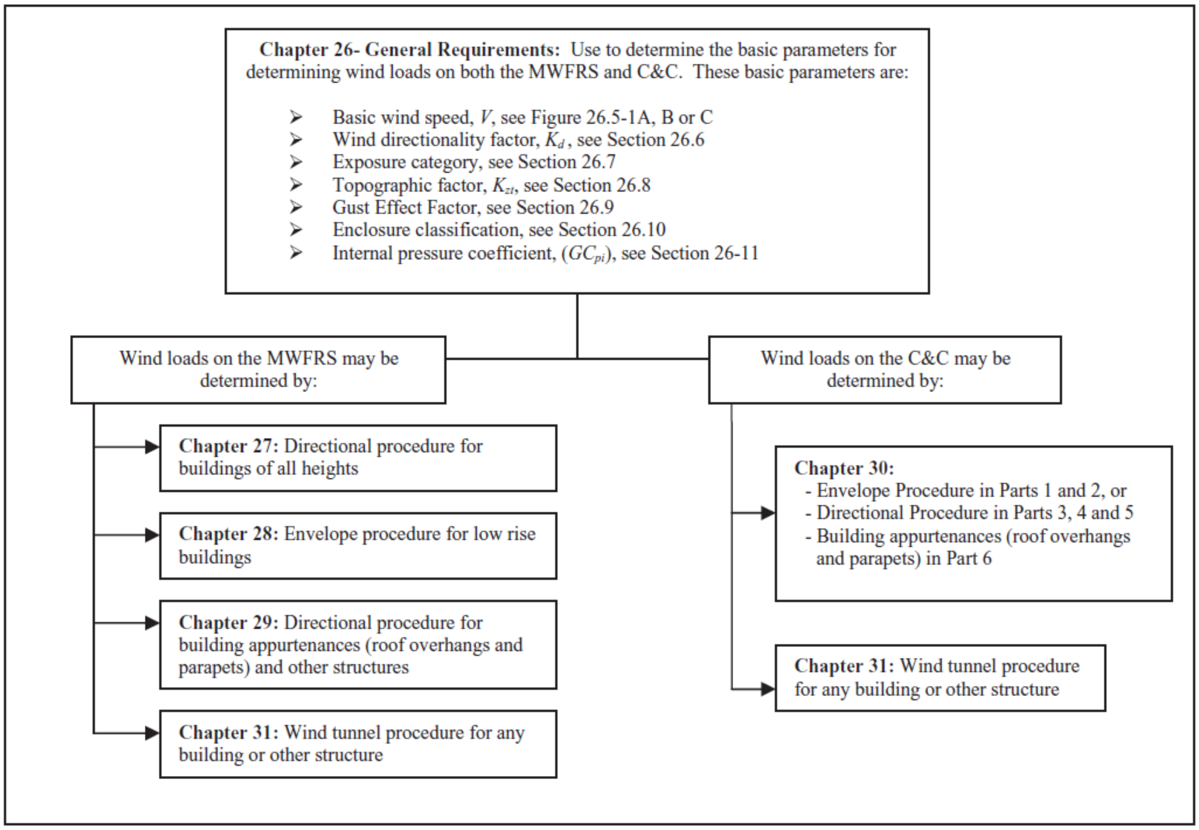


FIGURE 26.1-1 Outline of Process for Determining Wind Loads. Additional outlines and User Notes are provided at the beginning of each chapter for more detailed step-by-step procedures for determining the wind loads.

- Refer to ASCE/SEI 7-10 for the simplified procedure. This simplified procedure is applicable only to buildings with mean roof height less than 30 ft.

- The wind tunnel procedure consists of developing a small-scale model of the building and testing it in a wind tunnel to determine the expected wind pressures etc. It is expensive and may be utilized for difficult or special situations.

- The analytical procedure is used in most design offices. It is fairly systematic but somewhat complicated to account for the various situations that can occur.

- Wind velocity will cause pressure on any surface in its path. The wind velocity and hence the velocity pressure depend on the height from the ground level. Equation 1.3 is recommended by ASCE/SEI 7-10 (Equation 27.3-1) for calculating the velocity pressure (q_z) in lb/ft^2

$$q_z = 0.00256\, K_z\, K_{zt}\, K_d\, V^2 \quad (\text{lb/ft}^2) \qquad (1.3)$$

where, V is the basic wind speed (see Figure 26.5-1 in ASCE/SEI 7-10)

K_d is a wind directionality factor (=0.85 for CE 470)

K_{zt} is a topographic factor (= 1.0 for CE 470)

K_z varies with height z above the ground level and Exposure category (see Table 27.3-1 in ASCE/SEI 7-10)

- Basic Wind Speed (V) :
  - For Risk Category II buildings and structures – use Fig. 26.5-1A
  - For Risk Category III and IV buildings and structures – use Fig. 26.5-1B
  - For Risk Category I buildings and structures – use Fig. 26.5-1C
- A significant portion of the U.S. including West Lafayette has V = 115 mph for Risk Category II (Fig. 26.5-1A). At these locations, for Risk Category II

$$q_z = 28.78\ K_z \qquad (\text{lb/ft}^2) \qquad (1.4)$$

- The velocity pressure q_z is used to calculate the design wind pressure (p) for the building structure as follows:

$$p = q\ GC_p - q_i\ (GC_{pi}) \qquad (\text{lb/ft}^2) \qquad (1.5)$$

where, G = gust effect factor (=0.85 for CE 470)

C_p = external pressure coefficient from Figures 27.4-1, 27.4-2 and 27.4-3in ASCE/SEI 7-10

(GC_{pi})= internal pressure coefficient from Table 26.11-1 in ASCE/SEI 7-10

q depends on the orientation of the building wall or roof with respect to direction of the wind as follows:

$q = q_z$ for the windward wall – varies with height z

$q = q_h$ for leeward wall.
q_h is q_z evaluated at $z = h$ (mean height of building). q_h is constant.

$q_i = q_h$ for windward, leeward, side walls and roofs (fully enclosed building)

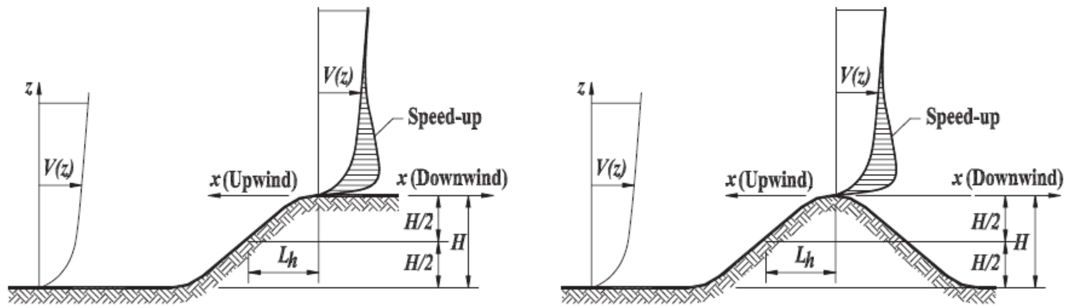
- Note that a positive sign indicates pressure acting towards a surface. Negative sign indicate pressure away from the surface

- Equation 1.5 indicates that the design wind pressure p consists of two components: (1) the external pressure on the building ($q\ GC_p$); and (2) the internal pressure in the building ($q_i GC_{pi}$)

ASCE/SEI 7-10 Figure 26.8-1 (Pg. 252-253)

Topographic Factor, K_{zt}

Figure 26.8-1



ESCARPMENT

2-D RIDGE OR 3-D AXISYMMETRICAL HILL

Topographic Multipliers for Exposure C										
H/L_h	K_1 Multiplier			x/L_h	K_2 Multiplier		z/L_h	K_3 Multiplier		
	2-D Ridge	2-D Escarp.	3-D Axisym. Hill		2-D Escarp.	All Other Cases		2-D Ridge	2-D Escarp.	3-D Axisym. Hill
0.20	0.29	0.17	0.21	0.00	1.00	1.00	0.00	1.00	1.00	1.00
0.25	0.36	0.21	0.26	0.50	0.88	0.67	0.10	0.74	0.78	0.67
0.30	0.43	0.26	0.32	1.00	0.75	0.33	0.20	0.55	0.61	0.45
0.35	0.51	0.30	0.37	1.50	0.63	0.00	0.30	0.41	0.47	0.30
0.40	0.58	0.34	0.42	2.00	0.50	0.00	0.40	0.30	0.37	0.20
0.45	0.65	0.38	0.47	2.50	0.38	0.00	0.50	0.22	0.29	0.14
0.50	0.72	0.43	0.53	3.00	0.25	0.00	0.60	0.17	0.22	0.09
				3.50	0.13	0.00	0.70	0.12	0.17	0.06
				4.00	0.00	0.00	0.80	0.09	0.14	0.04
							0.90	0.07	0.11	0.03
							1.00	0.05	0.08	0.02
							1.50	0.01	0.02	0.00
							2.00	0.00	0.00	0.00

Notes:

1. For values of H/L_h, x/L_h and z/L_h other than those shown, linear interpolation is permitted.
2. For $H/L_h > 0.5$, assume $H/L_h = 0.5$ for evaluating K_1 and substitute 2H for L_h for evaluating K_2 and K_3.
3. Multipliers are based on the assumption that wind approaches the hill or escarpment along the direction of maximum slope.
4. Notation:
   - H: Height of hill or escarpment relative to the upwind terrain, in feet (meters).
   - L_h: Distance upwind of crest to where the difference in ground elevation is half the height of hill or escarpment, in feet (meters).
   - K_1: Factor to account for shape of topographic feature and maximum speed-up effect.
   - K_2: Factor to account for reduction in speed-up with distance upwind or downwind of crest.
   - K_3: Factor to account for reduction in speed-up with height above local terrain.
   - x: Distance (upwind or downwind) from the crest to the building site, in feet (meters).
   - z: Height above ground surface at building site, in feet (meters).
   - μ: Horizontal attenuation factor.
   - γ: Height attenuation factor.

Topographic Factor, K_{zt}

Figure 26.8-1 (cont'd)

Equations:

$$K_{zt} = (1 + K_1 K_2 K_3)^2$$

K_1 determined from table below

$$K_2 = \left(1 - \frac{|x|}{\mu L_h}\right)$$

$$K_3 = e^{-\gamma z / L_h}$$

Parameters for Speed-Up Over Hills and Escarpments						
Hill Shape	**$K_1/(H/L_h)$**			**γ**	**μ**	
	Exposure				**Upwind of Crest**	**Downwind of Crest**
	B	**C**	**D**			
2-dimensional ridges (or valleys with negative H in $K_1/(H/L_h)$)	1.30	1.45	1.55	3	1.5	1.5
2-dimensional escarpments	0.75	0.85	0.95	2.5	1.5	4
3-dimensional axisym. hill	0.95	1.05	1.15	4	1.5	1.5

Figure showing the Basic Wind Speed map of US (Category II Buildings). (ASCE/SEI 7-10 Pages 290-291)

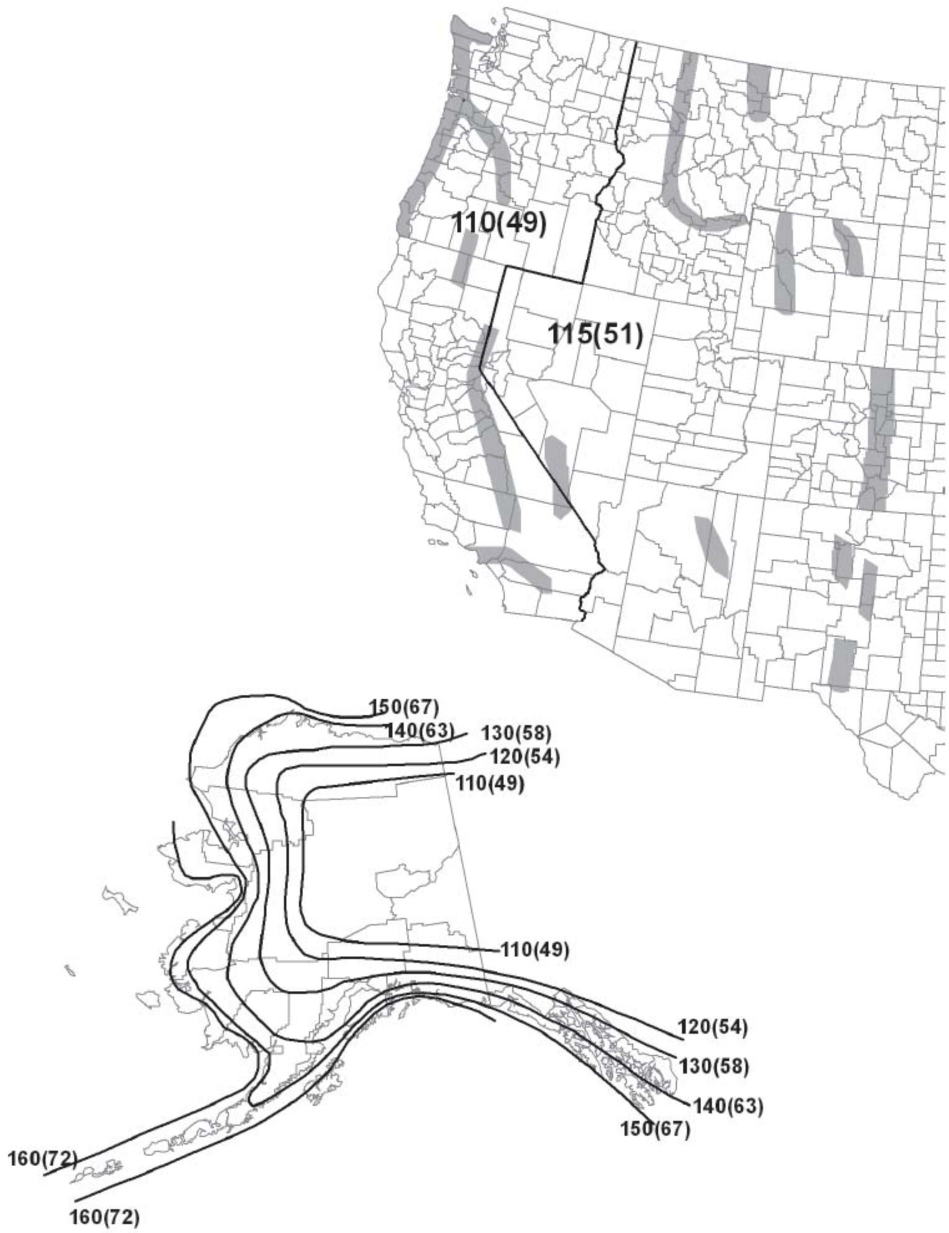
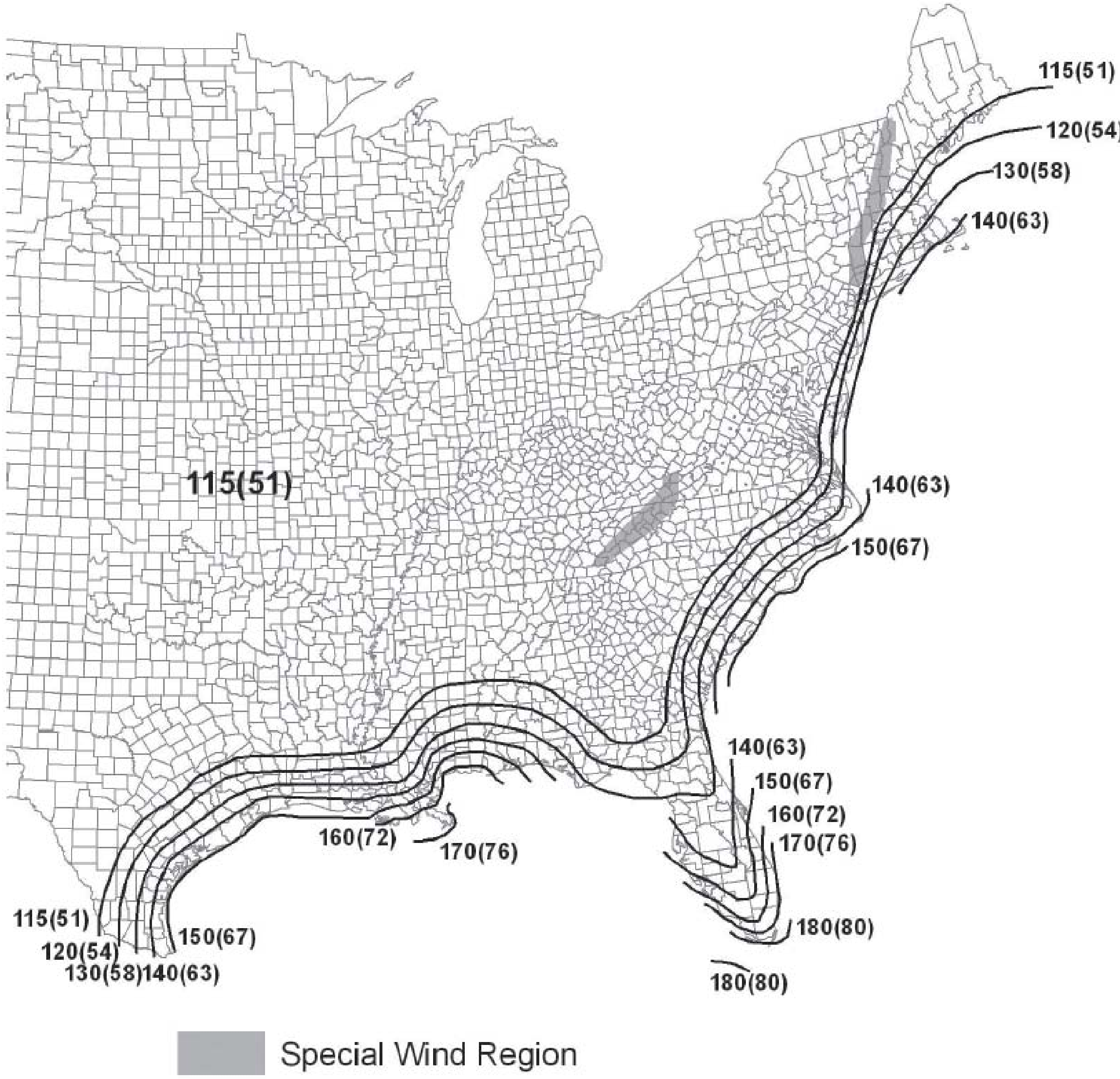


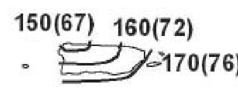
Figure 26.5-1A Basic Wind Speeds for Occupancy Category II Buildings and Other Structures.

Notes:

1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10m) above ground for Exposure C category.
2. Linear interpolation between contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.
5. Wind speeds correspond to approximately a 7% probability of exceedance in 50 years (Annual Exceedance Probability = 0.00143, MRI = 700 Years).



Location	Vmph	(m/s)
Guam	195	(87)
Virgin Islands	165	(74)
American Samoa	160	(72)
Hawaii – Special Wind Region Statewide	130	(58)

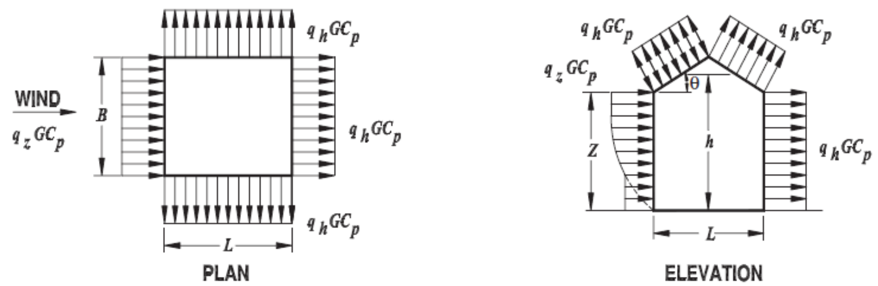


Puerto Rico

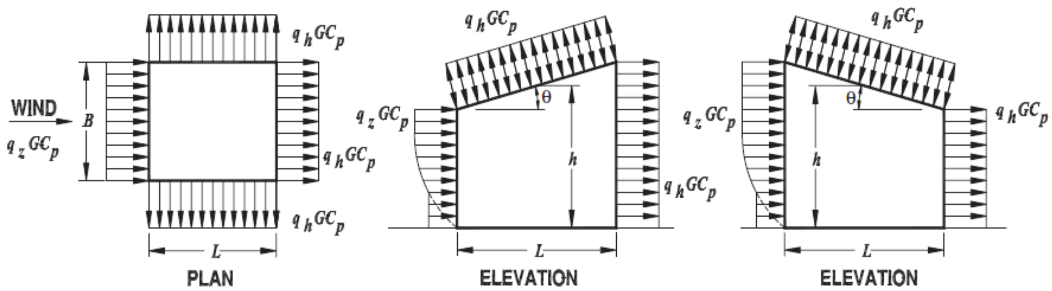
Figure 26.5-1A *(Continued)*

External Pressure Coefficients Figures 27.4-1, 2 &3 (ASCE/SEI 7-10 Pages.263-266)

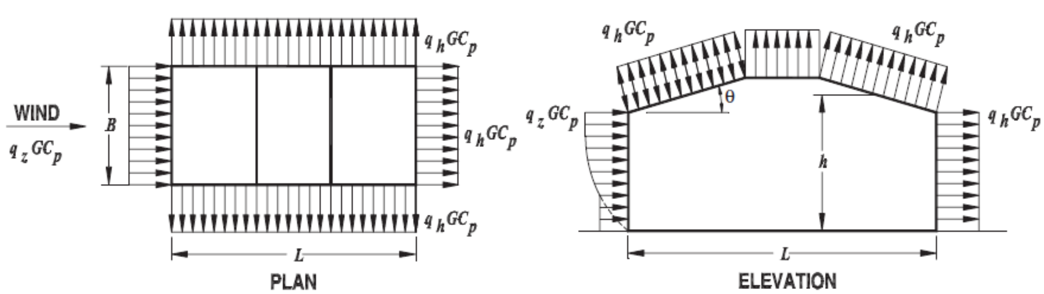
Main Wind Force Resisting System – Part 1		**All Heights**
Figure 27.4-1	**External Pressure Coefficients, C_p**	**Walls & Roofs**
Enclosed, Partially Enclosed Buildings		



GABLE, HIP ROOF



MONOSLOPE ROOF (NOTE 4)



MANSARD ROOF (NOTE 8)

Main Wind Force Resisting System – Part 1		All Heights
Figure 27.4-1 (cont.)	External Pressure Coefficients, C_p	**Walls & Roofs**
Enclosed, Partially Enclosed Buildings		

Wall Pressure Coefficients, C_p

Surface	L/B	C_p	Use With
Windward Wall	All values	0.8	q_z
Leeward Wall	0-1	-0.5	q_h
	2	-0.3	
	≥4	-0.2	
Side Wall	All values	-0.7	q_h

Roof Pressure Coefficients, C_p, for use with q_h

Wind Direction	Windward									Leeward		
	Angle, θ (degrees)									Angle, θ (degrees)		
	h/L	10	15	20	25	30	35	45	≥60#	10	15	≥20
Normal to ridge for θ ≥ 10°	≤0.25	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.3	-0.2 0.3	0.0* 0.4	0.4	0.01 θ	-0.3	-0.5	-0.6
	0.5	-0.9 -0.18	-0.7 -0.18	-0.4 0.0*	-0.3 0.2	-0.2 0.2	-0.2 0.3	0.0* 0.4	0.01 θ	-0.5	-0.5	-0.6
	≥1.0	-1.3** -0.18	-1.0 -0.18	-0.7 -0.18	-0.5 0.0*	-0.3 0.2	-0.2 0.2	0.0* 0.3	0.01 θ	-0.7	-0.6	-0.6

Wind Direction	h/L	Horiz distance from windward edge	C_p
Normal to ridge for θ < 10 and Parallel to ridge for all θ	≤ 0.5	0 to h/2	-0.9, -0.18
		h/2 to h	-0.9, -0.18
		h to 2 h	-0.5, -0.18
		> 2h	-0.3, -0.18
	≥ 1.0	0 to h/2	-1.3**, -0.18
		> h/2	-0.7, -0.18

*Value is provided for interpolation purposes.

**Value can be reduced linearly with area over which it is applicable as follows

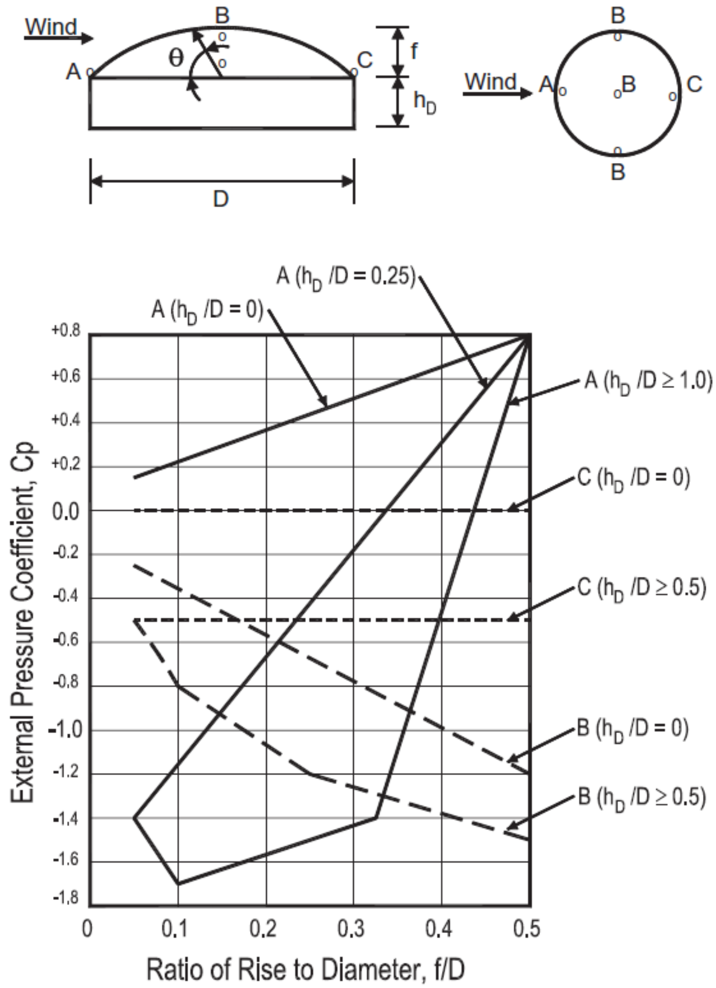
Area (sq ft)	Reduction Factor
≤ 100 (9.3 sq m)	1.0
250 (23.2 sq m)	0.9
≥ 1000 (92.9 sq m)	0.8

Notes:

1. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
2. Linear interpolation is permitted for values of *L/B, h/L* and θ other than shown. Interpolation shall only be carried out between values of the same sign. Where no value of the same sign is given, assume 0.0 for interpolation purposes.
3. Where two values of C_p are listed, this indicates that the windward roof slope is subjected to either positive or negative pressures and the roof structure shall be designed for both conditions. Interpolation for intermediate ratios of h/L in this case shall only be carried out between C_p values of like sign.
4. For monoslope roofs, entire roof surface is either a windward or leeward surface.
5. For flexible buildings use appropriate G_f as determined by Section 26.9.4.
6. Refer to Figure 27.4-2 for domes and Figure 27.4-3 for arched roofs.
7. Notation:
   B: Horizontal dimension of building, in feet (meter), measured normal to wind direction.
   L: Horizontal dimension of building, in feet (meter), measured parallel to wind direction.
   h: Mean roof height in feet (meters), except that eave height shall be used for θ ≤ 10 degrees.
   z: Height above ground, in feet (meters).
   G: Gust effect factor.
   q_z, q_h: Velocity pressure, in pounds per square foot (N/m^2), evaluated at respective height.
   θ: Angle of plane of roof from horizontal, in degrees.
8. For mansard roofs, the top horizontal surface and leeward inclined surface shall be treated as leeward surfaces from the table.
9. Except for MWFRS's at the roof consisting of moment resisting frames, the total horizontal shear shall not be less than that determined by neglecting wind forces on roof surfaces.

#For roof slopes greater than 80°, use C_p = 0.8

Main Wind Force Resisting System – Part 1		**All Heights**
Figure 27.4-2	**External Pressure Coefficients, C_p**	**Domed Roofs**
Enclosed, Partially Enclosed Buildings and Structures		



External Pressure Coefficients for Domes with a Circular Base.

(Adapted from Eurocode, 1995)

Notes:

1. Two load cases shall be considered:
   Case A. C_p values between A and B and between B and C shall be determined by linear interpolation along arcs on the dome parallel to the wind direction;
   Case B. C_p shall be the constant value of A for $\theta \leq 25$ degrees, and shall be determined by linear interpolation from 25 degrees to B and from B to C.
2. Values denote C_p to be used with $q_{(h_D+f)}$ where $h_D + f$ is the height at the top of the dome.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. C_p is constant on the dome surface for arcs of circles perpendicular to the wind direction; for example, the arc passing through B-B-B and all arcs parallel to B-B-B.
5. For values of h_D/D between those listed on the graph curves, linear interpolation shall be permitted.
6. $\theta = 0$ degrees on dome springline, $\theta = 90$ degrees at dome center top point. f is measured from springline to top.
7. The total horizontal shear shall not be less than that determined by neglecting wind forces on roof surfaces.
8. For f/D values less than 0.05, use Figure 27.4-1.

Main Wind Force Resisting System and Components and Cladding – Part 1		All Heights
Figure 27.4-3	**External Pressure Coefficients, C_p**	**Arched Roofs**
Enclosed, Partially Enclosed Buildings and Structures		

Conditions	Rise-to-span ratio, *r*	C_p		
		Windward quarter	**Center half**	**Leeward quarter**
Roof on elevated structure	$0 < r < 0.2$	-0.9	$-0.7 - r$	-0.5
	$0.2 \leq r < 0.3$*	$1.5r - 0.3$	$-0.7 - r$	-0.5
	$0.3 \leq r \leq 0.6$	$2.75r - 0.7$	$-0.7 - r$	-0.5
Roof springing from ground level	$0 < r \leq 0.6$	$1.4r$	$-0.7 - r$	-0.5

*When the rise-to-span ratio is $0.2 \leq r \leq 0.3$, alternate coefficients given by $6r - 2.1$ shall also be used for the windward quarter.

Notes:

1. Values listed are for the determination of average loads on main wind force resisting systems.
2. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
3. For wind directed parallel to the axis of the arch, use pressure coefficients from Fig. 27.4-1 with wind directed parallel to ridge.
4. For components and cladding: (1) At roof perimeter, use the external pressure coefficients in Fig. 30.4-2A, B and C with θ based on spring-line slope and (2) for remaining roof areas, use external pressure coefficients of this table multiplied by 0.87.

Wind Directionality Factor (ASCE/SEI 7-10 Page 250)

Wind Directionality Factor, K_d

Table 26.6-1

Structure Type	Directionality Factor K_d*
Buildings **Main Wind Force Resisting System** **Components and Cladding**	 0.85 0.85
Arched Roofs	0.85
Chimneys, Tanks, and Similar Structures **Square** **Hexagonal** **Round**	 0.90 0.95 0.95
Solid Freestanding Walls and Solid Freestanding and Attached Signs	0.85
Open Signs and Lattice Framework	0.85
Trussed Towers **Triangular, square, rectangular** **All other cross sections**	 0.85 0.95

*Directionality Factor K_d has been calibrated with combinations of loads specified in Chapter 2. This factor shall only be applied when used in conjunction with load combinations specified in Sections 2.3 and 2.4.

Velocity Pressure Exposure Coefficient (ASCE/SEI 7-10, Page 261)

Main Wind Force Resisting System – Part 1	**All Heights**
Velocity Pressure Exposure Coefficients, K_h and K_z	
Table 27.3-1	

Height above ground level, z		**Exposure**		
ft	**(m)**	**B**	**C**	**D**
0-15	(0-4.6)	0.57	0.85	1.03
20	(6.1)	0.62	0.90	1.08
25	(7.6)	0.66	0.94	1.12
30	(9.1)	0.70	0.98	1.16
40	(12.2)	0.76	1.04	1.22
50	(15.2)	0.81	1.09	1.27
60	(18)	0.85	1.13	1.31
70	(21.3)	0.89	1.17	1.34
80	(24.4)	0.93	1.21	1.38
90	(27.4)	0.96	1.24	1.40
100	(30.5)	0.99	1.26	1.43
120	(36.6)	1.04	1.31	1.48
140	(42.7)	1.09	1.36	1.52
160	(48.8)	1.13	1.39	1.55
180	(54.9)	1.17	1.43	1.58
200	(61.0)	1.20	1.46	1.61
250	(76.2)	1.28	1.53	1.68
300	(91.4)	1.35	1.59	1.73
350	(106.7)	1.41	1.64	1.78
400	(121.9)	1.47	1.69	1.82
450	(137.2)	1.52	1.73	1.86
500	(152.4)	1.56	1.77	1.89

Notes:

1. The velocity pressure exposure coefficient K_z may be determined from the following formula:

   For 15 ft. $\leq z \leq z_g$ — $K_z = 2.01\,(z/z_g)^{2/\alpha}$

   For $z <$ 15 ft. — $K_z = 2.01\,(15/z_g)^{2/\alpha}$
2. α and z_g are tabulated in Table 26.9.1.
3. Linear interpolation for intermediate values of height z is acceptable.
4. Exposure categories are defined in Section 26.7.

Internal Pressure Coefficient for buildings (ASCE/SEI 7-10. Page 258)

Main Wind Force Resisting System and Components and Cladding		All Heights
Table 26.11-1	Internal Pressure Coefficient, (GC_{pi})	Walls & Roofs
Enclosed, Partially Enclosed, and Open Buildings		

Enclosure Classification	(GC_{pi})
Open Buildings	0.00
Partially Enclosed Buildings	+0.55 -0.55
Enclosed Buildings	+0.18 -0.18

Notes:

1. Plus and minus signs signify pressures acting toward and away from the internal surfaces, respectively.
2. Values of (GC_{pi}) shall be used with q_z or q_h as specified.
3. Two cases shall be considered to determine the critical load requirements for the appropriate condition:

   (i) a positive value of (GC_{pi}) applied to all internal surfaces
   (ii) a negative value of (GC_{pi}) applied to all internal surfaces

Example 1.2

Consider the building structure with the structural floor plan and elevation shown below. Estimate the wind loads acting on the structure when the wind blows in the east-west direction. The structure is located in West Lafayette.

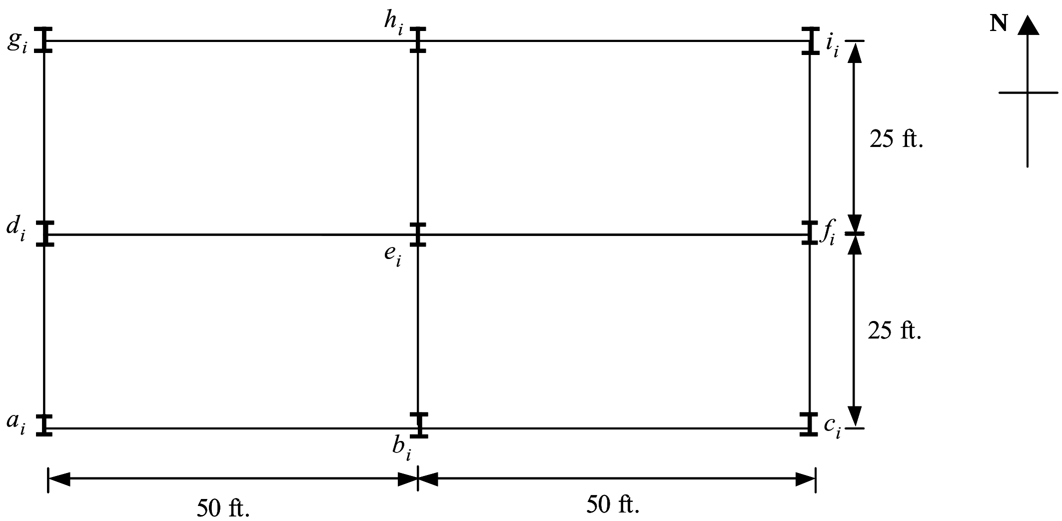


Figure 8. Structural floor plan

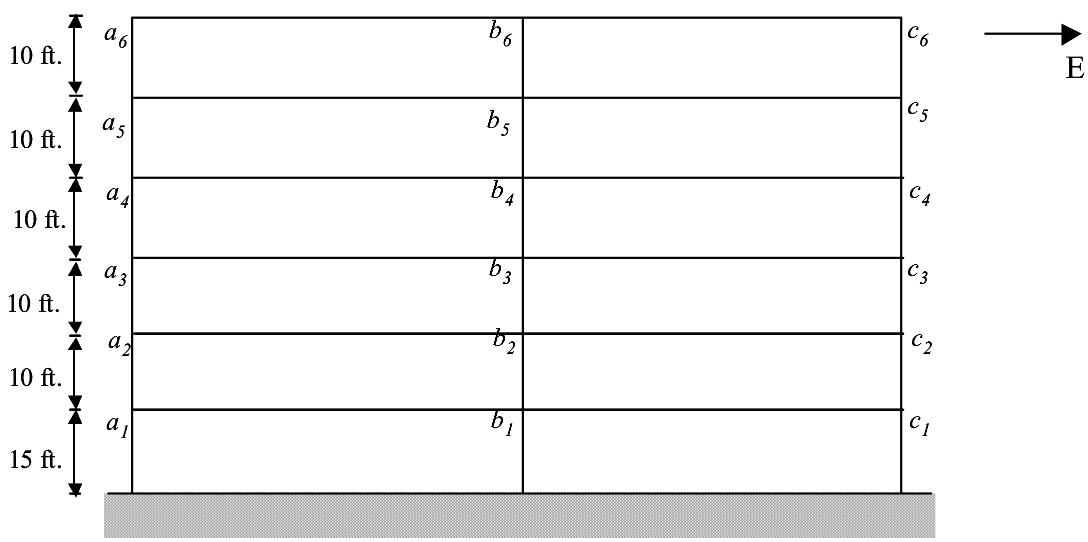


Figure 9. Structural elevation in east-west direction

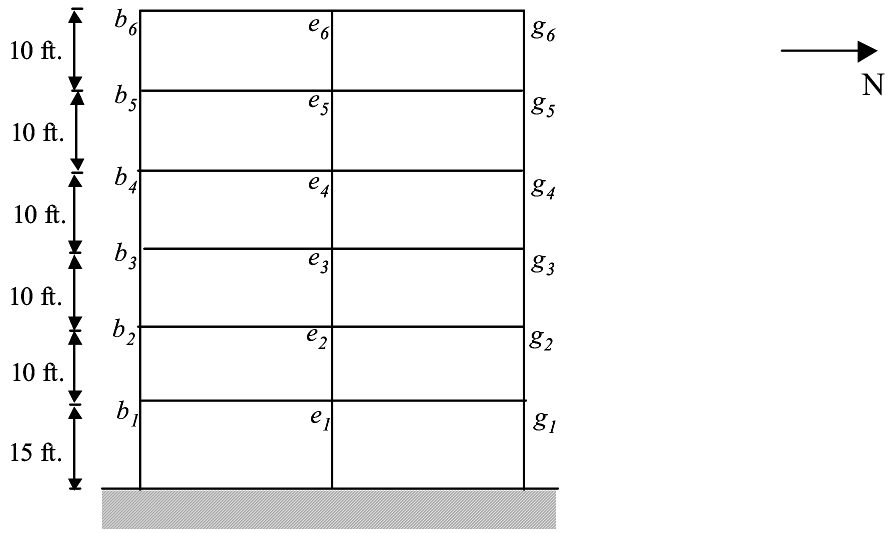


Figure 10. Structural elevation in north-south direction

- Velocity pressure (q_z)
  - K_d = directionality factor = 0.85
  - K_{zt} = topographic factor = 1.0
  - *K_h values for Exposure B, Case 2*

K_h	*z(ft)*
0.57	0 - 15
0.62	20
0.66	25
0.70	30
0.76	40
0.81	50
0.85	60
0.89	70

- $q_z = 0.00256\ K_z\ K_{zt}\ K_d\ V^2$
  - *In* West Lafayette, V = 115 mph for Risk Category II (Fig. 26.5-1A)
  - $q_z = 28.78\ K_z$ psf

- Wind pressure (p)
  - Gust factor = G = 0.85
  - For wind in east west direction; L/B = Length / width = 2.0
  - External pressure coefficient = C_p = +0.8 for windward walls
    C_p = -0.3 for leeward walls
    C_p = -0.7 for side walls
  - External pressure = $q\ G\ C_p$
  - External pressure on windward wall = $q_z\ GC_p$ = 28.78 K_z x 0.85 x 0.8
    = 19.57 K_z psf toward surface
  - External pressure on leeward wall = $q_h\ GC_p$ = 28.78 K_{65} x 0.85 x (-0.3)
    = 6.38 psf away from surface
  - External pressure on side wall = $q_h\ GC_p$ =28.78 K_{65} x 0.85 x (-0.7)
    = 14.90 psf away from surface
  - The external pressures on the structure are shown in Figures 11 and 12 below.

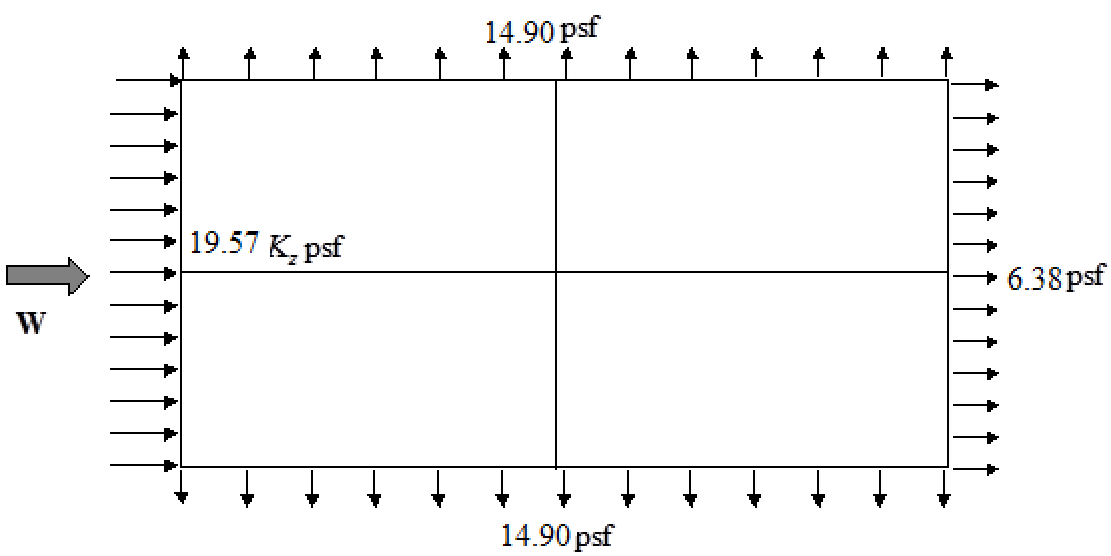


Figure 11. External pressures on structural plan

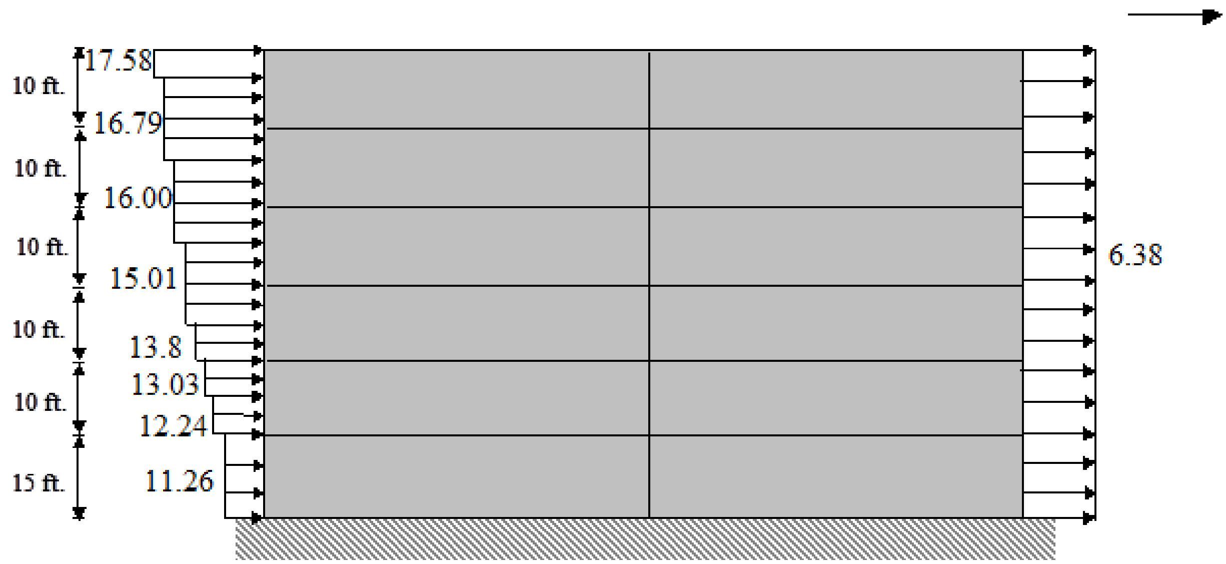


Figure 12. External pressure on structural elevation (east west)

- Internal pressure
  - $q_i\ (GC_{pi})$
  - $q_i = q_h = 28.78\ K_{65} = 28.78 \times 0.87 = 25.04\ psf$
  - Enclosed building; $GC_{pi} = +0.18$ (acting toward surface)
    $GC_{pi} = -0.18$ (acting away from surface)
  - $q_i\ (GC_{pi}) = 4.51$ psf acting toward or away from surface
  - See Figure 13 (a) and (b) below

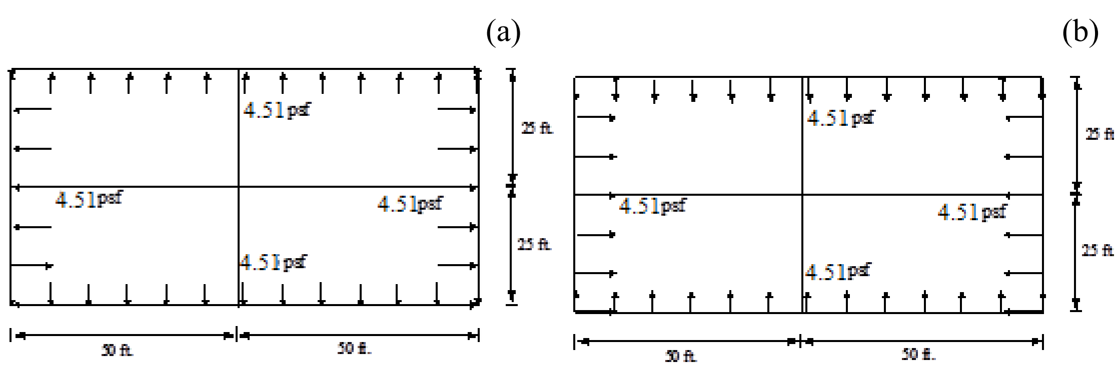


Figure 13. Internal pressure seen in structural plan

- Take the external pressure from Figure 11 and 12 and add to internal pressure from Figures 13 (a) and (b) to obtain the final pressure diagrams. Adding the internal pressure will not change the lateral forces in the structure.

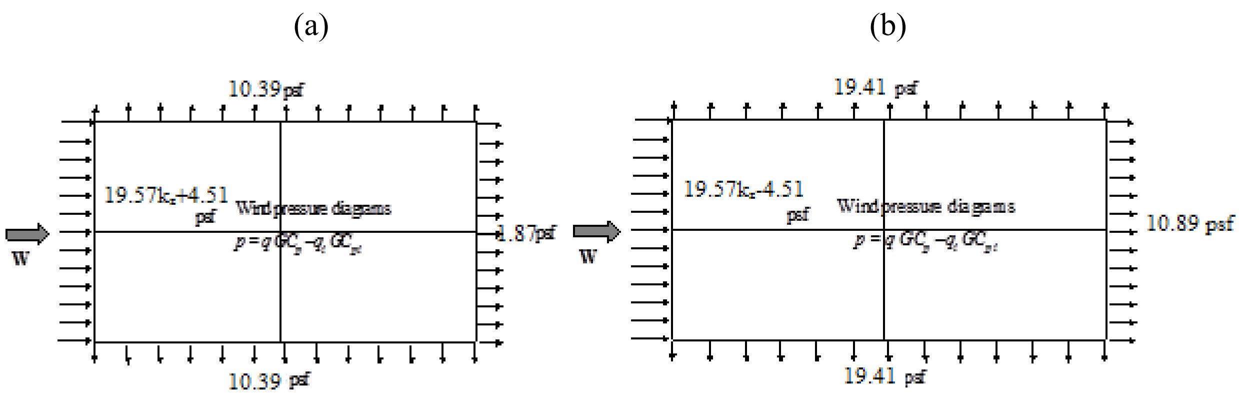


Figure 14. Resultant wind pressure diagrams including external and internal pressures

- Note: According to ASCE/SEI 7-10, the minimum wind design loading is equal to 16 lb/ft^2 multiplied by the area of the building projected on a vertical plane normal to assumed wind direction.

- Compare the determined design wind loading with the *minimum* value and continue with the greater as the design wind loading..

1.7 Load and Resistance Factor Design

The load and resistance factor design approach is recommended by AISC for designing steel structures. It can be understood as follows:

<u>Step I. Determine the ultimate loads acting on the structure</u>

- The values of D, L, W, etc. given by ASCE/SEI 7-10 are nominal loads (not maximum or ultimate)
- During its design life, a structure can be subjected to some maximum or ultimate loads caused by combinations of D, L, or W loading.
- The ultimate load on the structure can be calculated using <u>factored load combinations</u>, which are given by ASCE and AISC (see pages 2-10 and 2-11 of AISC manual). The most relevant of these load combinations are given below:

$$1.4\,D \qquad (1.6-1)$$

$$1.2\,D + 1.6\,L + 0.5\,(L_r \text{ or } S) \qquad (1.6-2)$$

$$1.2\,D + 1.6\,(L_r \text{ or } S) + (0.5\,L \text{ or } 0.5\,W) \qquad (1.6-3)$$

$$1.2\,D + 1.0W + 0.5\,L + 0.5\,(L_r \text{ or } S) \qquad (1.6-4)$$

$$0.9\,D + 1.0\,W \qquad (1.6-5)$$

Step II. Conduct linear elastic structural analysis

- Determine the design forces (P_u, V_u, and M_u) for each structural member

Step III. Design the members

- The failure (design) strength of the designed member must be greater than the corresponding design forces calculated in Step II. See Equation (1.7) below:

$$\phi\, R_n > \sum \gamma_i\ Q_i \qquad (1.7)$$

- Where, R_n is the calculated failure strength of the member
- ϕ is the resistance factor used to account for the reliability of the material behavior and equations for R_n
- Q_i is the nominal load
- γ_i is the load factor used to account for the variability in loading and to estimate the ultimate loading condition.