PDHonline Course S285 (6 PDH)

Hurricane Resistant Structures

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2015

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1. INTRODUCTION

This paper covers the description of hurricanes, determination of loads, and examples of non-flood and flood-resistant residences.

The description of hurricanes includes their location, season, and physical origin. A mathematical model is developed, with an example. Hurricane facts include classification and speed.

The loads considered are wind, missile impact, and flood Wind loads are of two types, Main Wind Force Resisting System (MWFRS) forces, and Components and Cladding (C&C) forces. Care must be used in differentiating between ASD and strength forces. Missile impact loads affect windows, door, and walls. All flood loads depend on ds, the design still water depth in feet.

Residence structures, both in the non-flood resistant and flood-resistant structures are light wood frame buildings. items developed include the box system, diaphragms, shear walls, nailing requirements, end wall framing, floor system, foundation, and connectors.

The structure in the flood zone is supported off the ground by corrosion- resistant tapered timber piles.

2. HURRICANE DESCRIPTION

North American hurricanes start in warm (>80°F) ocean waters between 8° and 15° north latitude. The season lasts from June 1 to November 3.

Water vapor evaporating from the ocean gains the latent heat of vaporization and ascends. As it rises, two things happen. First, surrounding denser air is condensed, forming clouds, releasing its heat. The entire process, if a hurricane is to be formed, relies constant winds with altitude. If wind shear, i.e., different wind directions at different altitudes, is present, the storm dissipates. The highest winds and rainfall are present at an approximately circular eye wall, which rapidly diminishes towards the center of the eye.

Wind speeds at the surface may be approximately modeled by the vector sum of a vortex flow and a sink flow.

The vortex flow, counter-clockwise in the northern hemisphere, is characterized by flow in a circular direction, magnitude constant at a certain radius, and inversely proportional to the radius from the center of the eye. The equation describing this flow in Cartesian coordinates is:

$\mathbf{h}1 = -k*(y*\mathbf{i} - x*\mathbf{j})/2*\Pi*(x\hat{}2+y\hat{}2)$, **where**

$\mathbf{h}1$ **= vector representing vortex flow, mph**

$\mathbf{i}, \mathbf{j}$ **= unit vectors in x and y direction, respectively, dimensionless**

k = strength of vortex flow, mph*mile

x,y = Cartesian coordinates in east and north

Note that the minimum value of (x^2+y^2)^(1/2) is the radius of the eye.

Let the magnitude of this component be h1.

h1 = k/(2*∏*(x^2+y^2)^(1/2)

Now if h1 is known at a certain radius, say r1, then

k = 2*∏*r1*h1

The sink flow, inwards, is characterized by constant value at a given radius, and inversely proportional to the radius from the center of the eye. Its equation is:

h2 = - q*(x*i + y*j)/2*∏*(x^2 + y^2)

q = strength of sink flow, mph/mile

In a similar manner to that above, if h2 is known at r0, then q = 2*∏*r0*h2.

The two components may be combined to give:

htotal = -((q*x+k*y)i+(q*y-k*x)j)/2*∏*(x^2+y^2)

The magnitude of the total flow at a given point may be shown as:

h = 0, (x^2+y^2)^(1/2) < radius of eye
h = (k^2+q^2)^(1/2)/2*∏*(x^2+y^2), where (x^2+y^2)^(1/2) >= radius of eye

Example 1.

Given : Vortex component at 50 mile radius = 30 mph
Sink component at 50 mile radius = 10 mph
Diameter of eye = 25 miles
Find : k, q
Wind speed at eye wall
Find radius where wind speed drops to 10 mph
Plot : Wind speed at surface vs. x and y, from center eye to location where wind speed = 10 mph.

Solution
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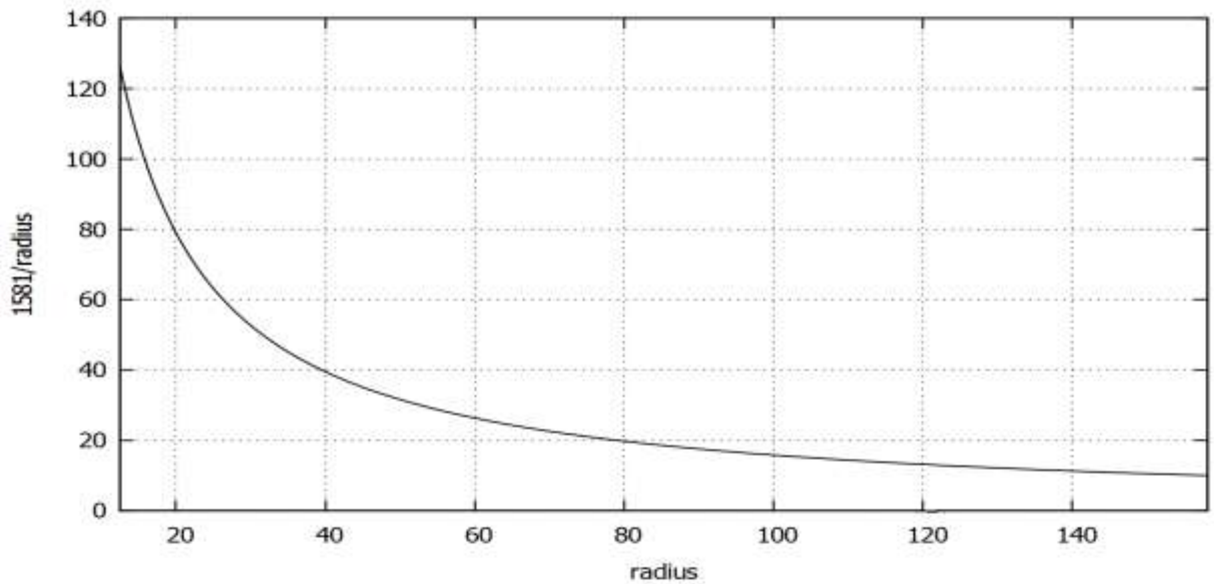
k = 2*∏*50*30 = 9425 mph*mi
q = 2*∏*50*10 = 3142 mph*mi

At the eye wall, h = (k^2+q^2)^(1/2)/2*∏*r0 where
r0 = radius of eye
h = 126 mph

If wind speed = 10 mph, then
r = (k^2+q^2)^(1/2)/2*∏*10
r = 158 miles



Other hurricane facts:

- **Hurricanes are classified by their highest 1 minute average wind speed using the Saffir-Simpson scale:**

Category	**Wind Speed(mph)**	**Damage**
1	**74- 95**	**minimal**
2	**96-110**	**moderate**
3	**111-130**	**extensive**
4	**131-155**	**extreme**
5	**>155**	**catastrophic**

- **Storm surge – water level above high tide at coastal area – most pronounced where ocean surface has a gradual slope – can be 100 miles wide and 15 feet deep**
- **Hurricanes move at approximately 10-20 mph and veer to the right because of the earth's rotation**
- **Names are given alternatively male and female, skipping Q,U,X,Y,Z**
- **Hurricanes can churn up the seas into peaks and valleys of 50 feet.**
- **The eye has the warmest temperatures, low humidity, low pressure, and calm winds.**

- **Viewed from above, the hurricane appears to be a clear eye, bordered by a ring of clouds (the eye wall), and surrounding by inward spirally bands.**
- **Typical eye diameters range from 18 to 36 miles.**
- **A very good source of hurricane forecasts is the website of the National Hurricane Center, shown as Reference 1.**

3. LOADS

There are three (3) major types of loads associated with hurricanes, namely wind loads, missile impact, and floods. This section examines these types in detail.

3.1 WIND LOADS

Reference 2, "Minimum Design Loads for Buildings and Other Structures", ASCE 7-10, is the specification here. In this specification, two sets of loads are generated, those applying to the main wind-force resisting system (MWFRS) and those to the components and cladding (C&C).
The first (MWFRS) covers the basic structure providing stability and support (eg, beams, columns, diaphragms), and the second (C&C) receiving the direct wind loads (roof and wall panels, purlins, girts, exterior wall studs). Some elements are analyzed under both conditions.

The method of analysis used here is the method given in Part 2 of Chapter 28 in ASCE 7-10.
It applies to simple diaphragm, low rise (<60 ft), regularly shaped buildings with a roof angle less than or equal to 45°.
If the building is within one (1) mile of a water line with wind speed >= 130 mph, or is within a region with wind speed >= 140 mph, windows and all openings shall be protected against windborne debris according to ASTM E1996, Reference 3.

It should be noted that this wind pressure derived by this Procedure has a wind load factor of 1.0 for strength design and a 0.6 factor for allowable stress design (ASD).

The MWFRS consists of six (6) major parts.

(1) Find the risk category of the building

Category	Description
I	minimum human occupation (eg. storage)
III	substantial risk to humans
IV	essential facilities
II	none of the above (eg. Residences)

(2) Find the design wind speed, based on category and geographic location. The top diagram below shows the maximum speed map for category II, and the bottom map categories III and IV.

(3) Find the Exposure Category – depends on surface roughness

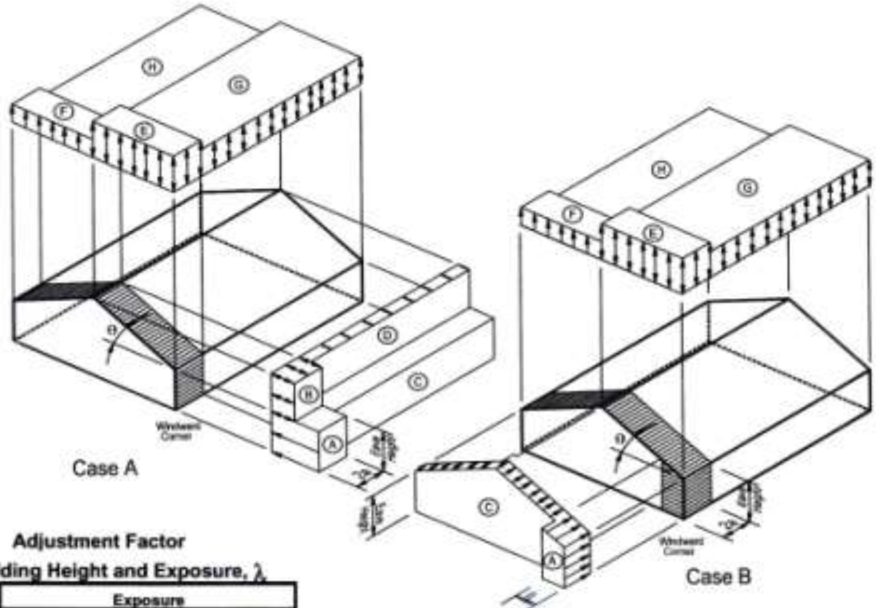
Surface Roughness Category	Characteristics	Exposure Category
B	urban or suburban – many single family residences	B
C	scattered obstructions	C
D	flat, unobstructed	D

Find Kzt, the Topographic factor – Is structure on a local hill or rise?
Kzt = 1 for no abrupt change in local elevation.
Kzt > 1 if there is an abrupt change in local elevation

(4) See figures on next page to find wind pressure pnet30. In the diagram below the dimension 'a' is given by the lesser of 10%of the least horizontal distance or 04.*h, and not less than 3 ft or 4% of the least horizontal dimension. Note that pnet30 is in strength terms.

(5) Find adjustment factor λ from table.

(6) Wind pressure = λ*Kzt*ps30
Multiply this figure by 0.6 to use Allowable Stress Design (ASD).



Adjustment Factor for Building Height and Exposure, λ

Mean roof height (ft)	Exposure B	Exposure C	Exposure D
15	1.00	1.21	1.47
20	1.00	1.29	1.55
25	1.00	1.35	1.61
30	1.00	1.40	1.66
35	1.05	1.45	1.70
40	1.09	1.49	1.74
45	1.12	1.53	1.78
50	1.16	1.56	1.81
55	1.19	1.59	1.84
60	1.22	1.62	1.87

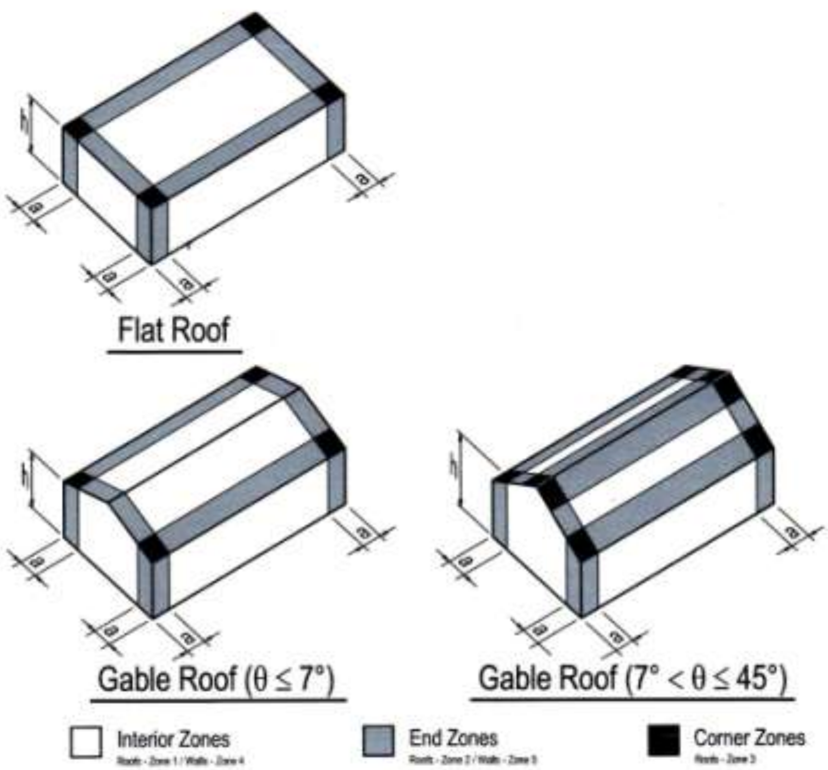
Simplified Design Wind Pressure, p_{S30} (psf) *(Exposure B at h = 30 ft.)*

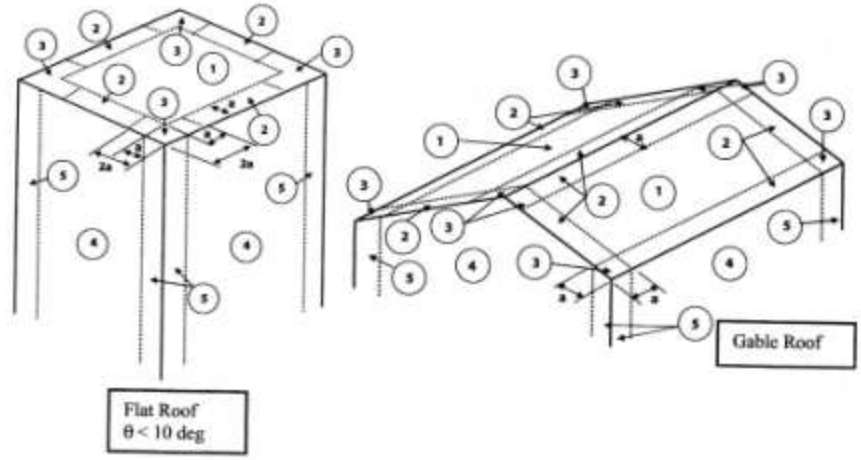
Basic Wind Speed (mph)	Roof Angle (degrees)	Load Case	Zones: Horizontal Pressures A	B	C	D	Vertical Pressures E	F	G	H	Overhangs E_{OH}	G_{OH}
160	0 to 5°	1	40.6	-21.1	26.9	-12.5	-48.8	-27.7	-34.0	-21.5	-68.3	-53.5
	10°	1	45.8	-19.0	30.4	-11.1	-48.8	-29.8	-34.0	-22.9	-68.3	-53.5
	15°	1	51.0	-16.9	34.0	-9.6	-48.8	-31.9	-34.0	-24.3	-68.3	-53.5
	20°	1	56.2	-14.8	37.5	-8.2	-48.8	-34.0	-34.0	-25.8	-68.3	-53.5
	25°	1	50.9	8.2	36.9	8.4	-22.6	-30.8	-16.4	-24.8	-42.1	-35.9
		2	——	——	——	——	-8.6	-16.8	-2.3	-10.7	——	——
	30 to 45	1	45.7	31.2	36.3	25.0	3.5	-27.7	1.2	-23.8	-16.0	-18.3
		2	45.7	31.2	36.3	25.0	17.6	-13.7	15.2	-9.8	-16.0	-18.3
180	0 to 5°	1	51.4	-26.7	34.1	-15.8	-61.7	-35.1	-43.0	-27.2	-86.4	-67.7
	10°	1	58.0	-24.0	38.5	-14.0	-61.7	-37.7	-43.0	-29.0	-86.4	-67.7
	15°	1	64.5	-21.4	43.0	-12.2	-61.7	-40.3	-43.0	-30.8	-86.4	-67.7
	20°	1	71.1	-18.8	47.4	-10.4	-61.7	-43.0	-43.0	-32.6	-86.4	-67.7
	25°	1	64.5	10.4	46.7	10.6	-28.6	-39.0	-20.7	-31.4	-53.3	-45.4
		2	——	——	——	——	-10.9	-21.2	-3.0	-13.6	——	——
	30 to 45	1	57.8	39.5	45.9	31.6	4.4	-35.1	1.5	-30.1	-20.3	-23.2
		2	57.8	39.5	45.9	31.6	22.2	-17.3	19.3	-12.3	-20.3	-23.2
200	0 to 5°	1	63.4	-32.9	42.1	-19.5	-76.2	-43.3	-53.1	-33.5	-106.7	-83.5
	10°	1	71.5	-29.7	47.6	-17.3	-76.2	-46.5	-53.1	-35.8	-106.7	-83.5
	15°	1	79.7	-26.4	53.1	-15.0	-76.2	-49.8	-53.1	-38.0	-106.7	-83.5
	20°	1	87.8	-23.2	58.5	-12.8	-76.2	-53.1	-53.1	-40.2	-106.7	-83.5
	25°	1	79.6	12.8	57.6	13.1	-35.4	-48.2	-25.6	-38.7	-65.9	-56.1
		2	——	——	——	——	-13.4	-26.2	-3.7	-16.8	——	——
	30 to 45	1	71.3	48.8	56.7	39.0	5.5	-43.3	1.8	-37.2	-25.0	-28.7
		2	71.3	48.8	56.7	39.0	27.4	-21.3	23.8	-15.2	-25.0	-28.7

The components and cladding (C&C) members, those receiving wind loads directly, are analyzed here by the simplified method of Part 2, Chapter 30, of ASCE 7-10. The buildings have a height <= sixty (60) feet' and have flat, gable, or hip roofs.

The loads are determined by the same six (6) step process as that used in the MWFRS analysis, with one difference, namely that in the MWFRS loads distributed loads are applied perpendicular to the projected surfaces (horizontal and vertical), while in the C&C method thew uniform loads are applied perpendicular to the building surfaces.

This page shows the dimensions of the critical areas and the following page the zone numbers and loads, in strength terms.

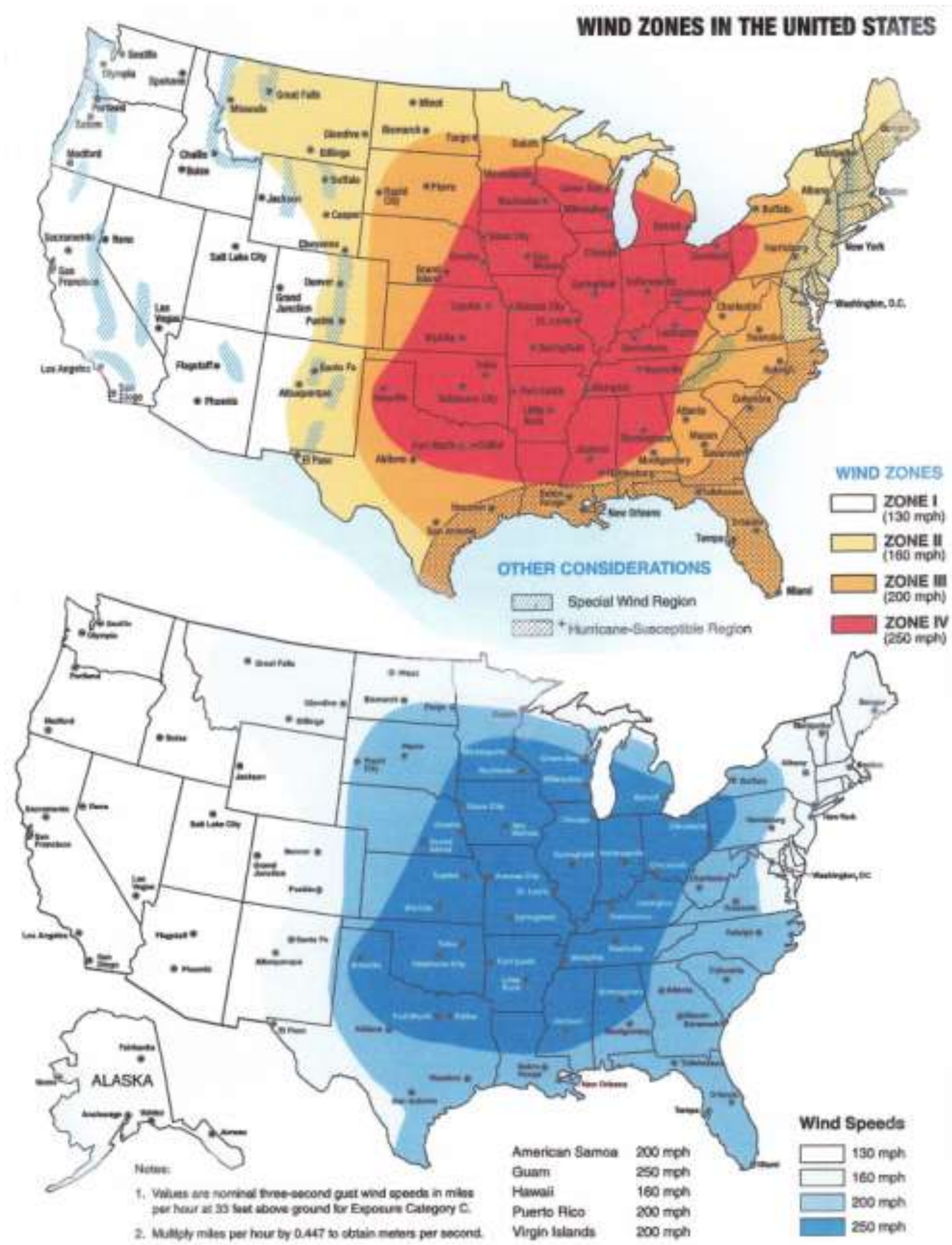




Net Design Wind Pressure, p_{net30} (psf) *(Exposure B at h = 30 ft.)*

	Zone	Effective wind area (sf)	Basic Wind Speed V (mph) 110		115		120		130		140		150		160		180		200	
Roof 0 to 7 degrees	1	10	8.9	-21.8	9.7	-23.8	10.5	-25.9	12.4	-30.4	14.3	-35.3	16.5	-40.5	18.7	-46.1	23.7	-58.3	29.3	-72.0
	1	20	8.3	-21.2	9.1	-23.2	9.9	-25.2	11.6	-29.6	13.4	-34.4	15.4	-38.4	17.6	-44.9	22.2	-56.8	27.4	-70.1
	1	50	7.6	-20.5	8.3	-22.4	9.0	-24.4	10.6	-28.6	12.3	-33.2	14.1	-38.1	16.0	-43.3	20.3	-54.8	25.0	-67.7
	1	100	7.0	-19.9	7.7	-21.8	8.3	-23.7	9.8	-27.8	11.4	-32.3	13.0	-37.0	14.8	-42.1	18.8	-53.3	23.2	-65.9
	2	10	8.9	-36.5	9.7	-39.9	10.5	-43.5	12.4	-51.0	14.3	-59.2	16.5	-67.9	18.7	-77.3	23.7	-97.8	29.3	-120.7
	2	20	8.3	-32.6	9.1	-35.7	9.9	-38.8	11.6	-45.6	13.4	-52.9	15.4	-60.7	17.6	-69.0	22.2	-87.4	27.4	-107.9
	2	50	7.6	-27.5	8.3	-30.1	9.0	-32.7	10.6	-38.4	12.3	-44.5	14.1	-51.1	16.0	-58.2	20.3	-73.6	25.0	-90.9
	2	100	7.0	-23.6	7.7	-25.8	8.3	-28.1	9.8	-33.0	11.4	-38.2	13.0	-43.9	14.8	-50.0	18.8	-63.2	23.2	-78.1
	3	10	8.9	-55.0	9.7	-60.1	10.5	-65.4	12.4	-76.8	14.3	-89.0	16.5	-102.2	18.7	-116.3	23.7	-147.2	29.3	-181.7
	3	20	8.3	-45.5	9.1	-49.8	9.9	-54.2	11.6	-63.6	13.4	-73.8	15.4	-84.7	17.6	-96.3	22.2	-121.9	27.4	-150.5
	3	50	7.6	-33.1	8.3	-36.1	9.0	-39.3	10.6	-46.2	12.3	-53.5	14.1	-61.5	16.0	-69.9	20.3	-88.5	25.0	-109.3
	3	100	7.0	-23.6	7.7	-25.8	8.3	-28.1	9.8	-33.0	11.4	-38.2	13.0	-43.9	14.8	-50.0	18.8	-63.2	23.2	-78.1
Roof > 7 to 27 degrees	1	10	12.5	-19.9	13.7	-21.8	14.9	-23.7	17.5	-27.8	20.3	-32.3	23.3	-37.0	26.5	-42.1	33.6	-53.3	41.5	-65.9
	1	20	11.4	-19.4	12.5	-21.2	13.6	-23.0	16.0	-27.0	18.5	-31.4	21.3	-36.0	24.2	-41.0	30.6	-51.9	37.8	-64.0
	1	50	10.0	-18.6	10.9	-20.4	11.9	-22.2	13.9	-26.0	16.1	-30.2	18.5	-34.6	21.1	-39.4	26.7	-49.9	32.9	-61.6
	1	100	8.9	-18.1	9.7	-19.8	10.5	-21.5	12.4	-25.2	14.3	-29.3	16.5	-33.6	18.7	-38.2	23.7	-48.4	29.3	-59.8
	2	10	12.5	-34.7	13.7	-37.9	14.9	-41.3	17.5	-48.4	20.3	-56.2	23.3	-64.5	26.5	-73.4	33.6	-92.9	41.5	-114.8
	2	20	11.4	-31.9	12.5	-34.9	13.6	-38.0	16.0	-44.6	18.5	-51.7	21.3	-59.3	24.2	-67.5	30.6	-85.4	37.8	-105.5
	2	50	10.0	-28.2	10.9	-30.9	11.9	-33.6	13.9	-39.4	16.1	-45.7	18.5	-52.5	21.1	-59.7	26.7	-75.6	32.9	-93.3
	2	100	8.9	-25.5	9.7	-27.8	10.5	-30.3	12.4	-35.6	14.3	-41.2	16.5	-47.3	18.7	-53.9	23.7	-68.2	29.3	-84.2
	3	10	12.5	-51.3	13.7	-56.0	14.9	-61.0	17.5	-71.6	20.3	-83.1	23.3	-95.4	26.5	-108.5	33.6	-137.3	41.5	-169.5
	3	20	11.4	-47.9	12.5	-52.4	13.6	-57.1	16.0	-67.0	18.5	-77.7	21.3	-89.2	24.2	-101.4	30.6	-128.4	37.8	-158.5
	3	50	10.0	-43.5	10.9	-47.6	11.9	-51.8	13.9	-60.8	16.1	-70.5	18.5	-81.0	21.1	-92.1	26.7	-116.6	32.9	-143.9
	3	100	8.9	-40.2	9.7	-44.0	10.5	-47.9	12.4	-56.2	14.3	-65.1	16.5	-74.8	18.7	-85.1	23.7	-107.7	29.3	-132.9
Roof > 27 to 45 degrees	1	10	19.9	-21.8	21.8	-23.8	23.7	-25.9	27.8	-30.4	32.3	-35.3	37.0	-40.5	42.1	-46.1	53.3	-58.3	65.9	-72.0
	1	20	19.4	-20.7	21.2	-22.6	23.0	-24.6	27.0	-28.9	31.4	-33.5	36.0	-38.4	41.0	-43.7	51.9	-55.3	64.0	-68.3
	1	50	18.6	-19.2	20.4	-21.0	22.2	-22.8	26.0	-26.8	30.2	-31.1	34.6	-35.7	39.4	-40.6	49.9	-51.4	61.6	-63.4
	1	100	18.1	-18.1	19.8	-19.8	21.5	-21.5	25.2	-25.2	29.3	-29.3	33.6	-33.6	38.2	-38.2	48.4	-48.4	59.8	-59.8
	2	10	19.9	-25.5	21.8	-27.8	23.7	-30.3	27.8	-35.6	32.3	-41.2	37.0	-47.3	42.1	-53.9	53.3	-68.2	65.9	-84.2
	2	20	19.4	-24.3	21.2	-26.6	23.0	-29.0	27.0	-34.0	31.4	-39.4	36.0	-45.3	41.0	-51.5	51.9	-65.2	64.0	-80.5
	2	50	18.6	-22.9	20.4	-25.0	22.2	-27.2	26.0	-32.0	30.2	-37.1	34.6	-42.5	39.4	-48.4	49.9	-61.3	61.6	-75.6
	2	100	18.1	-21.8	19.8	-23.8	21.5	-25.9	25.2	-30.4	29.3	-35.3	33.6	-40.5	38.2	-46.1	48.4	-58.3	59.8	-72.0
	3	10	19.9	-25.5	21.8	-27.8	23.7	-30.3	27.8	-35.6	32.3	-41.2	37.0	-47.3	42.1	-53.9	53.3	-68.2	65.9	-84.2
	3	20	19.4	-24.3	21.2	-26.6	23.0	-29.0	27.0	-34.0	31.4	-39.4	36.0	-45.3	41.0	-51.5	51.9	-65.2	64.0	-80.5
	3	50	18.6	-22.9	20.4	-25.0	22.2	-27.2	26.0	-32.0	30.2	-37.1	34.6	-42.5	39.4	-48.4	49.9	-61.3	61.6	-75.6
	3	100	18.1	-21.8	19.8	-23.8	21.5	-25.9	25.2	-30.4	29.3	-35.3	33.6	-40.5	38.2	-46.1	48.4	-58.3	59.8	-72.0
Wall	4	10	21.8	-23.6	23.8	-25.8	25.9	-28.1	30.4	-33.0	35.3	-38.2	40.5	-43.9	46.1	-50.0	58.3	-63.2	72.0	-78.1
	4	20	20.8	-22.6	22.7	-24.7	24.7	-26.9	29.0	-31.6	33.7	-36.7	38.7	-42.1	44.0	-47.9	55.7	-60.6	68.7	-74.8
	4	50	19.5	-21.3	21.3	-23.3	23.2	-25.4	27.2	-29.8	31.6	-34.6	36.2	-39.7	41.2	-45.1	52.2	-57.1	64.4	-70.5
	4	100	18.5	-20.4	20.2	-22.3	22.0	-24.2	25.9	-28.4	30.0	-33.0	34.4	-37.8	39.2	-43.1	49.6	-54.5	61.2	-67.3
	4	500	16.2	-18.1	17.7	-19.8	19.3	-21.5	22.7	-25.2	26.3	-29.3	30.2	-33.6	34.3	-38.2	43.5	-48.4	53.7	-59.8
	5	10	21.8	-29.1	23.8	-31.8	25.9	-34.7	30.4	-40.7	35.3	-47.2	40.5	-54.2	46.1	-61.7	58.3	-78.0	72.0	-96.3
	5	20	20.8	-27.2	22.7	-29.7	24.7	-32.4	29.0	-38.0	33.7	-44.0	38.7	-50.5	44.0	-57.5	55.7	-72.8	68.7	-89.9
	5	50	19.5	-24.6	21.3	-26.9	23.2	-29.3	27.2	-34.3	31.6	-39.8	36.2	-45.7	41.2	-52.0	52.2	-65.8	64.4	-81.3
	5	100	18.5	-22.6	20.2	-24.7	22.0	-26.9	25.9	-31.6	30.0	-36.7	34.4	-42.1	39.2	-47.9	49.6	-60.6	61.2	-74.8
	5	500	16.2	-18.1	17.7	-19.8	19.3	-21.5	22.7	-25.2	26.3	-29.3	30.2	-33.6	34.3	-38.2	43.5	-48.4	53.7	-59.8

The top diagram below is taken from "Taking Shelter from the Storm", FEMA P-320 (Reference 4). The bottom shows the safe room design speed (Ref. 3). Note that American Samoa, Guam, Hawaii, Puerto Rico, and the Virgin Islands are all in hurricane-susceptible regions.



3.2 Missile Impact

Reference 5, ICC 500, called for surviving a nine (9) pound missile impacting vertical faces at 40% of design wind speed, and impacting horizontal faces at 10% of design wind speed.

This Specification calls for testing of the structure or structure portions to resist the impacts. However, an estimate of design forces may be found from Reference 6, "Impact Dynamics of Rod Type Windborne Debris" as:

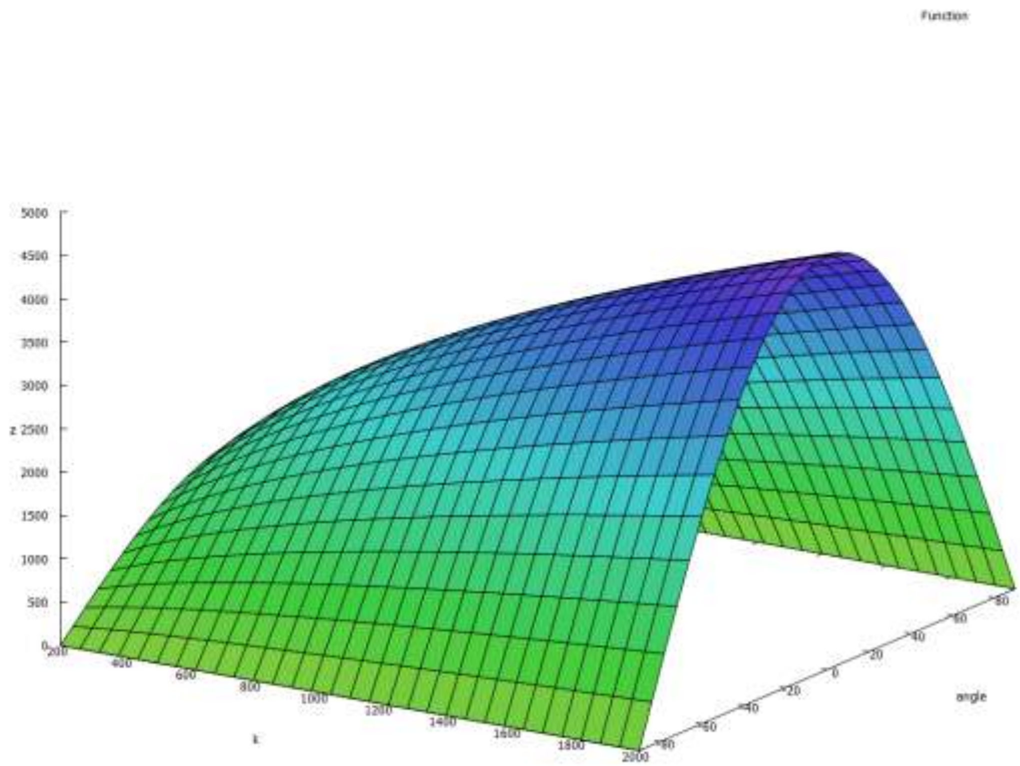
Fmax = V*(m*k)^1/2*cos(θ) where :

V = impact velocity. In./sec
m = missile mass, lbf-sec^2/in.,
= weight in lbf divided by 386.1 in./sec^2
(9 lbf 2x4 used for hurricane shelter)
k = structure stiffness, lbf/in.
θ = angle between missile path and perpendicular to surface

As an example for hurricane shelters, ASD, ICC 500-2008 uses 40% of design wind speed for vertical surface (10% for horizontal surfaces).
K = 987.9 lbf/in. from Reference 6
200 mph = 3520 in./sec

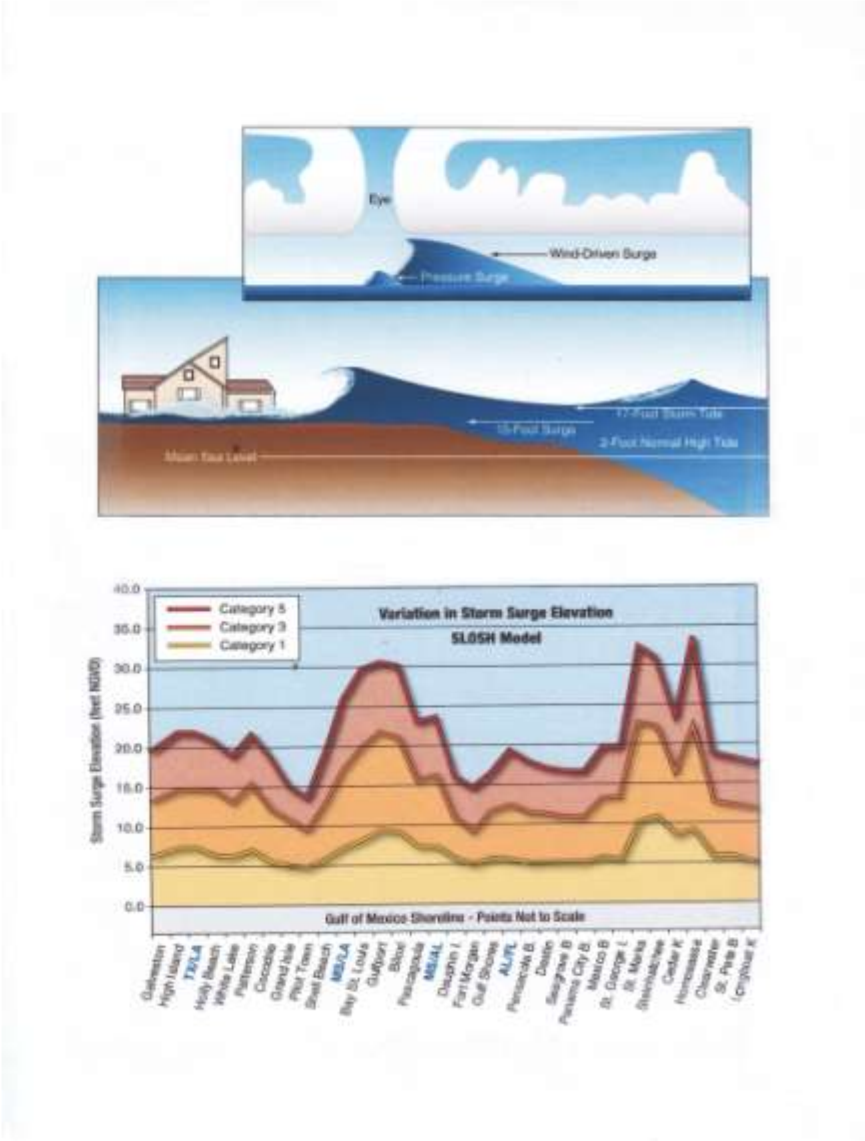
Fmax = .6*.4*3520*((9/386.1)*k)^1/2*cos(θ)

A plot of Fmax vs. k and θ is shown below.
Note that z = Fmax

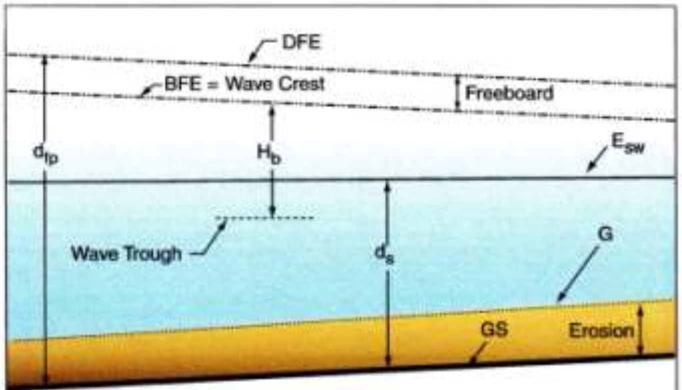


3.3 Storm Surge

Water that is pushed to shore by hurricane winds and low pressure is called a storm surge. The residence design must include this phenomenon in coast high hazard areas called "V-zones". The sketches below from "Recommended Residential Construction for Coast Areas" (Reference 7) show the hurricane moving ashore and variation in storm surge elevation for the Gulf Coast. This second chart is from the National Hurricane Center computer model, 'SLOSH' (Sea Lake and Overland Surge from Hurricanes).



The following figures show the various quantities used in design and a pictorial showing erosion and scouring.



DFE = Design Flood Elevation in feet above datum

d_{fp} = Design flood protection depth in feet

BFE = Base Flood Elevation in feet above datum

Freeboard = Vertical distance in feet between BFE and DFE

H_b = Breaking wave height = $0.78d_s$ (note that 70 percent of wave height lies above E_{sw})

E_{sw} = Design stillwater flood elevation in feet above datum

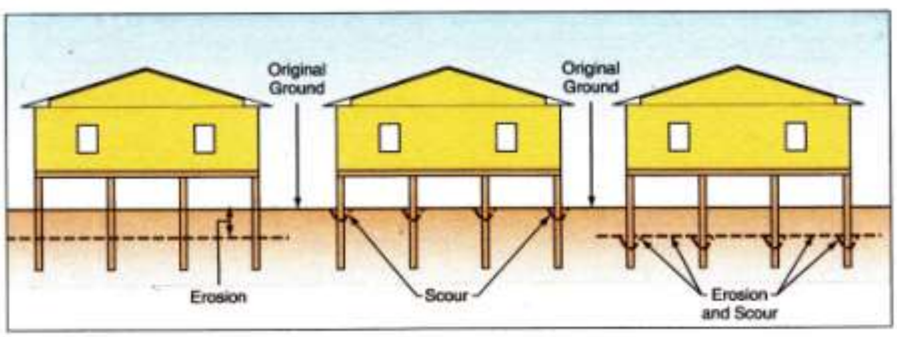
Wave Trough = Lowest part of the wave between crests

d_s = Design stillwater flood depth in feet

G = Ground elevation, existing or pre-flood, in feet above datum

Erosion = Loss of soil during design flood event in feet (not including effects of localized scour)

GS = Lowest eroded ground elevation adjacent to building in feet above datum (including the effects of localized scour)



V = design wave velocity in the V-zone
V = (g*ds)^(1/2)
g = 32.2 ft/sec^2
ds = design Stillwater flood depth, ft (see last page)

Hydrostatic loads need to be considered if there is a difference in water elevation on opposite sides of a member, which is not the case for pier support.
Buoyancy force = γ*(submerged volume)
γ = 64.0 lbf/ft^3 for salt water

Fbrkp = force from breaking waves on vertical piles or columns, lbf
Fbrkp = (CD)*γ*D*Hb^2/2.0
Cd = 1.2 for round piles
(CD) = coefficient of drag (1.75 for circular piles, 2.25 for square columns), dimensionless
D = pile diameter for circular section, 1.4 for square section, ft
Fdyn = 0.5*Cd*fluid density*V^2*area affected
Hb = breaking wave height (see last page)

Fi = debris impact load, lbf
Fi = Π*W*Vb*Ci*Co*Cd*Cb*Rmax/2*g*Δt
W = weight of debris, 1000 lbf if no other information available. Note that this does not include extreme impact caused by boats, barges, or collapsed buildings, which should considered during choice of a site.
Vb = debris velocity, use velocity of water V, Ft/sec
Ci = Importance coefficient, 0.6, 1.0, 1.2, 1.3 for Categories I, II, III, and IV respectively
Co = orientation coefficient = 0.8
Cd = depth coefficient, 1.0 for V-zone
Cb = blockage coefficient, 1.0 if flow path > 30 ft and no upstream screening
Δt = impact duration, varies from 0.01 to 0.05, use 0.03 if no other information or calculation available
Rmax = response factor for impulsive loads, from the following table where quotient = impact duration divided by natural period of structure

quotient	Rmax
0.00	0.0
0.10	0.4
0.20	0.8
0.30	1.1
0.40	1.4

0.50	1.5
0.60	1.7
0.70	1.8
0.80	1.8
0.90	1.8
1.00	1.7
1.10	1.7
1.20	1.6
1.30	1.6
>= 1.40	1.5

Also to be considered are erosion and scour. Erosion is the lowering of the ground surface by flood(s) or gradual erosion of the shore line. This increases the flood protection depth as well as lowering the pile embedment.
Scour refers to localized lowering of ground surface during a flood, which lowers pile embedment.

Design load combinations used here for Allowable Stress Design are:
(1) 0.6*DL + W
(2) 0.6*DL + 1.5*Fa + W
DL = dead load Fa = flood load W = wind load

Fa = Fbrkp on seaward piles, Fdyn on all other piles, and Fi on corner or critical pile only.

4. HURRICANE RESISTANT STRUCTURE – NOT IN FLOOD ZONE

4.1 Introduction

Light wood frame buildings ,used here, are characterized by construction with dimension lumber, wood sheathing, and wall footers. They transfer vertical loads by closely spaced (12″ to 24″ on center) rafters (or trusses) and columns, thence to the foundations and to the earth.

The load transfer of lateral loads is either two dimensional by moment-resisting frames or three dimensional box structures transferring load as impacted plane → roof (or ceiling) diaphragm → shear wall → foundation.

A complicating factor is that some of the lateral load is resisted by the moment-resisting frame, as well as by the box system. However, designing the frame to resist lateral loads by itself leads to a very inefficient structure. On the other hand, designing the frame to resist no lateral loads is not conservative, as the lateral support is not rigid, but has both diaphragm and shear wall deflections.

The approach taken here is design the box system to resist the full lateral loads and use finite element analysis of the basic transverse frame, with horizontal axial elements at the eaves to account for the box system support of the lateral loads. This implies that the horizontal (roof and/or ceiling) and vertical (wall) diaphragms must first be designed to find these non-rigid axial elements. The two-dimensional frame, by itself, must support the vertical loads (snow load, live load, dead, and vertical component of the wind load). In this analysis only dead, snow, live and wind loads are considered.

4.2 Example Definition

The structure considered is a light wood frame gable type residence with the following characteristics:

Importance Category II
Exposure C
Not in flood zone
Nominal plan dimensions = 24′ x 24′
Finished floor to top of double plate = 8′-0″
Roof slope = 3 in 12 (14.04°)
Transverse frames 16″ on center
No basement, 18″ minimum crawl space
Design wind = 150 mph, by the ASD (Allowable Stress Design) method
Vertical gravity loads :
- **Roof Load = 50 psf (snow)**
- **Roof DL = 10 psf**
- **Ceiling LL = 20 psf**
- **Ceiling DL = Wall DL = Floor DL = 10 psf**

Floor LL = 40 psf

Soil = sand or silty sand, with the following characteristics:

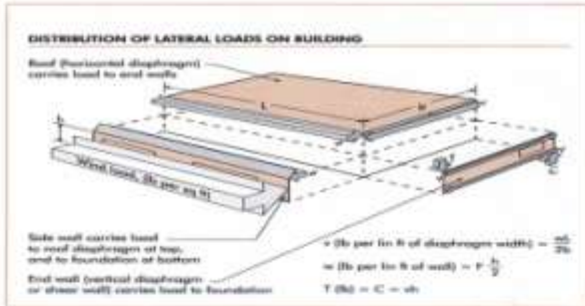
Maximum bearing pressure = 2000 psf
Lateral bearing = 150 psf/ft
Coefficient of friction = 0.25
(To be multiplied by dead load only in determining sliding resistance)

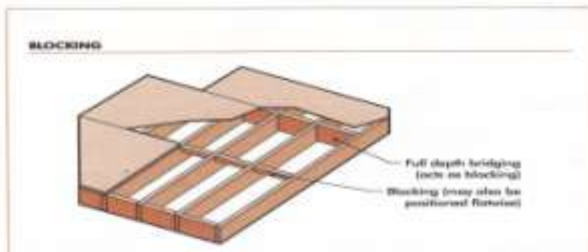
4.3 Box System

As mentioned previously, the box system for resisting lateral loads consists of horizontal diaphragms (roof and/or ceiling planes) and vertical diaphragms (walls) which act as vertical cantilevers, taking the lateral loads from the horizontal diaphragms to the foundation.
The development here follows that given, in general, in Reference 8, "Design of Wood Structures".

The horizontal diaphragms act as deep beams to carry the lateral load to the shearwalls. The plane sheathing acts as the beam web, the plates perpendicular to the wind (chords) acting as flanges, and the plates parallel to the wind (collectors or struts) acting as simple end connections.

The following diagrams from Reference 9, "Diaphragms and Shearwalls", illustrate the lateral force transmission method, and also illustrate blocking. Blocking is used both to provide nailing surfaces for sheathing and to provide weak axis support for axial and flexural loads in rafters and columns.





4.4 Wind Forces

Two sets of forces are required to be resisted by the structure. The first, the Main Wind Force Resisting System (MWFRS), applies to all the individual members and to the box system and foundation. The second, Components and Cladding (C&C), applies to the elements directly impacted by winds. These elements include roof sheathing, wall sheathing, rafters, and exterior columns. These elements must be checked for both MWFRS and C&C, the more severe governing.

The MWFRS applies loads perpendicular to the horizontal and vertical axes, and are tabulated in terms of roof angles.

The C&C method, on the other hand, specifies loads perpendicular to the element of interest, and are tabulated in terms of areas.

Both methods define wind boundaries in terms of "a", defined as :
<= 0.1 times least horizontal dimension
<= 0.4 times roof height
but not less than 3 feet or 0.04 times the least horizontal dimension. In this example, a = 3 ft

The ASCE 7-10 tables are meant to be used the wind diagrams in the table. These wind values incorporate a factor which converts the actual wind speeds to strength values. Conversion of these wind values to ASD values is given by: Vasd^2 = 0.6Vstr^2 → Vstr = Vasd/0.6^(1/2).

For the ASCE 7-10 tables to be converted to ASD values, the following procedure is recommended :

(1) **Strength wind value = 150/0.6^(1/2) = 193.649 mph**
(2) **Interpolate between 10° and 15° for 180 mph**
(3) **Interpolate between 10° and 15° for 200 mph**
(4) **Interpolate between 180 and 200 mph for 193.649 mph**
(5) **Convert to ASD pressure by multiplying by 0.6.**
(6) **Convert to Exposure C by multiplying by 1.21.**

For the ASCE 7-05 method:

(1) **Interpolate between 10° and 15° for 14.036°**
(2) **Convert to Exposure C by multiplying by 1.21.**

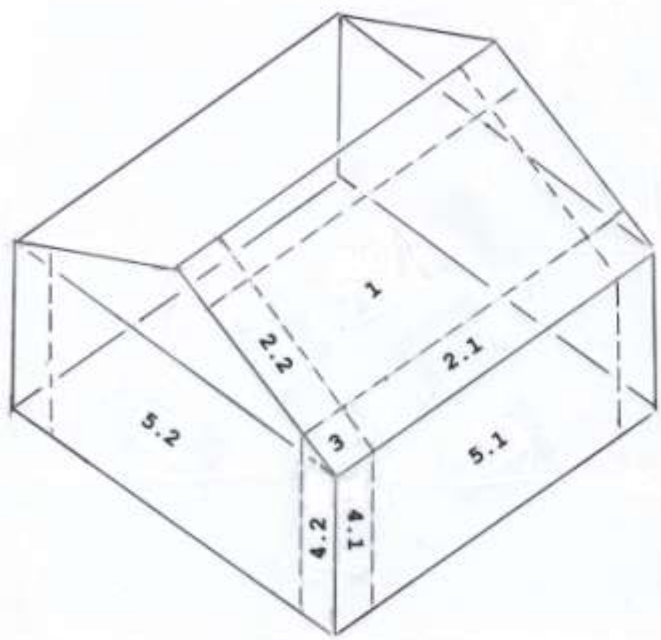
Compare MWRFS pressures for the two methods for Exposure C, actual wind design speed = 150 mph.

Area	ASCE 7-10	ASCE 7-05
A	+53.3	+53.1
B	-18.4	-18.4
C	+35.5	+35.4
D	-10.5	-10.6
E	-52.0	-51.9
F	-33.5	-33.5
G	-36.2	-36.1
H	-25.5	-25.6

Now the total loads on the structure can now be found.

Side wall horizontal load = 6767.915 lbf
End wall horizontal load = 8540.261 lbf
Uplift = 19498.602 lbf

The table in Part 3 for C&C pressures gives the maximum (towards the surface) and negative (away from the surface) values for the five (5) defined planar regions. The values in each region depend upon the region surface area. Thus region 2 at the roof edges will have a different pressure value from region 2 in the interior region. These regions are defined in the following figure, where all edge strips are three (3) feet wide:



Regions are symmetric about both
longitudinal and transverse center lines

Our approach here is to use positive pressure values for the windward side and negative pressure values for the leeward side, which maximizes lateral forces on the elements. The dividing line between windward and leeward forces is taken to be the longitudinal center line for side wall wind. The dividing line for end wall wind is taken to be the transverse center line.

Since the problem statement is given in ASD terms, and the table of pressures is given in strength terms, conversion is necessary as part of the load determination process. This process consists of four (4) parts :

(1) **Find strength wind speed for given ASD wind speed.**
For the wind speeds to be equivalent,
Vasd^2 = .6*Vstr^2 → Vstr = Vasd/(.6)^(1/2)
In this example, Vstr = 193.649 mph, and lies between 180 **mph and 200 mph values.**

(2) **Interpolate area of section. For example,**
Area 2.1 = 3(18) = 54 ft^2

Area	180 mph	200 mph
50 ft^2	26.7 -75.6	32.9 -93.3
100 ft^2	23.7 -68.2	29.3 -84.2

Interpolation of area gives
54 ft^2 26.530 -75.008 at 180 mph and
32.612 -92.572 at 200 mph

(3) Interpolate between 180 mph and 200 mph
for 193.649 mph.
Positive pressure (strength) = 30.681 psf
Negative pressure (strength) = 89.995 psf

(4) **The pressures in the part 3 table are for Exposure B, so for ASD Exposure C, multiply the values in (3) by 0.6*adjustment factor for Exposure C, h < 15 ft.**
Factor = 0.6*1.21
Positive pressure (ASD, Exposure C) = 22.274 psf
Negative pressure (ASD, Exposure C) = 63.158 psf

The results of these calculations for all eight (8) areas are :

Area	Ft^2	+ psf	-psf
1	114.648	20.004	40.787
2.1	54.000	22.274	63.158
2.2	19.108	25.893	72.916
3	9.000	28.308	115.680
4.1	24.000	46.488	50.640
4.2	24.750	46.414	50.566
5.1	144.000	41.194	49.904
5.2	171.000	40.248	49.213

Now the wind loads per horizontal foot may be found.
The suffix ww = windward and lw = leeward.
The angle θ = tan(-1)(3/12) = 14.04°

Wind on Side Wall

w1 = load/horizontal foot on edge areas, lbf/ft
w2 = load/horizontal foot on interior area, lb/ft

w1 = 4.1ww*4 + 2*3ww*3*tanθ + 2.2ww*6*tanθ +
2*3lw*3*tanθ + 2.2lw*6*tanθ + 4.1*lw*4
w1 = 752.708 lbf/ft

```
w2   =    5.1ww*4 + 2*2.1ww*3*tanθ  + 1ww*6*tanθ +
          2*2.1lw*3*tanθ + 1lw*6*tanθ + 5.1lw*4
w2   =    583.726 lbf/ft

Wind on End Wall
----------------
w3   =    load/horizontal foot on edge areas, lbf/ft
w4   =    load/horizontal foot on interior area, lbf/ft

w3   =    4.2ww*4 + 4.2ww*(.75/3) + 4.2lw*4 + 4.2lw*(.75/3)
w3   =    412.165 lbf/ft

w4   =    5.2ww*4 + 5.2ww*1.5 + 5.2lw*4 + 5.2lw*1.5
w4   =    495.336 lb/ft

Total Load, Side Wall Wind = 6*752.708 + 18*583.726
                           = 15023.316 lbf
Reaction, each side        =  7511.658 lbf = R
Moment                     = R*12 – w1*3*10.5 – w2*9*4.5
                           = 42788.691 lbf-ft

Total Load, End Wall Wind  = 6*412.165 + 18*495.336
                           = 11389.038 lbf
Reaction, each side        =  5694.519 lbf = R
Moment                     = R*12 – w3*3*10.5 – w2*9*4.5
                           = 35289.922 lbf-ft
```

4.5 Horizontal Diaphragms

The sloped roof is idealized as flat.
Design considerations are:
(1) **Out-of-plane normal loads**
(2) **Unit shear**
(3) **Uplift**
(4) **Nailing requirements**
(5) **Chords**
(6) **Collectors**
(7) **Deflection**

Int this and the following section, references to Tables and Figures is made to Reference 10, "Special Design Provisions for Wind and Seismic 2008".

(1) **Out-of-plane normal loads**

These loads model the panel as a beam continuous over supports. From Table 3.2.2, "Nominal Uniform Load Capacities (psf) for Roof Sheathing Resisting Out-of-Plane Loads", unless noted otherwise. For sheathing grade wood structural panels, 24/16 span ratio, 7/16" thick, and strong axis perpendicular to supports:

Rafter Spacing	Uniform Load
12 inches	540 psf
16 inches	365 psf

These are strength (ultimate) loads and must be divided by 1.6 (ASD reduction for out-of-plane loads) to obtain loads to be used for ASD analysis.
Note that the span ratio indicates the panel and span 24 inches roof load and 16 inches floor load.

(2) **Unit shear**

This is caused by the horizontal wind load on the upper wall half and onto the sloping roof. From section 4.4 the unit shear at transverse wall ends is 753 lbf/ft and unit in the interior 584 lbf/ft (both ASD values).

(3) **Uplift**

This term is greatest at areas 3, namely 116 lbf/ft. Considerations include nail pullout and panel pull-through.

(4) **Nailing requirements**

As shown in Table 4.2.4, the following conditions must be met to obtain a ahear capacity greater than 753 lbf/ft, namely 1595/2 = 798 lbf/ft. The factor 2 converts from the strength method to ASD. This capacity is required at the edges defined by areas 2.2 and 3.

- **APA rated sheathing, 7/16 in. thick**
- **Panel specific gravity >= 0.42 (Southern Pine)**
- **Common nail size = 8d (eight penny)**
- **Full blocking**
- **Minimum fastener penetration into framing member or blocking = 1-3/8 in.**
- **Minimum nominal width of nailed face at adjoining panel edges and at boundaries = 3 in.**

- **Nailing spacing at diaphragm boundaries = 2-1/2 in.**
- **Panel layout = Case 1 – same geometry in plan as masonry running bond in elevation.**

The interior of the diaphragm, where unit shear = 584 lbf/ft, can use the same criteria above except that the nominal width of adjoining panel edges need only be two (2) inches. It may, however, be less confusing to use the more restrictive three (3) inch specification above. This implies that rafters and blocking at adjoining panel inches be of 3 in. width. Field nailing of diaphragms is not specified in Reference 10.

Reference 11, "Retrofitting a Roof for High Wind Uplift" gives, for edge areas, 8d common nail spacing at panel edges, and six (6) inch field nail spacing. This reference also recommends deformed shank (either ring shank or spiral shank) nails for hurricane conditions. The grooves in these nails increase the friction between the shank of the nail and the adjacent wood fibers, increasing withdrawal resistance. Deformed shank nails also have larger heads to improve pull-through resistance.

(5) **Chords**

The axial force in the chords is found by resolving the moment in the diaphragm into a couple, as:
Chord force = Fchord = moment/width
In the example here,
Fchord = 42789/24 = 1783 lbf
The chords are physically realized by the upper top plates, and may be required to be spliced if single members the length of the chords are not available.

(6) **Collectors (Drag struts)**

The force in the collector, also realized by a double top plate, may be conservatively taken as the unit shear in the diaphragm times the sum of the lengths of the wall openings. The connections at both ends of the collectors, as well as the collector itself, are designed for this force.
In this example, for a single 48" wall opening,
Fcollector = 4*753.708 = 3015 lbf

Now the roles of collector and strut can be reversed, depending upon the wind direction. Thus the larger of the two vales in (5) and (6) should be used for design.

(7) **Deflection**

Deflection of the diaphragm depends upon bending deflections, shear deflections, fastener deformation, and chord splice slip, shown in equation form as:

δdia = 5*ν*L^3/8*E*A*W + 0.25*ν*L/1000*Ga + slip due to chord splice + offset, where:

E = chord modulus of elasticity, psi
A = chord cross-section area, in.^2
Ga = apparent diaphragm shear stiffness from nail slip and panel shear definition, kips/in.
L = diaphragm length, ft
ν = induced unit shear in diaphragm, lbs/ft
W = diaphragm width, ft
Chord splice slip + offset, at the induced unit shear, in.
δdia = maximum mid-span deflection, in.

In this example,
E = 1600000 psi (Southern Pine, No. 2}
A = 10.88 in.^2, (2x8)
Ga = 14 kips/in.
L = 24 ft
ν = 753 lbf/ft
W = 24 ft
Estimated third term, including offset of splice members = 2*0.072 in. = 0.144 in. (Example C4.2.2.3)
Assume full length chords, no splices
δda = .0156 + .3227 + .1440
δda = .482 in.

4.6 Shear Walls

Shear walls are the vertical elements in a box system resisting lateral loads. They support the horizontal diaphragm in conjunction with the transverse frames, and transfer the lateral loads from the horizontal diaphragm to the foundation. Design considerations are:

(1) **Types of analysis**

(2) **Shear panel proportions**
(3) **Sheathing thickness and nailing pattern**
(4) **Chords**
(5) **Deflection**
(6) **Requirements to resist combined shear and uplift**

(1) **Types of analysis**

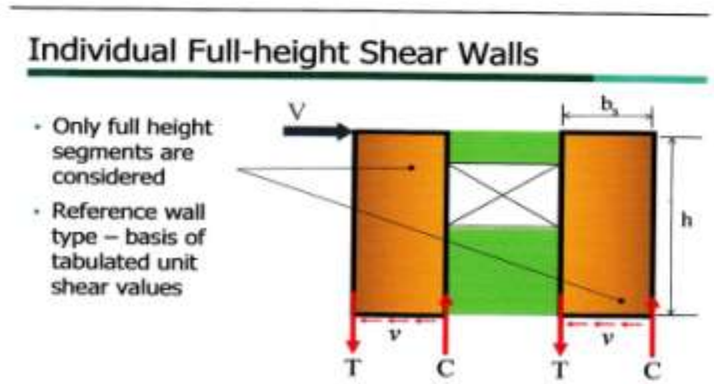
There are three (3) types of analysis, the segmented shear wall method, the force transfer around openings method, and the perforated shear wall design method.

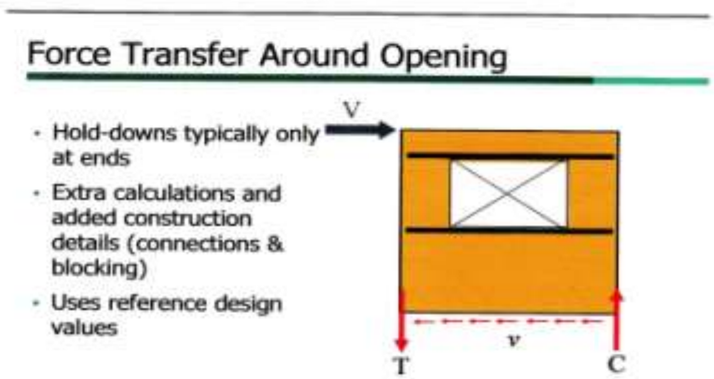
The first method, the segmented shear wall type, assumes only full-height segments of the wall are effective in resisting wind forces. No resistance credit is given to segments where doors, windows, or other openings are present. The unit shear force is calculated to be the load delivered by the diaphragm through the collector divided by the sum of the widths of the full-height segments. Each end of each line of full-height segments must be anchored to the foundation by a hold-down bracket. This method is the traditional approach.

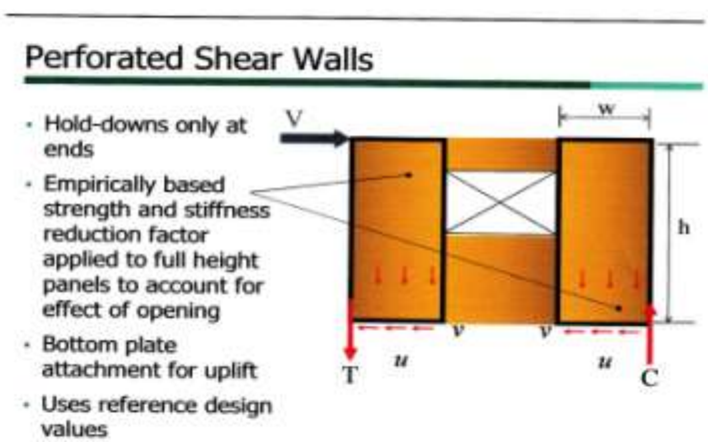
The second method, the force-transfer method, is applicable to walls with openings, but framing members, blocking, and connections around openings are designed for force transfer.

The third method, labeled perforated shear walls, reduces, but not eliminate, the shear strength of panels with openings. The capacity is based on empirical equations and tables. This method, as well as the second method, requires only hold-downs at the wall ends.

The methods are illustrated in the following diagrams of each type, as shown in reference 12, "Shear Wall Design with Examples".

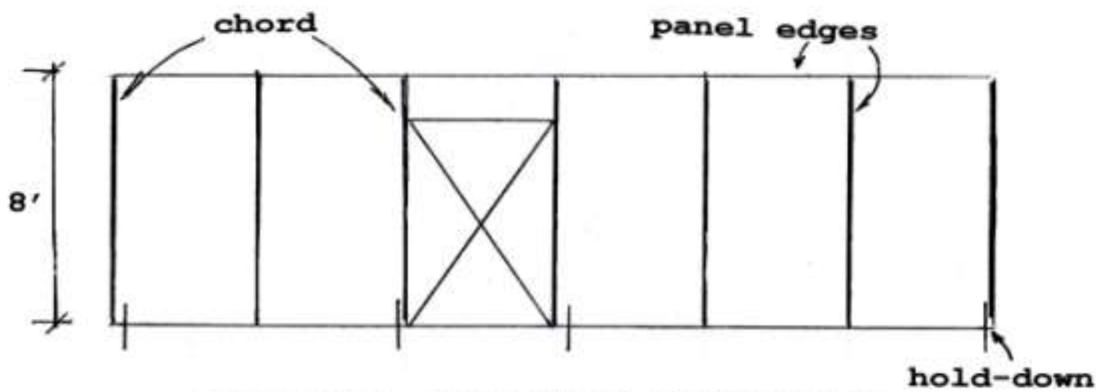




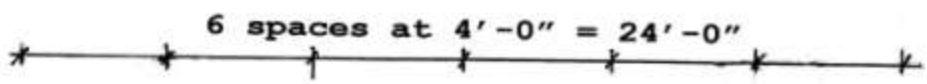


The individual full-height wall segment shear wall method The first above) is used here. Consider the following wall with a single four (4) foot door opening. Here the larger of the resultant collector

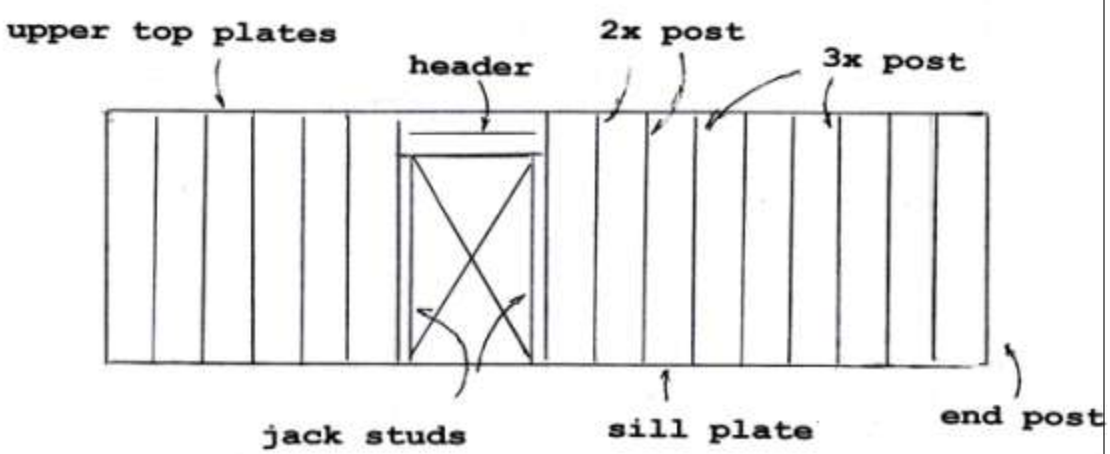
forces is used, so that the calculations following may be used for either the side or end wall with a total of 4′ openings.



SIDE WALL SEGMENTED SHEAR WALL



SIDE WALL FRAMING MEMBERS



Unit shear = 7511.1/(8+12) = 376 lbf/ft

(2) Shear wall proportions (Table 4.3.4)

The maximum shear wall aspect ratios for wind:
Wood structural panels, unblocked = 2:1
Wood structural panels, blocked = 3.5:1

Aspect ratio = height of shear wall segment divided by the width of the segment.

(3) Sheathing thickness + nailing pattern

Here the maximum loads normal to the wall surface are +47 psf and -51 psf (area 4.1). Table 3.2.1 gives maximum normal loads of 90/1.6 = 56 psf, for 15/32″ thick wood structural panels. Table 4.3A is used to find minimum thickness.
A 15/32″ thick wood structural panel with 8d common nails, panel edge spacing of 4″ has an ASD capacity of 1065/2 = 532 lbf/ft > 376 lbf/ft.
Here the minimum nail penetration is 1-3/8″.
These values apply to only Douglas Fir Larch (DFL) or Southern Pine (SP). The panel joints At the 4″ spacing must be offset to fall on different framing members or the nailed face of the framing member shall be 3″ nominal or greater at adjoining panel edges and nails at all panel edges shall be staggered.

(4) Chords

The vertical members at the end of an unbroken line of full-height panels are the chords of the vertical diaphragm. The force in each chord may be tension or compression, and is equal to the unit shear times the panel height. For two contiguous full height four (4) foot wide panels,
Lateral force = 8*376 = 3008 lbf.
Moment = 3008*8 = 24064 lbf-ft
Chord force = moment/arm = 24064/8 = 3008 lbf
Now consider three contiguous full height panels.
Lateral force = 12*376 = 4512 lbf
Moment = 36096 lf/ft, chord force = 36096/12 = 3008 lbf.
Thus, for contiguous full-height panels, chord force = panel height times unit shear.

(5) Deflection

δsw = 8*ν*h^3/E*A*b + ν*h/1000*Ga +h*Δa/b where:
δsw = maximum shear wall deflection, in.
The first term represents flexural deflection of the vertical cantilever and the second term the shear deflection. Numeric values are given for

this example.

- ν = induced unit shear, lbf/ft = 376 lbf/ft
- h = shear wall height, ft = 8 ft
- E = End posts Young's modulus,psi = 1600000psi
- A = end post cross-section,in.^2 = 18.13 in.^2
- Ga = apparent shear wall stiffness from Table 4.3A, kip/in.= 13 kip/in.
- Δa = total vertical elongation of wall, in. estimate 1/16 in.
- b = shear wall length, ft, = 24 ft
- δsw = .0022 + .2314 + .0208 = 0.254 in.

(6) Requirements to resist combined shear and uplift

- Panels to be not less than 4' x 8', except at boundaries and changes in framing, such as end wall framing above 8'.
- Panels have a minimum thickness of 7/16".
- Openings to have framing and connections to resist and transfer the uplift loads around the opening and into the foundation.
- Sheathing top plate and bottom plate connections as shown in Figure 4G on page 36 of SDPWS.

4.7 Transverse Frames

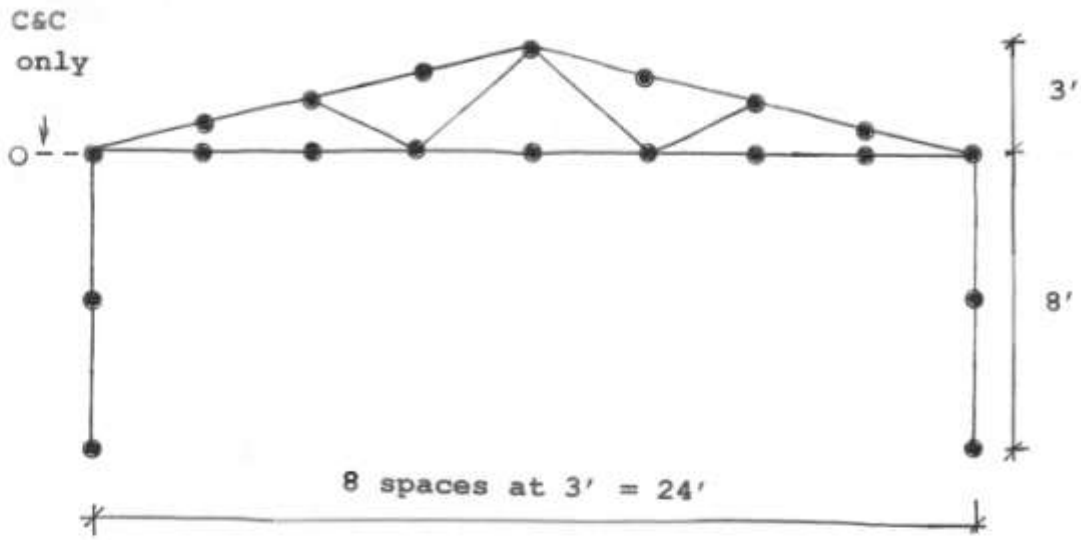
The transverse frames provide full support for gravity loads (DL,LL,SL) and partial support for lateral loads. It is too conservative to design these frames to withstand lateral loads (wind) by themselves. On the other hand, the box system has a finite deflection under load, which means the frames must deflect by the same amount.
The design here uses a fictional horizontal eave element to give the restraining effect of the box system. In this example, this element is adjusted to to provide approximately 0.6 inches of lateral eave deflection under C&C loads. This element is not used in the gravity load calculations.

A metal-plate connected Fink truss (also called a “W” truss) significantly reduces stresses on the rafters and ties. Members and loads are shown on the following page, as well as the finite element outline.

In the finite element model, connections between members are treated as semi-rigid. For the ridge and rafter and tie ends, the rotational stiffness is taken as 4000 kip-in./radian. All others are set to 400 kip-in./radian, except for the column bases, which are assumed to be pinned. It should be noted that considerable difference in moments are calculated among all-pinned, semi-rigid, ands all-fixed connections.

Finite element analysis gives:

Member	Load	Axial(lbf)	Moment(lbf-in.)
Column	C&C	1907 (T)	6564
	Gravity	1423 (C)	1459
Tie	C&C	5596 (C)	2618
	Gravity	3634 (T)	3772
Rafter	C&C	4751 (T)	11315
	Gravity	3489 (C)	4567
Diagonal	C&C	2172 (T)	521
	Gravity	568 (C)	304




```
Wall Studs = 2x10, Spruce-Pine_Fir No. 2
Ties       = 2x8, Southern Pine No.1
Rafters    = 2x8, Southern Pine No. 1
Diagonals  = 2x8, Spruce-Pine-Fir No.2

Total Gravity Loads : Rafters = 60 psf
                      Ties = 30 psf
                      Columns = 10 psf
```

Wind Loads (C&C) in psf

Component	Windward	Leeward
Wall Studs	+46.5	-50.6
Rafters, 3′ edges	+28.3	-115.7
Rafters	+ 25.9	-72.9

Frames 16 inches on center.

It is seen that the axial loads of tension and compression are reversed between the gravity and wind loads. Moment sign of each member also changes. Thus the roof diaphragm provides continuous lateral flexural support for the gravity loads, but does not for the C&C loads. In this case, blocking provides lateral flexural support.

The bracing distances of interest are:

lu = weak axis flexural support, in.
lx = strong axis compression upport, in.
ly = weak axis compression support, in.

For this example the following support lengths are calculated. For compression, K = 1. Blocking at 24″ is assumed.

Member	lu(in.)	lx(in.)	ly(in.)
column	0	96	24
tie	24	108	24
rafter	24	74.2	24
diagonal	50.9	50.9	50.9

To calculate allowable member forces, lumber grades must be chosen. The maximum stress for the types used here , in psf, are:

Stress	SPF No.2	SP No.1 (8″ wide)
Fb (flexural)	875	1500
Fc (compression, ∥ to grain)	1150	1650
Ft (tension)	450	825
Fv (shear)	175	175
E (modulus of	1400000	1600000

Shear calculations are not shown because the cross-section of the members chosen render the shear stresses much lower than the allowable types.

Also used in the calculations of the allowable loads are two factors, namely:

CD = load duration factor, 1.15 for gravity loads (2 months), and 1.25 for wind (7 days)
CR = repetitive member factor – applies to Fb equals 1.15

Calculation of stresses are done in the usual manner, using actual, as opposed to nominal, dimensions. For example, a 2x8 member is actually 1-1/2″ x 7-1/4″.

After calculation of member stresses, two distinctly different combinations must be considered, axial tension with bending, and axial compression with bending.

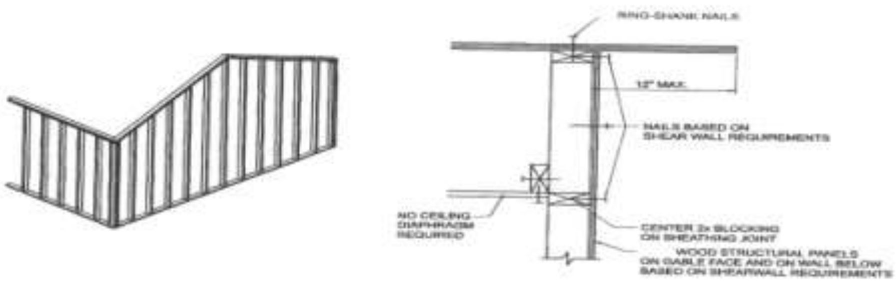
For the tension case it must be true that :
f+t = ft/Ft* + fb/Fb* < 1
and
(fb-ft)/Fb < 1 where**
Ft* = CD*Ft
Fb = CD*CR*Fb*fraction depending on flexural Bracing**
For the compression case it must be true that
fc/Fc < 1**
and
f+c = (fc/Fc)^2 + fb/(Fb*(1-fc*FcE)) < 1 where**
Fc =CD*Fc(reduction for axial bracing)**
FcE = factor dependent on unbraced length

Appendix I shows a detailed analysis of these stresses, as well as a program for calculating the above factors, for the interested reader.

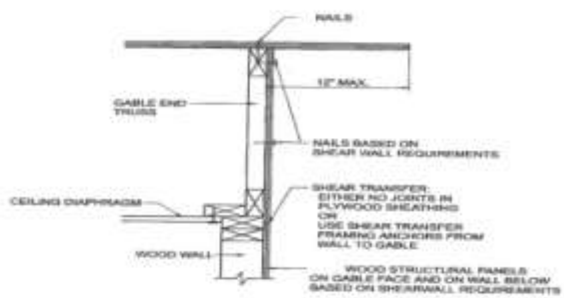
Member	tension		compression	
	f+t	(Fb-ft)Fb**	f+c	fc/Fc**
column	.386	-.138	.100	.127
tie	.467	-.025	.519	.862
rafter	.788	+.206	.262	.318
diagonal	.386	-.132	.035	.025

4.8 End Wall Frame

There are two methods of end wall construction, balloon framing and platform framing. In balloon framing, the studs extend from the sill plate to the roof rafter. In platform framing, each floor level is framed as a separate unit or platform. Reference 13, "Guidelines for Hurricane Resistant residential Framing", illustrates the basics of the two types, as shown below.



GABLE ENDWALL, BALLOON FRAMING
PREFERRED METHOD



GABLE ENDWALL, PLATFORM FRAMING
ALTERNATE METHOD

For wind loads perpendicular to the side walls, the end wall frame functions as a shear wall, as described in 4.5 above. The frame rafter acts as the collector.
For wind loads perpendicular to the end walls, the end wall frame resists the wind loads as a series of vertical beams, spaced at 16″ on center., framed between the roof rafter and the sill plate.
Let the posts be 2x8 SP No.2, except at panel boundaries, Where they are taken to be 3x8 SP No.. The increased width is needed for nailing at the panel boundaries. The same reasoning applies to the roof rafter, taken as 3x8.

Maximum length = 11 feet
From the C&C loads in section 4.3, psf(leeward) = 49.213

w = (16/12)49.213(1/2) = 5.468 lbf/in.

M = (1/8)wL^2 = (1/8)w(132^2) = 11910 lbf-in.
S = 1.14 in.^3, for a 2x8, strong axis
fb = 906 psi, o.k.
Max. deflection = 5wL^4/384EI = 0.283 in., o.k.
Reaction each end = (1/2)wL =(16/12)49.213(11/2) = 361 lbf

4.9 Floor System

The floor load = 40 psf live load and 10 psf dead load. Let the floor joists be 2x10 No.2 SP, 16 inches on center. These joists are nominally 12 feet in length, supported by the foundation on one side, and a twenty-four (24) foot floor girder on the other.
Let the floor girder be centrally supported by an isolated spread footings and pier, and let the center beam be three (3) 2x10 M-23 Machine Evaluated Lumber (Fb = 2400 psi)

Floor Joists

M = (1/8)(16/12)50(12^2) = 1200 lbf-ft= 14400 lbf-in.
fb = M/S = M/21.39 = 673 psi, o.k.

Floor Girder

M = (1/8)12(50)12^2 = 10800 lbf-ft = 129600 lbf-in.
Fb = M/S =M/3(21.39) = 2020 psi, o.k.
Note that maximum moment is negative at center support.

4.10 Foundation

The loads on the superstructure must be delivered to the earth in a manner which does not exceed allowable building pressure under maximum gravity loads and also is stable under maximum wind (MWFRS) loads.

First determine the superstructure loads in each of these two cases.

Gravity Loads :

Term	Area(ft^2)	DL(lbf)	LL(lbf)
Roof	576	5760	28800
Ceiling	576	5760	11520
Walls	768	7680	0
Floor	432	4320	17280
		-----	-----
Total		23520	57600

Floor center	144	1440	5760

Wind Loads: This is done by a four (4) step procedure similar to that used in determining C&C loads in section 4.3. as,

(1) Find strength wind speed corresponding to given ASD wind speed. As above, this equals 193.649 mph.
(2) Interpolate angle of roof, θ = tan^-1(3/12) = 14.036°, between 10° and 15°.
(3) Interpolate between 180 mph and 200 mph.
(4) Multiply by .6(1.21) to correct to ASD and Exposure C.

These results of these calculations are:

Area	psf
A	+53.323
B	-18.444
C	+35.516
D	-10.544
E	-51.778
F	-33.534
G	-36.222
H	-25.638

First consider Case A

Horizontal force = 6(8)A + 18(8)C + 6(3)B +18(8)D
= 6772.440 lbf
Vertical force = 6(12)E + 6(12)F +18(12)G + 18(12)H
= 19518.624 lbf (uplift)

Now examine Case B

Horizontal force = 3(8.375)A + (228-3(8.375))C
= 8545.048 lbf
Vertical force = 3(12)E +21(12)G + 3(12)F +21(12)H
= 18667.152 lbf (uplift)

It is seen that Case A governs for uplift and Case B for horizontal force.

Let foundation be wall footing, four (4) foot from top to base, One (1) foot thick with three (3) foot wide base.
Foundation DL = 6(150)4(24) = 86400 lbf

Bearing Pressure

Max. gravity load = superstructure LL + superstructure Dl + foundation DL
= 57600 + 23520 + 86400 = 167520 lbf

Load/linear foot = 167520/4(24) = 1745 lbf/linear foot
Base pressure = 1745/3 = 582 psf < 1500 psf, o.k.

Factor of Safety Against Uplift

F.S. = DL/Uplift = (23250 + 86400)/19519 = 5.63, o.k.

Factor of Safety Against Sliding

F.S. = 0.25(DL)/Maximum horizontal force
= 0.25(109920)/8545 = 3.22, o.k.

Factor of Safety Against Overturning

F.S. = Righting Moment/Overturning moment
F.S. = 109920(12)/8545(15) = 10.291, o.k.

4.11 Connectors

The connections considered here are mechanical connections, defined as attachment of structural members with hardware as opposed to connections made with adhesives.
The hardware may be nails, bolts, or steel plates and shapes.
Shear loads may be parallel to or perpendicular to the wood grain. Except for nails, allowable loads perpendicular to the grain are less than those parallel to the grain. For intermediate values of load angle to the wood grain, the Hankinson Formula is used.

$$F = \frac{Fc^{\perp} Fc^{\parallel}}{Fc^{\perp}(\cos\theta)^2 + Fc^{\parallel}(\sin\theta)^2}$$

The fasteners briefly discussed here are common nails, metal plate connectors, preformed connectors, and anchor bolts.

4.11.1 Nails

The two basic forms of nailed connections are those to resist shear and those to resist withdrawal. The geometry may be flat, toenail, or end grain. The first two can resist both shear and withdrawal, but an end grain connection can only resist shear.
Allowable loads are tabulated in Reference 14, "National Design Specification for Wood Construction", except for toenail connections, which are tabulated in Reference 15. These allowable loads are multiplied by the appropriate

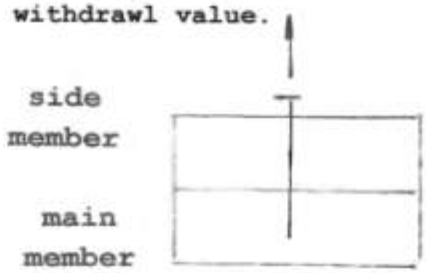
adjustment factors such as the effects of load duration, diaphragm nailing, moisture, and temperature.

Common nails have the following dimensions:

Pennyweight	Length (in.)	Diameter (in.)
6d	2.00	0.113
8d	2.50	0.131
10d	3.00	0.148
12d	3.25	0.148
16d	3.50	0.162

WITHDRAWL

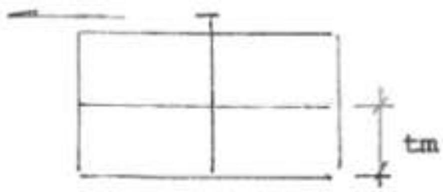
Loads shown as lbf/in. in main member. Embedment in end grain (other than side grain shown) has no withdrawl value.



Size	SP	SPF
6d	35	18
8d	41	21
10d	46	23
12d	46	23
16d	50	26

LATERAL

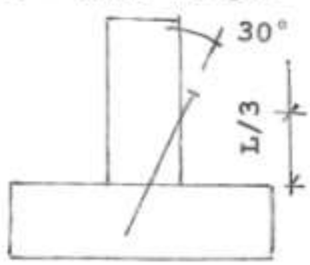
Here allowable loads are shown for 1-1/2″ thick side members, in lbf.



Size	SP	SPF	min tm
8d	81	63	1.00″
10d	128	100	1.50″
16d	154	120	2.00″

TOE NAIL

L = nail length



Lateral/withdrawl load :

Size	SP	SPF	min tm
6d	65/29	44/15	1.07″
8d	8042	62/21	1.33″
10d	107/57	80/29	1.60″
16d	128/72	99/37	1.86″

4.11.2 Metal Plate Connectors

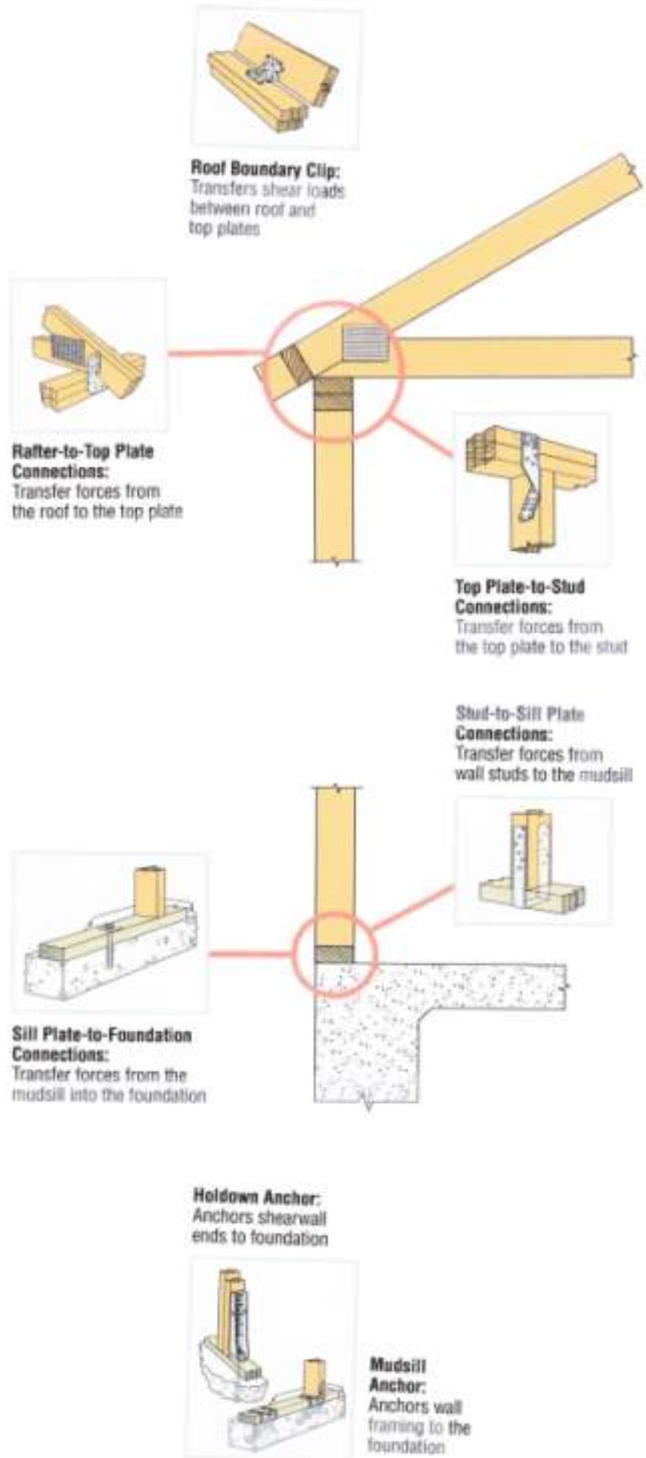
These are toothed rectangular metal plates, ranging in thickness from 20 gage (.0356 in.) to 16 gage (.0466in.), composed of Grade 60 steel and galvanized. They are used as joint connectors for pre-fabricated trusses in light wood-framed buildings. The connectors are factory-applied on each side of the joint by hydraulic presses.
Three ratings apply to connections, lateral resistance, tension , and shear. The lateral resistance refers to the tooth-holding resistance between the connector and wood. The units are psi/plate. The values depend on the wood grade and the orientation of the plate with respect to the wood grade. The tension and shear capacities are those of the metal plate.
Some specifications require that the truss analysis be performed with all connections pinned, yielding only axial forces at joints.

4.11.3 Preformed Metal Connections

These connections are factory-constructed. A diagram of some of these types, from Reference 16, "High Wind-Resistant Construction".

4.11.4 Anchor Bolts

These are either cast-in-place or epoxy anchors. The advantage of the latter type is that the holes in the bottom plate do not have to be aligned with embedded anchors.
A drawing of some connections (Ref. 16) follows.


Roof Boundary Clip:
Transfers shear loads between roof and top plates
Rafter-to-Top Plate Connections:
Transfer forces from the roof to the top plate
Top Plate-to-Stud Connections:
Transfer forces from the top plate to the stud
Stud-to-Sill Plate Connections:
Transfer forces from wall studs to the mudsill
Sill Plate-to-Foundation Connections:
Transfer forces from the mudsill into the foundation
Holdown Anchor:
Anchors shearwall ends to foundation
Mudsill Anchor:
Anchors wall framing to the foundation

5. COASTAL FLOOD ZONE RESIDENTIAL STRUCTURE

5.1 Introduction

The structure here is in what is called a "V-Zone". This is area extending from offshore to the inland limit of a primary frontal dune along an open coast. Wind forces are determined as Exposure D values. The foundation here must lift the building above the design flood elevation and withstand wind forces, flood forces, breaking wave forces, erosion, scour, and floating debris. As stated in reference 17, "Home Builder's Guide to Coastal Construction", the only practical way to accomplish these ends is with a deeply embedded and open pile foundation. The most common type piles for this application, are preservative-treated wood piles, which have comparatively low cost and ease of handling. They are, however, very difficult to splice.

5.2 Residence Loads

The residence in this example has the same dimensions and type as the residence in Part 4, so the dead and live loads are unchanged, as:
dead load = 24960 lbf
live load = 63360 lbf

The wind loads increase because of the increased elevation (less than 15 feet to 18 feet for centroid) and change in Exposure from C to D.
Wind load horizontal force = 8541(1.06/.85) = 10650 lbf
Wind load centroid above top of piles = 6 feet
Wind load uplift = 19499(1.06/.85) = 24316 lbf
The residence shown in Section 4 would have to be redesigned for this heavier wind loading.

5.3 Support Structure and Flood Loads

The structure supporting the residence if chosen to be nine piles, spaced twelve feet each way. The piles are interconnected at top by girders 18-32 shown on the finite element stick drawing in Section 5.5.

The piles are timber, Southern Pine, described in reference 20, "Timber Pile Design and Construction Manual", with fourteen feet above scour depth, and twenty feet embedment. The piles are tapered, with an 18 in. diameter at the top, and a 14 in. diameter at the tip. The equation for pile diameter y, in feet, in terms of x, distance from top, is:

y = -.017533x + 1.52258 = mx + b

The allowable stresses are : Fc = 1200 psi
Fb = 2400 psi
Fv = 110 psi and
E = 1500000 psi

The interconnecting horizontal beams are visually graded timbers, 8x12 No.1 Southern Pine, with the following allowable stresses : Fc = 825 psi
Fb = 1350 psi
Fv = 165 psi and
E = 1500000 psi

The configuration of the horizontal beams aids in distributing loads to piles, especially important for debris impact loads.

ds = design still water flood depth (including erosion)
ds = 6 ft
Hb = wave height = 0.78*ds = 4.68 ft
Let freeboard = 1 ft, and use formula for dfp from p.16
dfp = design flood depth = ds + .7*Hb + freeboard
dfp = 10.28 ft, say 12 ft, conservatively.
V = design flood velocity = (g*ds)^0.5 = (32.2*6)^0.5
V = 13.9 ft/sec

Fbuoy = vertical buoyant force on submerged object
Fbuoy = water density*volume
volume = (∏/4)*diameter^2*length

$$\text{volume} = (\pi/4)\int_0^{14} (mx+b)^2\,dx = 23.362\ \text{ft}^3$$

Multiplying by (64-45) pcf density gives net buoyant force = 444 lbf/pile

The breaking wave load, Fbrkp, is assumed to act at the the still water design depth (ds) and is given by
Fbrkp = .5*(CD)*density of water*D*Hb^2, where

CD = drag coefficient, 1.75 for round piles, 2.25 for square piles
D = average pile diameter, 1.457 ft
Fbrkp = .5*1.75*64*1.457*4.68^2 = 1787 lbf

The hydrodynamic force, Fdyn = .5*Cd*ρ*V^2*A, where
Cd = drag coefficient = 1.2 for round piles, 2.0 for rectangular piles
ρ = mass density of fluid = 1.99 slugs/ft^3 for salt water
A = affected area of pile = 1.457*12 = 17.484 ft^2
Fdyn = (1/2)*1.2*1.99*13.9^2*17.484 = 4033 lbf/pile

The debris impact load, Fi is due to the impact of portions of damaged buildings, utility poles, portions of previously embedded piles, and empty storage tanks. Assume the importance coefficient, depth coefficient, blockage coefficient and maximum response ratio are all equal to one. Let the orientation coefficient, Co = 0.8.
Fi = Co*∏*W*V/(2*g*Δt), where:
W = debris weight, 1000 lbf recommended by ASCE 7-10
Δt = duration of impact, 0.03 sec recommended by ASCE 7-10
Fi = 0.8*∏*1000*13.9/(2*32.2*0.03) = 18082 lbf !
The point of impact, is taken as the design flood depth,dsp.

Localized scour about each pile is assumed to be two (2) feet. This adds to the length of the pile above the ground, increasing pile height from 15 feet to 17 feet. It does not change the loads above.
The load combinations used for flood loads are:
(1) **Fbrkp (all piles) + Fi(one corner or critical pile)**
(2) **Fbrkp (front row of piles only) + Fdyn (on all piles except front row) + Fi (one corner or critical pile)**
It is seen from the above loads that case 2 governs.

5.4 Gravity Calculations

From 5.2 above, total residence load = DL + LL
Total residence load = 24960 + 63360 = 88380 lbf

Horizontal support beams = 211.9 ft x 26.95 plf = 5710 lbf
where support beams = 8x12

Total pile volume = $(\prod/4)*\int_0^{34} (mx+b)^2*dx = 48.926\ ft^3$

Thus total pile weight = 48.926 x 45 = 2202 lbf
Vertical load/pile = 88380/9 +5710/9 + 2202 = 12656 lbf

To find the ultimate compressive load capability, Qult, use the following formula from reference 19, "Foundations and Earth Structures" :

$$Qult = Pt*Nq*At + \sum_{H=0}^{D} Khc*Po*(\tan\delta)*S \text{ where}$$

At = area of pile tip, .7854 ft^2
D = pile depth, ft
H = location of calculation of pile depth
Khc = ratio of horizontal to vertical effective stress on side of element in compression - varies from 1.5 to 2.0 for single displacement tapered pile - assume 1.5
Nq = bearing capacity factor - varies from 10 to 145 for Friction angle Φ varying from 26° to 40°. Assuming Φ = 33°, then Nq = 35.
Po = effective vertical stress over certain length of pile embedment = soil density times depth. Here we assume soil density = 65 lbf/ft^3. Maximum depth for calculation = twenty (20) times tip diameter.
Pt = effective vertical stress at tip = D*density, Dmax for calculation = 20 ft
S = surface are of pile length, ft^2
δ = friction angle between pile and soil, for timber, δ = (3/4)*Φ, for Φ = 33°, δ = 24.75°

To find this capacity in continuous terms, use the following:

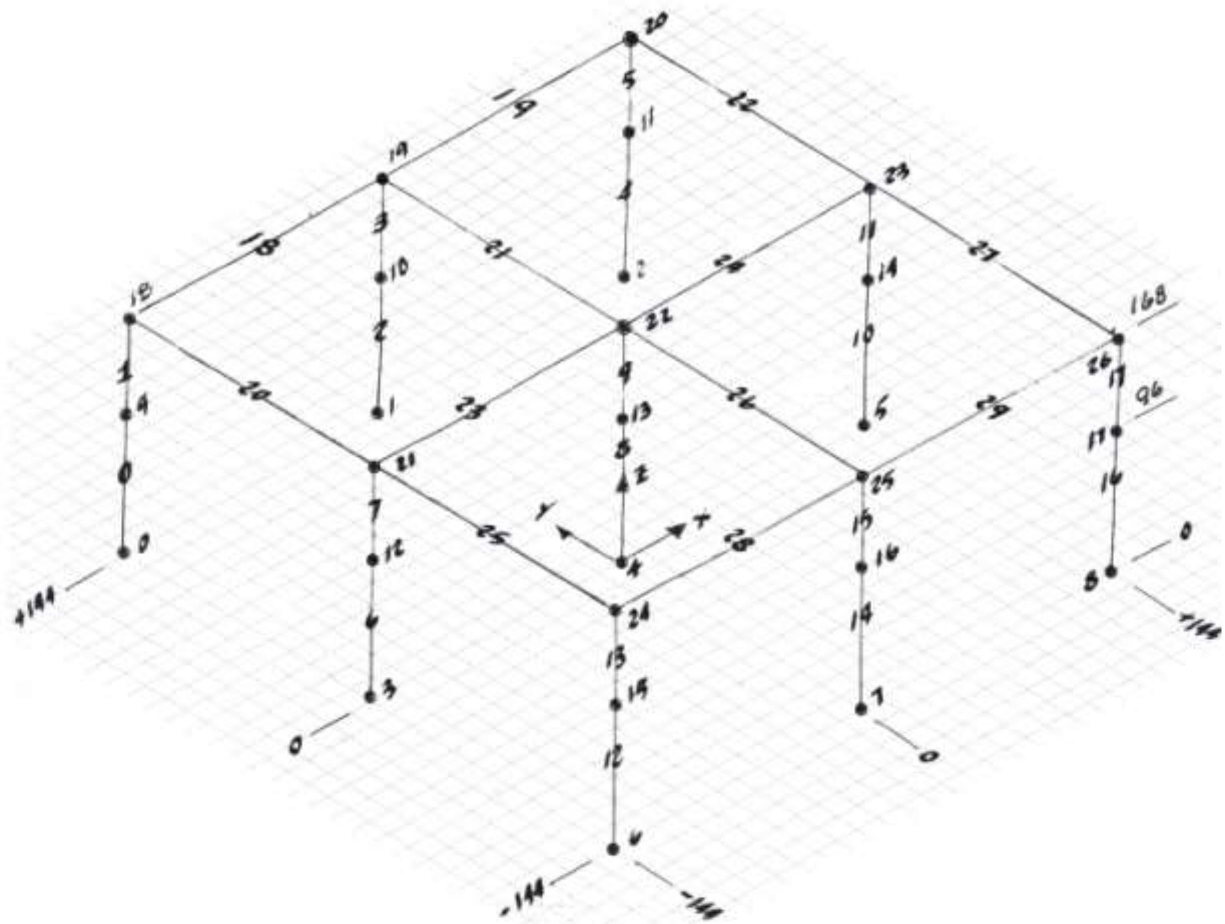
$$Qult = Pt*Nq*At + Khc*\tan\delta*\int_{14}^{34} Po*dS \quad \text{where}$$

Po = 65(x-14) and
dS = ∏*diameter*dx = ∏*(mx+b)*dx

The result of this calculation is:
Qult = 48640 + 70680 = 119320 lbf
F.S. = Factor of Safety = 119320/12656 = 9.43, o.k.

5.5 Wind and Flood Calculations

The finite element model shown here is used to find axial forces, moments, and shears at and above the scour depth level. Dimensions are in inches.



Nodes 0-8 represent base of piles at scour level.
Nodes 6-8, 15-17, and 24-26 represent seaside for flood conditions and the windward side.
Beam elements 18-29 model the supporting girders for the residence structure.
Beam elements 30-33, connecting nodes 18-22, 22-26, 20-32, and 32-24 respectively (not shown for clarity) represent diagonal girders for distributing lateral loads

```
WIND
----
Horizontal force      = 10650 lbf
Vertical force        = 24316 lbf (uplift)
Moment               = 10650*6  = 63900 lbf-ft
Moment resisted by    = 63900/24 = 2663 lbf,
Uplift on windward side, downwards on leeward side
Windward pile uplift = 24316/9+2663/3 = 3589 lbf
Center pile uplift    = 24316/9        = 2702 lbf
Leeward pile uplift   = 24316/9-2663/3 = 1814 lbf

The load combination used for wind alone is .6DL + W
DL at top of pile = residence + horizontal beams
                  = 24960/9 + 5710/9 = 3408 lbf
.6DL = 2045 lbf/pile

Thus the net uplift for .6DL downwards is:
Leeward pile (nodes 18-20)  = -231 lbf (downwards)
Center pile (nodes 21-23)   =  657 lbf
Windward pile (nodes 24-26) = 1544 lbf
```

The horizontal force, 10650 lbf, is assumed to be divided equally among the three windward posts as 10650/3 = 3550 lbf, applied to nodes 24-26.

For the wind alone, the wind on the piles must also be considered.
The exposed area for each pile is calculated to be 17.613 square feet. The total wind load on each pile is taken from the MWFRS area A as 53.3(1.06/.85) = 66.47 lbf/ft^2. Total load per pile is then 1179 lbf, with loads to nodes 9-17 of 585 lbf, and to nodes 18-26 of 293 lbf.

```
Results of the finite element analysis with these loads:
     Maximum moment = bases of 12, 16 = 197500 lbf-in
fbx = M/S = 197500/447.51 = 441 psi
Maximum tension   =  2677 lbf
ft  = T/A         = 2677/204.88 = 13 psi
Maximum shear     = base of 8 = 2702 lbf
fv  = V/A         = 2702/204.88 = 13 psi
Max. horiz. beam = 67431/165.3 = 408 psi
FLOOD
-----
```

As described in Section 5.3 above, the flood loads are applied as 1.5 Fa, resulting in:

```
Fdyn  = 4033*1.5  = 6050 lbf, applied to nodes 9-14
Fbrkp = 1787*1.5  = 2631 lbf, applied to nodes 15-17
```

Fi = 18082*1.5 = 27123 lbf, applied to node 24
All the .6DL + wind loads are also carried in addition to those above except for those at nodes 9-17.

The result of this analysis:
M at base of 12 = 1011100 lbf-in. → fbx = 2260 psi
Max. shear, at base of 8 = 39760 lbf → fv = 194 psi
Note impact load adjustment factor = 2.0.
Horizontal beam:
Mx = 329700 lbf-in. → fbx = 1994 psi
My = 10490 lbf-in. → fby = 97 psi

Maximum uplift = 5873 lbf at base of 12

Again from reference 19, the ultimate tension capacity, Tult , is given by

$$Tult = \sum_{H=0}^{D} Kht*Po*\tan\delta*S + \text{pile DL}$$

Kht = ratio of horizontal to vertical effective stress on side of element when element is in tension – varies from 1.0 to 1.3 for driven single displacement pile.

By an analysis similar to that of Section 5.4,
Uplift capacity = 47120 + 2202 = 49322
F.S. against uplift = 49322/5873 = 8.40, o.k.

If the point of application of Fi is changed from node 24 to node 15, the maximum flexural stress changes as:
fbx = 1433860/447.511 = 3204 psi.
Thus the location, as well as the magnitude of the impact force has a major bearing on pile moments.

5.6 Pile Connections

Reference 17, "Home Builder's Guide to Coastal Construction", details typical connections of the horizontal residence support beams to the piles. The connections must have the following properties:

- **Enough bearing area for maximum gravity loads.**
- **Sufficient uplift capacity to resist wind and flood loads, typically maximum at the eastward and windward piles.**
- **The support beams and diagonals must remain in position.**
- **Lateral loads must be successfully resisted. These loads**

may occur as shear, tension, or compression at the interface of the vertical faces of the support beams and the piles.

- **The connectors, as well as the piles and support beams Should be corrosion resistant.**
- **The detailing of the junctions of the horizontal beams, diagonals, and pile tops is especially important at the locations of debris impact, as maximum forces are transferred at these locations.**

This reference also suggests possible solutions to piles that are misaligned in construction.

Drawings of a typical bolted connection and a pile-to-beam bolted connection from Reference 17 are shown on the following page.

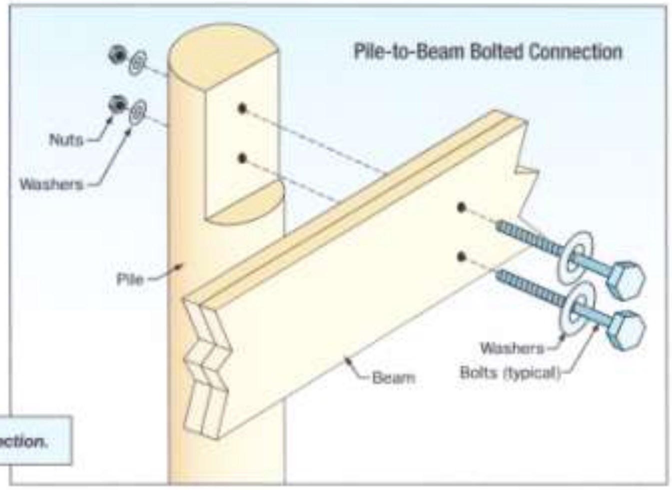
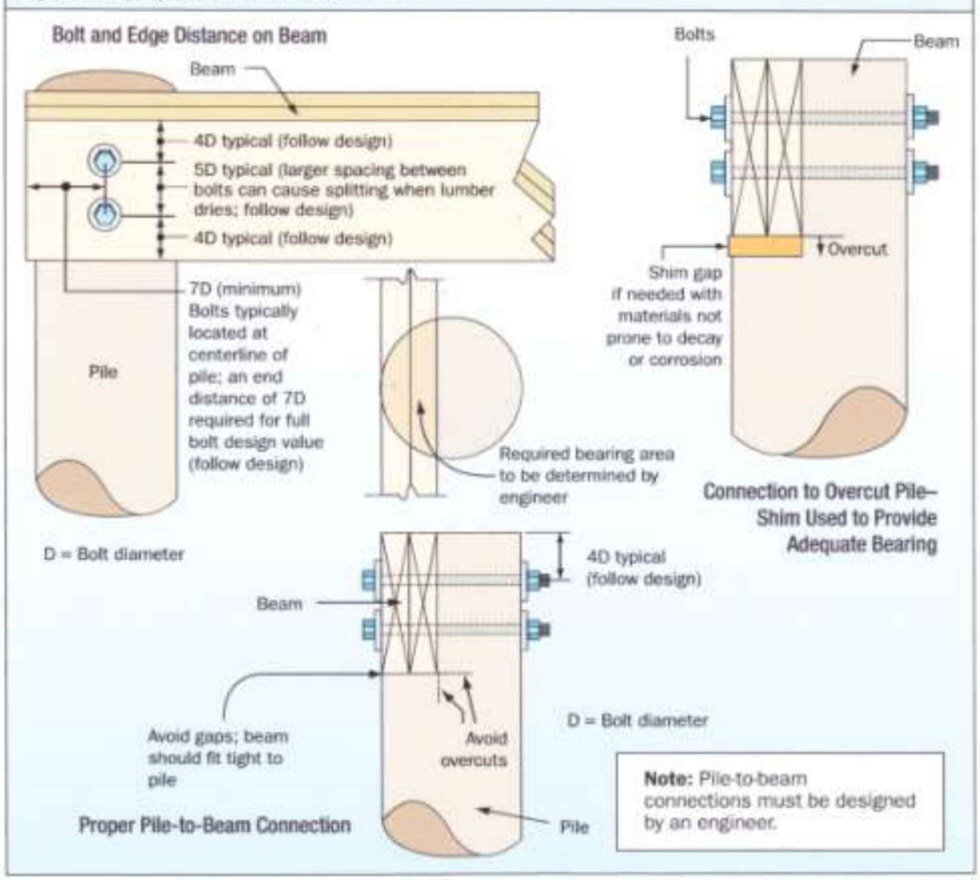


Figure 1. Pile-to-beam bolted connection.

Figure 2. Proper pile-to-beam bolted connection.



6. CONCLUDING REMARKS

- **The two most important factors in wind loading are the wind speed and Exposure Category. Other factors are shown in Section 3.**
- **The still water depth is the most important factor in flood loads. Erosion and scour should closely match site conditions.**
- **The debris impact load magnitude and all possible locations have a strong effect on pier size.**
- **As shown in Section 5, the soil type and friction angle determine ultimate uplift and bearing pile values.**
- **For the wind and flood loads chosen, a reinforced masonry residential structure and/or concrete piles may prove to be less expensive options, particularly if ds increases from the value of six feet used in the example.**
- **Items not covered are stairs, doors, windows, and installed facilities.**
- **The finite element representation for the flood resistant structure used fixed connections at the pile intersection with the ground surface. As shown in the NAVFAC Design Manual cited, there will be pile movement at these intersections due to shear and moment present there. A total design, utilizing the above ground structure and the soil-structure interaction for the below ground piles should be used.**

REFERENCES

1. **National Hurricane Center, www.nhc.noaa.gov**

2. **"Minimum Design Loads for Buildings and Other Structures",**

ASCE/SEI 7-10, ISBN 978-0-7844-1085-1, www.asce.org

3. "Standard Specification for Performance of Exterior Windows, Curtain Walls, Doors, and Impact Protective Systems Impacted by Windborne Debris in Hurricanes", ASTM E1996-09, www.stormsolutionsusa.com

4. "Taking Shelter from the Storm", FEMA P-320, 3rd Edition, August 2008, www.fema.gov

5. "ICC/NSSA Standard for the Design and Construction of Storm Shelters", ICC 500-2008, www.iccsafe.org

6. "Impact Dynamics of Rod Type Windborne Debris", Richards, etal., Seventh Asia-Pacific Conference on Wind Engineering, 2009, Taipei, Taiwan www.google.com

7. "Recommended Residential Construction for Coastal Area", FEMA P-550, 2nd Edition, December 2009, www.fema.gov

8. "Design of Wood Structures – ASD", Breyer, etal., 2003, ISBN 0-07-137932-0, McGraw-Hill, www.bn.com

9. "Diaphragms and Shear Wall", American Plywood Association, www.apawood.org

10. "Special Design Provisions for Wind and Seismic", 2008, ISBN 0-9786245-9-9, www.afandpa.org

11. "Retrofitting a Roof for High Wind Uplift", The Engineering Wood Association, 2000, Data File Number A410, www.apawood.org

12. "Shear Wall Design with Examples", DES 413-1, American Wood Council, 2014, www.awc.org

13. "Guidelines for Hurricane Resistant Residential Construction", Institute for Business and Home Safety, International Code Council, 2005, www.google.com

14. "National Design Specification for Wood Construction", ANSI/AWC NDS-2015, www.awc.com

15. "Toenail Connections", Design Aid No. 2, 2007, AF&PA, www.awcorg

16. "High Wind-Resistant Connections"

17. "Home Builder's Guide to Coastal Construction", Technical Fact Sheet Series, FEMA P-499, December 2010, www.FEMA.gov

18. "Recommended Residential Construction for Coastal Areas", FEMA P-550, Second Edition, December 2009, www.FEMA.gov

19. "Foundations and Earth Structures", NAVFAC DM-7.2, www.google.com

20. "Timber Pile Design and Construction Manual", American Wood Preservers Institute, 2002, www.wwpinstitute.org

APPENDIX 1 – BEAMS AND COLUMNS

DEFINITIONS

A = cross-section area, in.^2
b = member width, in.
c = term used in calculating C_P,
0.8 for sawn lumber
0.85 for round timber poles and piles
C_D = load duration factor
1.60 for wind/earthquake load
1.15 for snow load
C_L = beam stability factor, < = 1
C_P = column stability factor, < = 1
C_R = repetitive design factor for bending = 1.15
d = member depth, in.
E' = E, modulus of elasticity for dimension lumber, psi
f_b = calculated flexural stress, psi

f_c = calculated axial stress (compressive), psi

f_t = calculated axial stress (tensile), psi

f_v = calculated shear stress, psi

F_b = tabulated bending stress, psi

F_c = tabulated compressive stress, || to grain, psi

F_t = tabulated tensile stress, || to grain, psi

F_v = tabulated shear stress, psi

F_b^* = F_b times all applicable multipliers except C_L, psi

F_c^* = F_c times all applicable multipliers except C_P, psi

F_t' = F_t times all applicable multipliers, psi

F_v' = F_v times all applicable multipliers, psi

F_b^{**} = F_b^* times C_L, psi

F_c^{**} = F_c^* times C_P, psi

F_{bE} = term used in calculation of C_L, $K_{bE}E'/R_B^2$

F_{cE} = term used in calculation of C_P, $K_{cE}E'/(l_e/d)^2$

K_{bE} = 0.439 for dimension lumber

K_{cE} = 0.3 for visually graded members

l_e = effective braced length, for uniform loads
d' = b or d to give maximum l_e
if $l_u/d' < 7 \rightarrow l_e = 1.63\, l_u$, else $1.63\, l_u + 3d'$

l_u = actual braced length, in.

M = calculated moment, lbf-in.

P = calculated axial force (+ for compression), lbf

R_b = beam slenderness ratio = $(l_e d/b^2)^{(1/2)}$,
must be < = 50

V = calculated shear, lbf

AXIAL FORCE (TENSION, P < 0)

f_t = $-P/A$, must be < = F_t'

SHEAR STRESS

f_v = $1.5\, V/A$, must be < = F_v'

FLEXURAL STRESS

$$C_L = (1+(F_{bE}/F_b^*))/1.9 - (((1+(F_{bE}/F_b^*))/1.9)^2 - F_{bE}/0.95F_b^*)^{(1/2)}$$

f_b = M/S, must be < = F_b^{**}

AXIAL STRESS (COMPRESSION, P > = 0))

$$C_P = (1+(F_{cE}/F_c^*))/2c - (((1+F_{cE}/F_c^*))/2c)^2 - F_{cE}/cF_c^*)^{(1/2)}$$

f_c = P/A, must be < = F_c^{**}

COMBINED BENDING AND TENSION

$f_t/F_t + f_b/F_b^*$, must be < = 1.0

and if $f_b > f_t$ then $(f_b - f_t)/F_b^{**}$ must be < = 1.0

COMBINED BENDING AND COMPRESSION
(fc/Fc*)^2 + fb1/Fb1*(1-(fc/FcE1)), must be < = 1.0
where the subscript 1 refers to the wide face dimension

A program, called wood.c, implements the calculations above:

```
/* wood.c : dimension lumber calculations : 02-10-15 : ml */

#include<math.h>
#include<stdio.h>
#include<stdlib.h>

int main(void)
{
     int i;
     double wrkf;
     double A,b,CD,d,E,Fb,Fc,Ft,Fv,luf,lux,luy,M,P,S,V;
     /* inputs */
     double CL,CP,den,fb,fc,ft,fv,Fb_,Fb__,Fc_,Fc__,Ft_,Fv_; /*
     variables */
     double FbE,FcE,FcE1,le,lex,ley,ratio,Rb;
     double *data;
     FILE *inn;
     FILE *out;
     data =    calloc(16,sizeof(double));
     inn  =    fopen("wood.in","r");
     out  =    fopen("wood.out","w+");

     for(i=0;i<=15;i++)
     {
          fscanf(inn,"%lf",&wrkf);
          *(data+i) =    wrkf;
     }
     fclose(inn);
     A    =    *(data+0);
     b    =    *(data+1);
     CD   =    *(data+2);
     d    =    *(data+3);
     E    =    *(data+4);
     Fb   =    *(data+5);
     Fc   =    *(data+6);
     Ft   =    *(data+7);
     Fv   =    *(data+8);
     luf  =    *(data+9);
     lux  =    *(data+10);
     luy  =    *(data+11);
     M    =    *(data+12);
     P    =    *(data+13);
```

```
S     =     *(data+14);
V     =     *(data+15);

/* shear */

Fv_   =     CD*Fv;
fv    =     1.5*V/A;
ratio       =     fv/Fv_;
fprintf(out,"fv/Fv*= ");fprintf(out,"%6.3f\n",ratio);

/* bending */

Fb_   =     1.15*CD*Fb;
if(luf==0)
{
      CL    =     1.0;
}
else
{
      if(luf/d<7)
      {
            le    =     2.06*luf;
      }
      else
      {
            le    =     1.63*luf+3*d;
      }
      Rb    =     sqrt(le*d/(b*b));
      if(Rb>50)
      {
            fprintf(out,"Rb = ");fprintf(out,"%6.0\n",Rb);
            exit(0);
      }
      else
      {
            ;
      }
      FbE   =     .439*E/(Rb*Rb);
      ratio       =     FbE/Fb_;
      CL    =     (1.0+ratio)/1.9
                  -sqrt(((1.0+ratio)/1.9)*((1.0+ratio)/1.9)-
                  ratio/.95);
}
Fb__  =     CL*Fb_;
fb    =     M/S;
ratio       =     fb/Fb__;
fprintf(out,"fb/Fb**= ");fprintf(out,"%6.3f\n",ratio);
```

```
/* axial */

if(P<0)
{
	Fc_	=	0;
	Fc__	=	0;
	Ft_	=	CD*Ft;
	ft	=	-P/A;
	ratio	=	(fb-ft)/Fb__;
	fprintf(out,"(fb-ft)/Fb** =”);
	fprintf(out,"%6.3f\n",ratio);
	ratio	=	ft/Ft_+fb/Fb_;
	fprintf(out,"f + t       =”);
	fprintf(out,"%6.3f\n",ratio);
}



else
{
	Ft_	=	0;
	Fc_	=	CD*Fc;
	if(lux/d<7)
	{
		lex	=	2.06*lux;
	}
	else
	{
		lex	=	1.63*lux+3.0*d;
	}
	if(luy/b<7)
	{
		ley	=	2.06*luy;
	}
	else
	{
		ley	=	1.63*luy+3.0*b;
	}
	if(lex>ley)
	{
		le	=	lex;
		den	=	lex/d;
	}
	else
	{
		le	=	ley;
```

```
                den  =    ley/b;
            }
            FcE  =    .3*E/(den*den);
            ratio     =    FcE/Fc_;
            CP   =    (1.0+ratio)/1.6
                      -sqrt(((1.0+ratio)/1.6)*((1.0+ratio)/1.6)-
                      ratio/.8);
            Fc__ =    CP*Fc_;
            fc   =    P/A;
            ratio     =    fc/Fc__;
            fprintf(out,"fc/Fc**      =");
            fprintf(out,"%6.3f\n",ratio);
            FcE1 =    .3*E/((lex/d)*(lex/d));
            Ratio=     (fc/Fc_)*(fc/Fc_)+fb/(Fb_*(1.0-fc/FcE1));
            fprintf(out,"f + c        =");
            fprintf(out,"%6.3f\n",ratio);
        }

        fprintf(out,"Fb*          ="); fprintf(out,"%6.1f\n",Fb_);
        fprintf(out,"Fc*          =");
        fprintf(out,"%6.1f\n",Fc_);
        fprintf(out,"Ft'          ="); fprintf(out,"%6.1f\n",Ft_);
        fprintf(out,"Fv'          ="); fprintf(out,"%6.1f\n",Fv_);
        fprintf(out,"le           =");
        fprintf(out,"%6.1f\n",le);
        fprintf(out,"Fb**         ="); fprintf(out,"%6.1f\n",Fb__);
        fprintf(out,"Fc**         ="); fprintf(out,"%6.1f\n",Fc__);
        fclose(out);
        return 0;
}
```