STRUCTURAL SYSTEMS

A preparatory course assembled for the
Architectural Record Examinations
for AIA Baltimore

by Jason E. Charalambides PhD, PE, M.ASCE, AIA, ENV_SP

Data and graphics composed by author
and accumulated from various sources of the web
(This is only for educational purposes)

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ESSENTIAL PARAMETERS IN STRUCTURAL DESIGN

- A Structure fails when it does not do what it is intended to do. *The above happens to be good definition of unsuccessful design in general*
  - Fracture
  - Yielding
  - Buckling
  - Connection failure
  - Excessive displacement
  - Vibration
- Causes of failure:
  - Wrong estimation of loads
  - Mistakes in Analysis of elements
  - Connection failures
  - Imprecise processes during construction phase

A well designed structure should reflect the form and vice versa

Some poorly designed structural forms collapse by themselves. No earthquake is necessary.

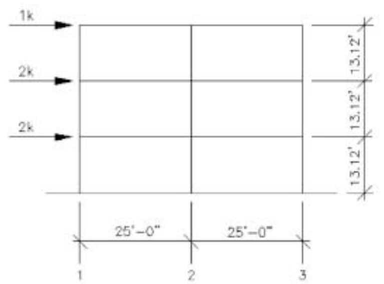
What we will discuss

- Loads (Axial & Flexural) - Forces / Vectors & Moments
  - *Tributary Areas (& refer Portal Method)*
  - *Overview of Code ASCE07 and IBC*
  - *Loading and definition of Stress and Strain*
- Material and Geometric qualities
  - *Types of Materials, Elastic vs Brittle behavior*
  - *Modulus of Elasticity, Resilience, Toughness*
- Axial Loading and Buckling
  - *Tensile Members*
  - *Slender Members & Euler's Formula*
  - *Truss systems*
    - *Joint & Section Method*
- Loading of Flexural Members
- Shear and integration to Moment
  - *Standardized formulae for Moment Calculation*
- Transfer of loads through structure (includes shear, bracing, trusses, connections, etc)
- Effects of geometric forms vs material
  - *Moment of Inertia and the concept of Stiffness*
  - *Center of Gravity vs Center of Shear (Center of rotation)*
- Types of connections
- Continuity
- Systems
- Lateral Loads
  - Wind & Earthquake
- Deflections

Loading

What is the Portal Method?

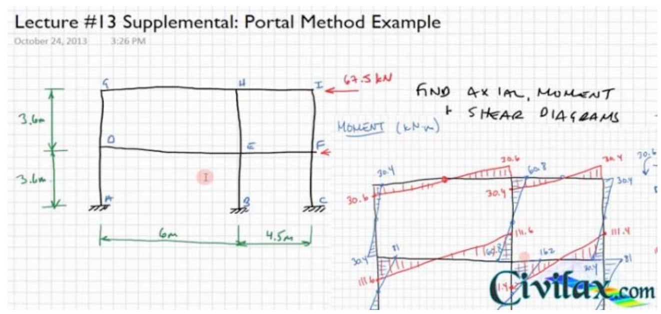
SAMPLE MULTIPLE-CHOICE QUESTIONS



21. For the rigid frame structure shown, the approximate horizontal shear at the base of column 2 (assuming all column stiffnesses are equal) is

[ ] k

23. In the C… the min… transfer… …elds)
- [ ] 3t
- [ ] 6t
- [ ] 9t
- [ ] 12t



More Fun? OK Go Figure!

13. Cast-in-place concrete beams and columns with No. 11 rebar or smaller reinforcing bars that are not exposed to weather or in contact with the ground should have a minimum coverage of concrete over the bars of
- [ ] 1/2 in
- [ ] 3/4 in
- [ ] 1 in
- [ ] 1 1/2 in

4. continuous wall footings
5. 57.3 in³
6. 67 kips
7. 16 sf
8. Bending forces in the vertical members
9. 28
10. compensate for deflection
11. 2.0 in
12. Limit the number of visitors in spaces to the available live load.
13. 1 1/2 in
14. 33 percent
15. Gypsum shaft wall
16. Parking
17. A building with a symmetrical square plan
18. A, B, D, E
19. A, B, C
20. frame in which diagonal members are connected to a beam a short distance from the column joint
21. 2.50 k
22. Four-story
23. 6t
24. A, C, D, E
25. liquefiable soils
26. L/600
27. Architect

31. The structure has redundancy.
32. Wind forces
33. A, B, F
34. C
35. B, C

There is no special condition about rebar sizes for beams and columns in the ACI 318-14 code. So are we talking about slabs, joists, and non structural walls, or is this some tricky type of question by NCARB, ...or could it be that someone interprets tables differently?

Table 20.6.1.3.1—Specified concrete cover for cast-in-place nonprestressed concrete members

Concrete exposure	Member	Reinforcement	Specified cover, in.
Cast against and permanently in contact with ground	All	All	3
Exposed to weather or in contact with ground	All	No. 6 through No. 18 bars	2
		No. 5 bar, W31 or D31 wire, and smaller	1-1/2
Not exposed to weather or in contact with ground	Slabs, joists, and walls	No. 14 and No. 18 bars	1-1/2
		No. 11 bar and smaller	3/4
	Beams, columns, pedestals, and tension ties	Primary reinforcement, stirrups, ties, spirals, and hoops	1-1/2

20.6.1.3.2 Cast-in-place prestressed concrete members shall have specified concrete cover for reinforcement, ducts, and end fittings at least that given in Table 20.6.1.3.2.

RISK CATEGORY 1 (FOR ACCESSORY STRUCTURES)

RISK CATEGORY 2 (FOR SINGLE FAMILY RESIDENTS)

RISK CATEGORY 3 & 4 (HEAVY COMMERCIAL)

Risk Category of Buildings and other Structures

Building Risk Categories are listed in Table 1604.5 of 2010 FBC Building. (page 16.5 in code):

- Safety Factors:
  - We apply safety factors in measuring loads and in designing
  - Once the loads are estimated we call them "Service loads"
  - Usually we apply 1.2 to Dead Load and 1.6 to Live Load. The loads after application of safety factors are called "Design Loads"
  - But we also apply a safety factors on the design of elements too. That can vary between 0.9 all the way down to 0.65 depending on the importance of the specific behavior.

Some of the Specifics of Codes

- Load reduction:
  - In certain cases, especially when very large areas are covered, the calculated live load can be reduced through specific formulae that are available in the IBC and the ASCE-7.

TABLE 1607.9.1
LIVE LOAD ELEMENT FACTOR, K_{LL}

ELEMENT	K_{LL}
Interior columns	4
Exterior columns without cantilever slabs	4
Edge columns with cantilever slabs	3
Corner columns with cantilever slabs	2
Edge beams without cantilever slabs	2
Interior beams	2
All other members not identified above including: Edge beams with cantilever slabs Cantilever beams One-way slabs Two-way slabs Members without provisions for continuous shear transfer normal to their span	1

1607.9.1 General. Subject to the limitations of Sections 1607.9.1.1 through 1607.9.1.4, members for which a value of $K_{LL}A_T$ is 400 square feet (37.16 m²) or more are permitted to be designed for a reduced live load in accordance with the following equation:

$$\boxed{L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right)} \qquad \textbf{(Equation 16-22)}$$

60

200

For SI: $L = L_o\left(0.25 + \frac{4.57}{\sqrt{K_{LL}A_T}}\right)$

where:

L = Reduced design live load per square foot (square meter) of area supported by the member.

L_o = Unreduced design live load per square foot (square meter) of area supported by the member (see Table 1607.1).

K_{LL} = Live load element factor (see Table 1607.9.1).

A_T = Tributary area, in square feet (square meters).

L shall not be less than $0.50L_o$ for members supporting one floor and L shall not be less than $0.40L_o$ for members supporting two or more floors.

Some of the Specifics of Codes

- Load reduction continued:
  - This extends further to roof live loads.

1607.11.1 Distribution of roof loads. Where uniform roof live loads are reduced to less than 20 psf (0.96 kN/m²) in accordance with Section 1607.11.2.1 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 most unfavorable *load effect*. See Section 1607.11.2 for reductions in minimum roof live loads and Section 7.5 of ASCE 7 for partial snow loading.

1607.11.2 Reduction in roof live loads. The minimum uniformly distributed live loads of roofs and marquees, L_o, in Table 1607.1 are permitted to be reduced in accordance with Section 1607.11.2.1 or 1607.11.2.2.

1607.11.2.1 Flat, pitched and curved roofs. Ordinary flat, pitched and curved roofs, and awnings and canopies other than of fabric construction supported by lightweight rigid skeleton structures, are permitted to be designed for a reduced roof live load as specified in the following equations or other controlling combinations of loads in Section 1605, whichever produces the greater load.

In structures such as greenhouses, where special scaffolding is used as a work surface for workers and materials during maintenance and repair operations, a lower roof load than specified in the following equations shall not be used unless *approved* by the *building official*. Such structures shall be designed for a minimum roof live load of 12 psf (0.58 kN/m²).

$L_r = L_o R_1 R_2$ **(Equation 16-25)**

where: $12 \le L_r \le 20$

For SI: $L_r = L_o R_1 R_2$

where: $0.58 \le L_r \le 0.96$

L_r = Reduced live load per square foot (m²) of horizontal projection in pounds per square foot (kN/m²).

The reduction factors R_1 and R_2 shall be determined as follows:

$R_1 = 1$ for $A_t \le 200$ square feet (18.58 m²) **(Equation 16-26)**

$R_1 = 1.2 - 0.001A_t$ for 200 square feet $< A_t < 600$ square feet **(Equation 16-27)**

For SI: $1.2 - 0.011A_t$ for 18.58 square meters $< A_t < 55.74$ square meters

$R_1 = 0.6$ for $A_t \ge 600$ square feet (55.74 m²) **(Equation 16-28)**

where:

A_t = Tributary area (span length multiplied by effective width) in square feet (m²) supported by any structural member, and

$R_2 = 1$ for $F \le 4$ **(Equation 16-29)**

$R_2 = 1.2 - 0.05F$ for $4 < F < 12$ **(Equation 16-30)**

$R_2 = 0.6$ for $F \ge 12$ **(Equation 16-31)**

where:

F = For a sloped roof, the number of inches of rise per foot (for SI: $F = 0.12 \times$ slope, with slope expressed as a percentage), or for an arch or dome, the rise-to-span ratio multiplied by 32.

AND MORE CODES

- LRFD system
- The abbreviation stands for Load and Resistance Factor Design System developed much earlier and implemented in late 80s replacing the Allowable Stress Design (ASD) that came back again in 2005 integrated in the same AISC Steel Construction Manual.

Nominal Loads

D	=	deal load
L	=	live load
L_R	=	roof live load
S	=	snow load
R	=	rain load
W	=	wind load
E	=	earthquake load

LRFD Load Combinations

1. $1.4D$
2. $1.2D + 1.6L + 0.5(L_R \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_R \text{ or } S \text{ or } R) + (0.5L \text{ or } 0.5W)$
4. $1.2D + 1.0W + 0.5L + 0.5(L_R \text{ or } S \text{ or } R)$
5. $1.2D + 1.0E + 0.5L + 0.2S$
6. $0.9D + 1.0W$
7. $0.9D + 1.0E$

For simplification, in this class, combination #2 is a default pg. 2-10 of AISC manual.

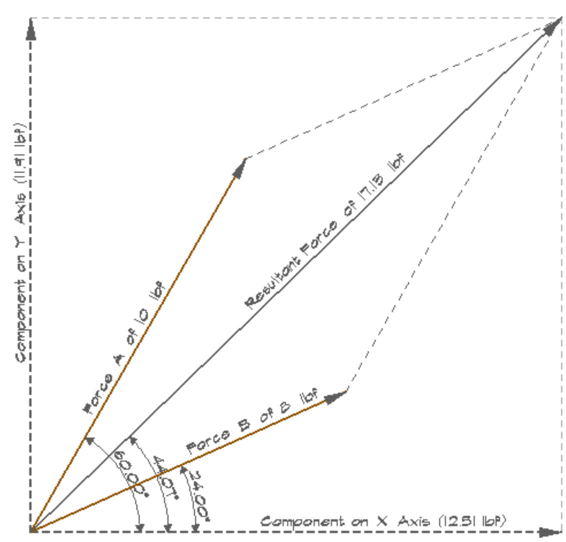
Exception: The load factor on *L* in load combinations 3, 4, and 5 shall equal 1.0 for garages, areas occupied as places of public assembly, and all areas where the live load is greater than 100 psf

LET'S ADDRESS THE FUNDAMENTALS OF LOADS IN TERMS OF VECTORS

- Example:
  - Determine the resultant force of the represented forces in the two dimensional diagram, and analyze it in components on the x and y directions:

$$x\ component = 10^{lbf} * \cos 60^{o} + 8^{lbf} * \cos 24^{o} = 12.31^{lbf}$$

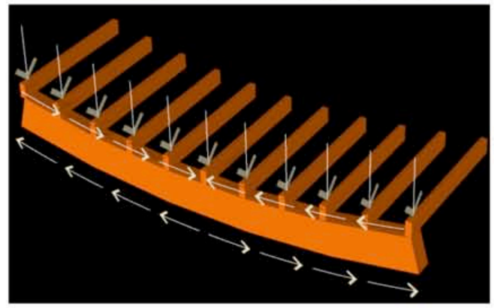
$$y\ component = 10^{lbf} * \sin 60^{o} + 8^{lbf} * \sin 24^{o} = 11.91^{lbf}$$



How are Loads Applied?

- Point Loads:
  - A column will exercise a load on a specific point.
    - *It actually receives load from above and transfers it all to a point.*

- Distributed Loads
  - A floor carries dead and love loads throughout its surface. Joists pick it up and distribute it on a beam, and the beam transfers it to the columns.



Dead Loads vs Live Loads

- Live loads are usually given by codes such as the IBC or the ASCE-7:

- Dead Loads are calculated

  - Anything that can move easily (not just living organisms but furniture, furnishings etc are considered live load)

Uniform and Concentrated Loads

USE OR OCCUPANCY		Uniform load (psf) *.048 for KN/m^2	Concentrated load (lbf) *.00446 for KN/m^3
Access floor systems	Office use	50	2000
	Computer use	100	2000
Armories		150	0
Assembly areas and auditoria with balconies therewith	Fixed seating areas	50	0
	Movable seating & other	100	300
	Stage areas and enclosed platforms	125	0
Cornices and marquees		60	0
Exit facilities		100	0
Garages	Gen. Storage & repair	100	See ASCE_7
	Private vehicle storage	50	See ASCE_7
Hospitals	Wards & rooms	40	1000
Libraries	Reading	60	1000
	Stacks	125	1500
Manufacturing	Light	75	2000
	Heavy	125	2500
Offices		50	2000
Printing plants	Press rooms	150	2500
Residential	Basic floor area	40	0
	Exterior balconies	60	0
	Decks	40	0
	Storage	40	0
Restrooms	Equal to occupancy but not to exceed 50lbf/ft^2		
Reviewing stands, bleachers & folding seats		100	0
Roof decks	Same as area served or for the type of occupancy accommodated		
Schools	Classrooms	40	1000
Sidewalks & driveways	Public access	250	See ASCE_7
Storage	Light	125	
	Heavy	250	
Stores		100	3000
Pedestrian bridges and walkways		100	

The above are only as basic reference. For more detailed info please refer to ASCE_7 or the applicable building code

Weights of Common Building Materials

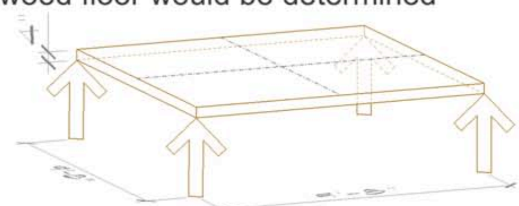
Material	Load
Brick (4" - on wall)	40 psf
Curtain wall (aluminum & glass)	15 psf (avg)
Earth (Soil)	100 – 130 pcf
Glass (1/4")	3.3 psf
Granite	170 pcf
Gypsum board (1/2")	1.8 psf
Hardwood floor (7/8")	2.5 psf
Heavy aggregate concrete block	83 pcf
Marble	165 pcf
Plaster (1/2")	4.5 psf
Plywood (1/2")	1.5 psf
Quarry tile (1/2")	5.8 psf
Reinforced concrete	150 pcf
Roofing (5-ply)	6 psf
Shingles (asphalt)	2 psf
Steel decking	2.5 psf
Suspended acoustical ceiling	1 psf
Terazzo 2 1/2" sand cushion	27 psf
Water	62.4 pcf
Wood @ 20% moisture	30 – 40 pcf

EXAMPLE

- Calculating:
  - A square 9^{ft}-8^{in} x 9^{ft}-8^{in} concrete continuous slab has a thickness of 4in and is supported by four columns at the edges. It is covered by standard hardwood floor and is part of a residential character building. Determine the load that is distributed by this slab to each of the four columns:
  - Considering the tributary area for each of the columns it is fair to suggest that the depth of the slab multiplied by half the length and by half the width and then by the weight of concrete shall provide the dead load contribution of the concrete distributed to each column. →

$$DL_{conc} = 0.333^{ft} \cdot 4.833^{ft} \cdot 4.833^{ft} \cdot 150^{pcf} = 7.787^{ft^3} \cdot 150^{pcf} = 1168^{lbf}$$

  - Also, by reference to the tables, the dead load from the hardwood floor would be determined by the area multiplied by the weight per square foot.



$$DL_{flr} = 4.833^{ft} \cdot 4.833^{ft} \cdot 2.5^{pcf} = 58.4^{lbf}$$

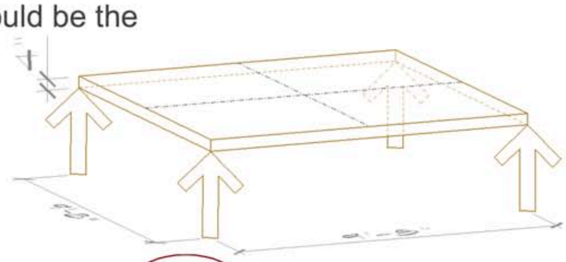
  - Producing a total Dead Load of 1226.47^{lbf}

EXAMPLE

- Continuing:
  - Also, the live load that can be calculated for that would be calculated in a fashion similar to that applied for the flooring. The value for the basic floor load of a residential building will be multiplied by the tributary area:

$$L\,L = 4.833^{ft} \cdot 4.833^{ft} \cdot 40^{psf} = 934.4^{lbf}$$

  - There are two formulae from the list given above that would be the most applicable.



    - The first yields:

$$1.4 \cdot Dead\ Load = 1717^{lbf}$$

    - The second one yields:

$$1.2 \cdot Dead\ Load + 1.6 \cdot Live\ Load + 0.5 \cdot Roof\ Load = 1472^{lbf} + 1495^{lbf} + 0^{lbf} = 2967^{lbf}$$

  - Which is evidently the one that governs.

Load Reductions

- When we have larger areas, the live load can be reduced!
  - **4.7.2 Reduction in Uniform Live Loads**
    Subject to the limitations of Sections 4.7.3 through 4.7.6, members for which a value of $K_{LL}A_T$ is 400 ft^2 (37.16 m^2) or more are permitted to be designed for a reduced live load in accordance with the following formula:

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) \qquad (4.7\text{-}1)$$

where

L = reduced design live load per ft^2 (m^2) of area supported by the member
L_o = unreduced design live load per ft^2 (m^2) of area supported by the member (see Table 4-1)
K_{LL} = live load element factor (see Table 4-2)
A_T = tributary area in ft^2 (m^2)

L shall not be less than $0.50L_o$ for members supporting one floor and L shall not be less than $0.40L_o$ for members supporting two or more floors.

EXCEPTION: For structural members in one- and two-family dwellings supporting more than one floor load, the following floor live load reduction shall be permitted as an alternative to Eq. 4.7-1:

$$L = 0.7 \times (L_{o1} + L_{o2} + \ldots)$$

L_{o1}, L_{o2}, … are the unreduced floor live loads applicable to each of multiple supported story levels regardless of tributary area. The reduced floor live load effect, L, shall not be less than that produced by the effect of the largest unreduced floor live load on a given story level acting alone.

Material Qualities

What is Stress?

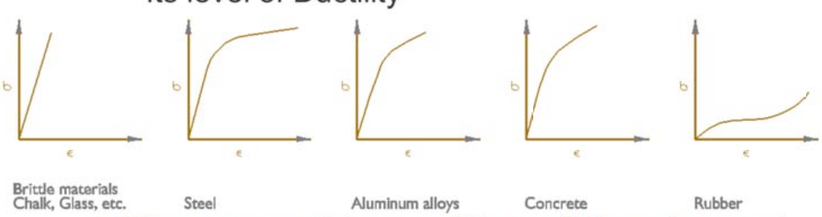
- Besides taking these ARE?
  - At this point we want to focus on the axial type of stress which is given by the division of Load over Area.
    - The term psi stands for pounds per square inch!
  - There is also the flexural stress which is given by the Moment, multiplied by the distance from the N/A to the extreme fiber, and then divided by the Moment of Inertia.

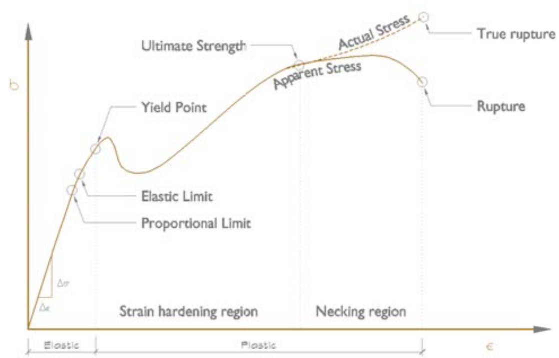
$$\sigma_A = \frac{P}{A}$$

$$\sigma_B = \frac{M \cdot c}{I}$$

Essential Parameters for the Strength of a Structural Element

- The embedded qualities of the material
  - The Young's Modulus of Elasticity
  - Its Strength
  - Its level of Ductility





Note: Between elastic limit and yield point, material deforms plastically on extremely small scale.

- The geometric qualities of the element
  - The Length
  - The Cross Sectional Area
  - The Moment of Inertia
    - *One can argue about the radius of gyration, the Section modulus, etc, but those are included in the above in different ways. e.g.* $r = \sqrt{\frac{I}{A}}$

Example – Steel

- Main advantages:
  - Strength
  - Homogeny
  - Elasticity
  - Ductility
  - Speed of erection
  - Defined set of forms (dimensions)
  - Adaptability
  - Longevity
  - Simplicity
  - Quality control
  - Recyclability / Scrap value *(Use bolts instead of welds)*

- Main disadvantages:
  - Corrosion
  - Fireproofing
  - Susceptibility to buckling
    - *Very strong but thin*
  - Fatigue
  - Brittle fracture
    - *A rapid propagation of cracks that allows no chance for plastic deformation to happen before fracture.*

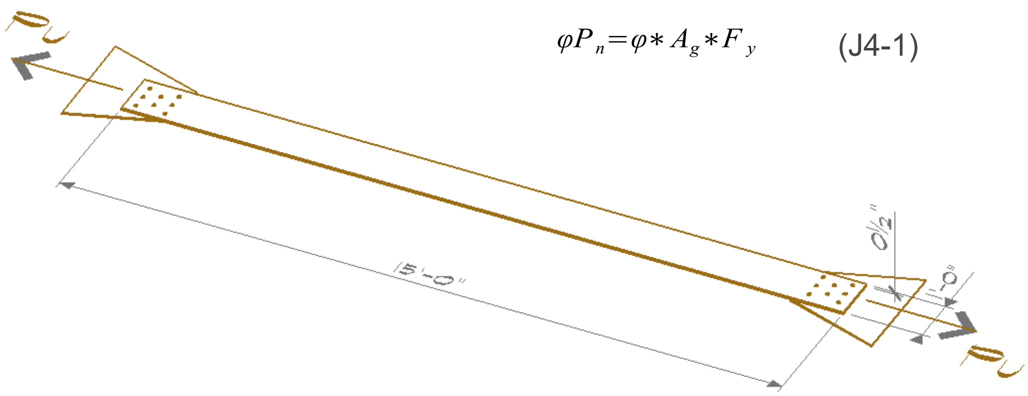
Material Effects through Axial Loading

- Let's start with Axial Tension:
  - An element in tension will experience a stress that will be equal to the
    - Load over the area
  - Effect on the right is after stress went through the necking region and reached fracture in axial tension

Let's Address the Behavior of Steel

- 1: Yield of Gross Area:
  - This occurs when the cross sectional area of the steel member yields and deforms within its plastic region
  - Let's consider a 15' long plate of A-572 steel, welded to gussets and subjected to a tensile force. It carries a thickness of ½" and width of 12"

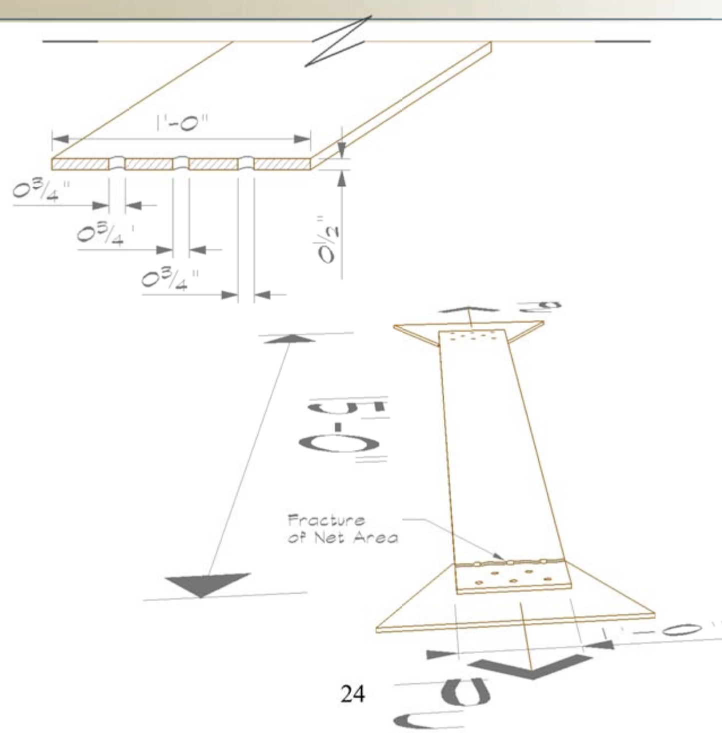
$$\varphi P_n = \varphi * A_g * F_y \qquad \text{(J4-1)}$$



Let's Address the Behavior of Steel

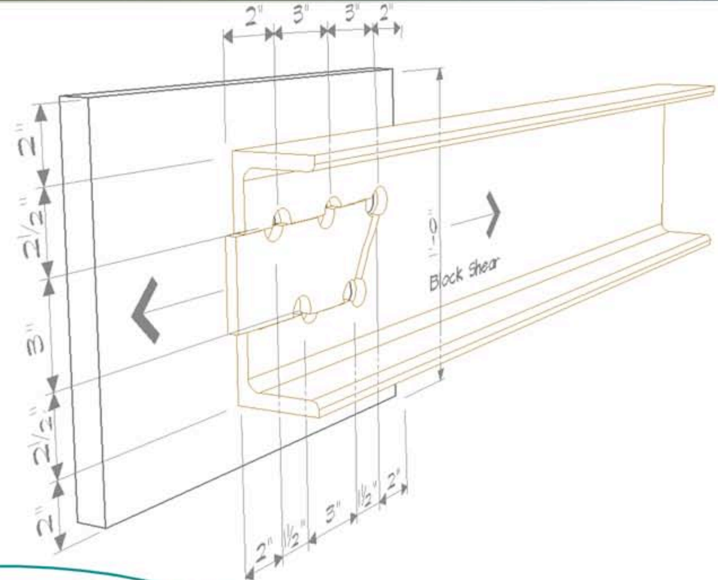
- 2: Fracture of Net Area:
  - This occurs when the cross sectional area of the steel member fractures between the points where bolts are located
  - The element shall not fracture where there are no bolt holes. Nature always finds the easiest and most comfortable locations for solid materials to fail. They shall not fail at the second weakest point.

$$\varphi P_n = \varphi * A_n * F_u \qquad \text{(J4-2)}$$



Let's Address the Behavior of Steel

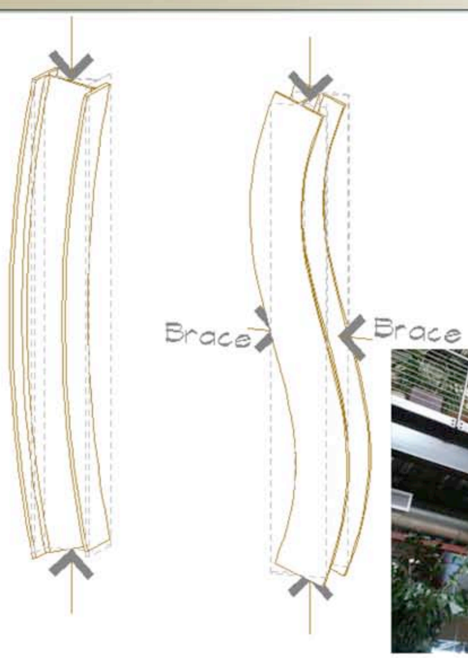
- 3: Block Shear Fracture:
  - The element in tension shears along the lines of the bolts.
  - It may follow a number of paths to shear.
  - The 0.6 factor is applied on the Tension portion of the equation because the Fu and Fy values are determined by tension tests. The comparable terms in shear are approximately 0.6 times the tension values.

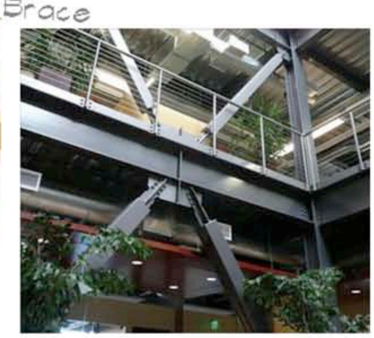


$$\varphi R_n = \varphi * (0.6 * F_u * A_{nv} + U_{bs} * F_u * A_{nt}) \leq \varphi * (0.6 * F_y * A_{gv} + U_{bs} * F_u * A_{nt}) \qquad \text{(J4-5)}$$

Steel is very strong in Compression

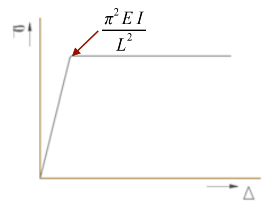
- Due to its high strength, Steel requires less area.
- However, a thin geometry together with compressive loads may cause the effect that we call buckling
  - Due to time limitations let's not engage into the details of the mechanics, but we can at least mention that there are methods to alleviate this issue
- Bracing reduces the "slenderness" and is one method to solve this problem



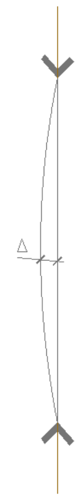


EULER'S BUCKLING PRINCIPLE

- Euler's elastic buckling:
  - The buckled shape resembles ½ a sinusoidal distribution.
  - The buckling load Pe is proportional to the Moment of Inertia of the element
  - Buckling is proportional to the Young's modulus of elasticity (E) but independent of the yield strength of the material (Fy)
  - The buckling load is inversely proportional to the square value of the length of the element (L^2)
  - The longer the element the more susceptible to buckling



$$P_E = \frac{\pi^2 E I}{L^2}$$



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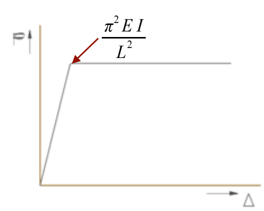
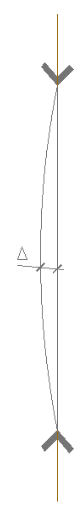
  - The longer the element the more susceptible to buckling

$$P_E = \frac{\pi^2 E I}{L^2}$$





Euler's Buckling principle in Stress

- Load divided by Area will give us...
  - A lot of stress!
  - Not really a lot but it will give us the stress.
- But there's another factor that is very important to note here. It is the Slenderness ratio of the column (λ)
  - The greatest reductions in strength are noticed in columns that have a “Slenderness ratio” between the values of 70 – 90.
- But there is also the “K” factor that we did not talk about.

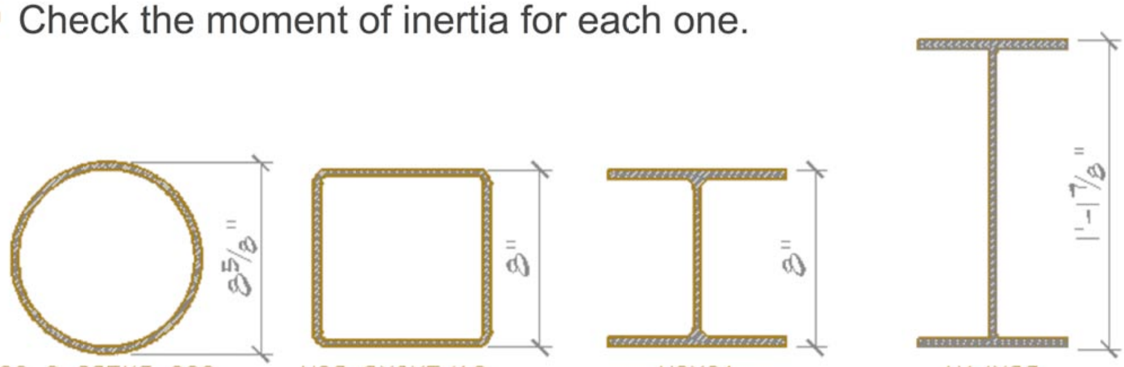
$$F_E = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$$

$$\lambda = \frac{L}{r}$$

	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape of column is shown by dashed line						
Theoretical *K* value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated	0.65	0.80	1.0	1.2	2.0	2.0
End condition code	Rotation fixed, Translation fixed Rotation free, Translation fixed Rotation fixed, Translation free Rotation free, Translation free					

How Geometry Affects Structural Strength

- Here we see four sections of very similar cross sectional areas.
  - From left to right we see better forms for axial design to better forms for flexural design.
  - Check the moment of inertia for each one.



HSS 8.625X0.322
I=68.1
WEIGHT=28.58

HSS 8X8X5/16
I=86.6
WEIGHT=31.84

W8X31
I=110
I(Y)=37.1

W14X30
I=291
I(Y)=19.6

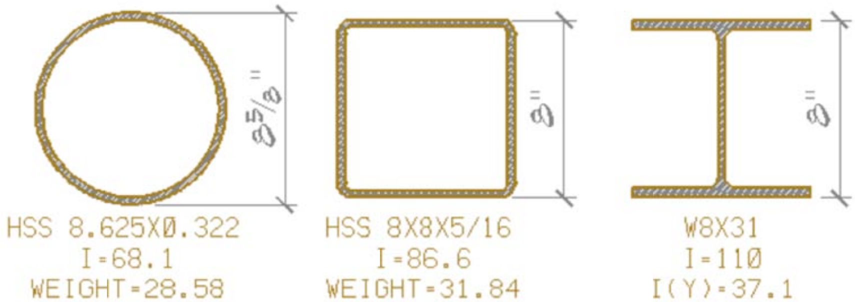
How Geometry Affects Structural Strength

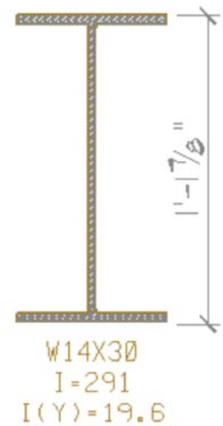
- Here are some basic differences:
  - An element that is efficient in flexural loads would have:

$$d \gg b_f \qquad r_x \gg r_y$$

  - An element that is efficient in axial loads would have:

$$d \approx b_f \qquad \frac{r_x}{r_y} \approx 1.6\, to\, 3$$

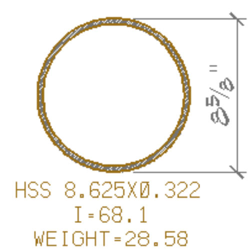




How Geometry Affects Structural Strength

- For the HSS shape @ F_y=42^{ksi}, L_u=15':

$$\frac{I_x}{I_y} = 1 \qquad r_x = r_y = 2.95 \qquad \Phi Pn = \underline{236\, kip}$$



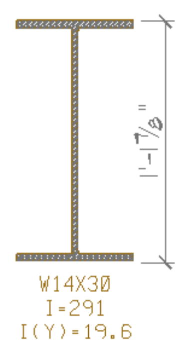
- For the W 14x30 @ F_y=50^{ksi}, L_u=15':

$$\frac{I_x}{I_y} = 14.85 \qquad \frac{r_x}{r_y} \approx 5.5 \qquad r_y = 1.49$$

$\Phi Pn = not\ even\ listed\ within\ Table\ 4-1\ of\ the\ AISC.\quad Using\ Table\ 4-22$

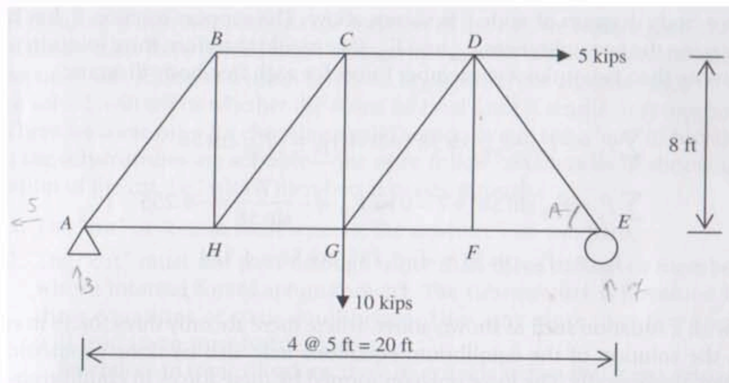
$$\frac{KL}{r} = \frac{1*15*12}{1.49} = 120.8 \qquad \Phi P_n = \Phi * A_g * F_{cr} \qquad \Phi * F_{cr}\quad from\ table \approx 15.5^{ksi}$$

$$A_g = 8.85^{inches} \qquad \Phi P_n \approx 15.5^{ksi} * 8.85^{inches} \approx \underline{137\, kip}$$



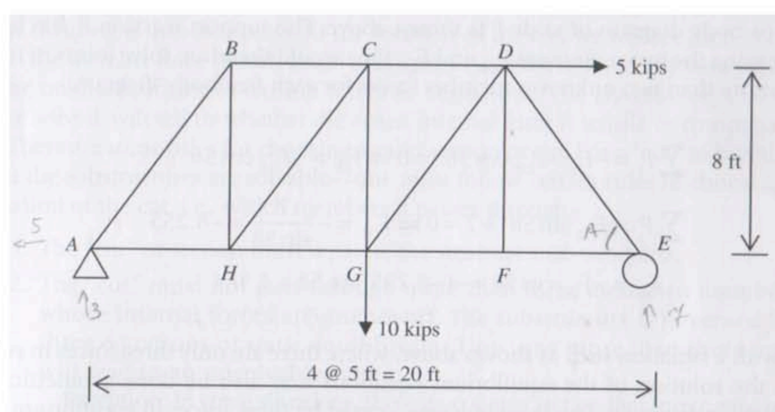
Trusses
A Direct Application of Axial Loading!

- A Truss is a special structure that transfers loads only axially
  - Loads may be applied along the outer elements, but the nodes pick those loads.
  - Elements may deflect a little, and as they deflect slightly the load is transferred axially through the nodes



Analyzing Trusses

- There are two basic methods to analyze, and design the members of a truss:
  - The method of joints involves the listing of all equations of equilibrium, proceeding joint by joint.
  - Each joint of a 2D (plane) truss yields two equilibrium equations — $\Sigma Fx = 0$ and $\Sigma Fy = 0$, where x and y are two mutually perpendicular axes in the plane.
  - On the other hand, the number of unknown quantities for the plane truss is the number of external reactions plus the number of member forces.



$$A = Atan\frac{8}{5} = 58 deg$$

Solution For the truss shown above, $R = 3$, $M = 13$, and $J = 8$. Thus, we have $R + M = 16 = 2J$. The truss is statically determinate.

Prior to solving for member forces, we need to solve for the external reactions. As long as there are no more than three external reactions, the truss is externally statically determinate.

$$\sum F_x = A_x + 5 = 0 \Rightarrow A_x = -5$$

$$\sum F_y = A_y - 10 + E_y = 0 \Rightarrow A_y + E_y = 10$$

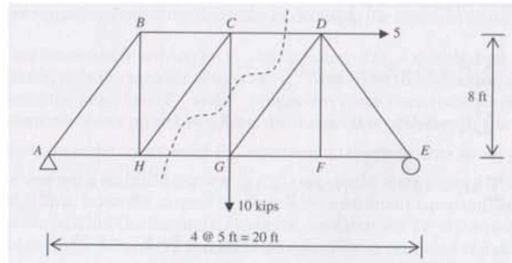
$$\sum M_A = -(10 \times 10) - (8 \times 5) + (20 \times E_y) = 0 \Rightarrow E_y = 7$$

$$\therefore \quad A_y = 10 - E_y = 3$$

- Joint Method:
  - The method of Joints, ...and
  - The method of Sections!

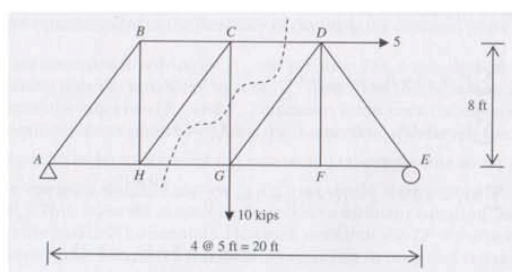
ANALYZING TRUSSES

- Section Method:
  - In order to apply the method of sections, we want to cut the structure with a section, thus dividing it into two parts. At the location of the cut, we replace each cut member with the member force (which is still unknown). As in the method of joints, we will insert these unknown forces as tension forces to begin with. The algebraic sign of the force, once solved, will tell us whether the actual internal force is tensile or compressive.
  - There are some rules for choosing a valid section or cut. For a "cut" to be valid—such that the substructures are solvable —one must follow certain rules in choosing the orientation of the cut, i.e., which members it passes through



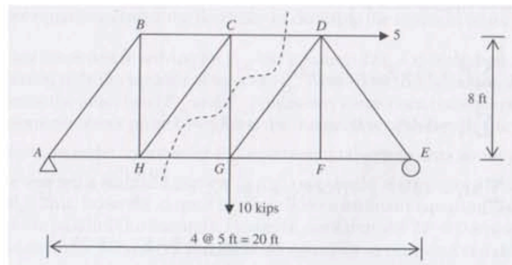
ANALYZING TRUSSES

- Section Method:
  - 1. The "cut" or section must separate the structure into two parts.
  - 2. The “cut” must not pass through more than three unknown members (i.e., whose internal forces are unknown).
  - The substructure is governed by the three equations of static equilibrium. Thus, any more than three unknowns will lead to an unsolvable system.
  - Exception: In some situations, there is no single section that meets criteria 1 and 2. In those cases, it may be necessary to make two successive sections, producing a system of six simultaneous equations in six unknown member forces.



Analyzing Trusses

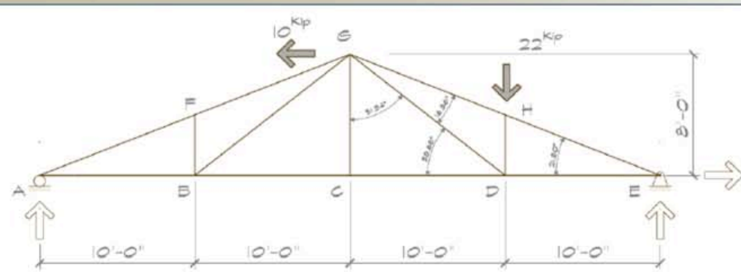
- Section Method:
  - 3.The unknown forces must not be concurrent. If these forces all pass through a common point, the problem will reduce to equilibrium of that point—producing two equations of equilibrium — ΣFx= 0 and ΣFy= 0.
  - The equation ΣM = 0 will be satisfied identically (0 = 0) and will not yield any useful information.



Analyzing Trusses

The diagram indicates the dimensions and the forces applied on a Pratt Truss. All elements are made of 60 Grade steel.

- Calculate the loads that correspond to each of the elements.
- Determine the dimensions of the critical element if it should have a square cross section.
- Determine the dimensions of the critical element if it has to be cylindrical.



Solution:

a) Analyzing the geometry and the forces in the truss using the joint method:

$F_y := 60ksi$ $L_{AC} := 20ft$ $L_{CG} := 8ft$

By taking sums of moments and sums of forces we can determine the reactions:

$L_{BC} := 10ft$ $L_{CD} := 10ft$ $L_{DE} := 10ft$

$F_{Ey} := \dfrac{22kip \cdot 30ft - 10kip \cdot 8ft}{40ft}$ $F_{Ey} = 14.5 \cdot kip$ $F_{Ex} := 10kip$ $F_{Ex} = 10 \cdot kip$ $L_{BF} := 4ft$ $L_{DH} := 4ft$

$F_{Ay} := 22kip - F_{Ey}$ $F_{Ay} = 7.5 \cdot kip$

Solving for the geometry:

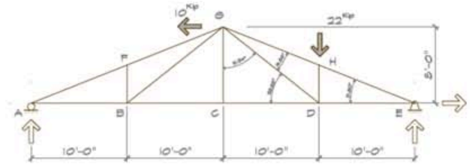
$L_{AG} := \sqrt{L_{AC}^2 + L_{CG}^2}$ $L_{AG} = 21.54\ ft$ $Angle_{BAF} := asin\left(\dfrac{L_{CG}}{L_{AG}}\right)$ $Angle_{BAF} = 21.8 \cdot deg$

ANALYZING TRUSSES

By observation:

$L_{AF} := \frac{L_{AG}}{2} = 10.77\,ft$ $\quad L_{FG} := L_{AF} = 10.77\,ft$ $\quad L_{GH} := L_{AF} = 10.77\,ft$

$L_{HE} := L_{AF} = 10.77\,ft$ $\quad Angle_{DEH} := Angle_{BAF} = 21.801 \cdot deg$



Solving for remaining geometry:

$L_{BG} := \sqrt{L_{BC}^2 + L_{CG}^2}$ $\quad L_{BG} = 12.81\,ft$ $\quad Angle_{CBG} := asin\left(\frac{L_{CG}}{L_{BG}}\right)$ $\quad Angle_{CBG} = 38.66 \cdot deg$

Solving for forces FHE and FDE:
By considering that the sum of vertical forces should be equal to 0, and using compression as -ve and tension as +ve...

$F_{HEy} := F_{Ey}$ $\quad F_{HEy} = 14.5 \cdot kip$ $\quad F_{HE} := \frac{F_{HEy}}{\sin(Angle_{DEH})}$ $\quad F_{HE} = 39.04 \cdot kip$ Compression

$F_{DE} := F_{HE} \cdot \cos(Angle_{DEH})$ $\quad F_{DE} = 36.25 \cdot kip$ Tension

Continuing with junction H (rotate it 21.6 degrees counterclockwise to your visual aid):

If we take the component of that 22kip load that is perpendicular to GE, that would be equalized only by a component reaction by DH. Therefore, the force on DH will be equal to the 22kip load.

$F_{DH} := 18kip$ Compression

Similarly, if we take elements GH and HE we can see that they take identical loads

$F_{GH} := F_{HE}$ $\quad F_{GH} = 39.04 \cdot kip$ Compression

Proof through the use of the Section Method: $(8ft \cdot F_{GH} \cdot \cos(Angle_{BAF})) + 20ft \cdot F_{GH} \cdot \sin(Angle_{BAF}) - 30ft \cdot 22kip + 8ft \cdot 10kip = 0 \cdot k'$

ANALYZING TRUSSES

Taking junction D:

$F_{GD} := \frac{F_{DH}}{\sin(Angle_{CBG})}$ $\quad F_{GD} = 28.81 \cdot kip$ Tension $\quad F_{CD} := F_{DE} - F_{GD} \cdot \cos(Angle_{CBG})$ $\quad F_{CD} = 13.75 \cdot kip$ Tension

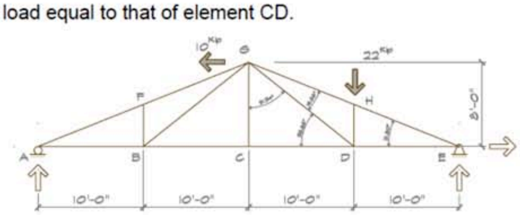
Junction C may be omitted as element GC is taking no loads and evidently element BC is taking a load equal to that of element CD.

$F_{BC} := F_{CD} = 13.75 \cdot kip$ Tension

An easy joint to solve for is joint A:

$F_{AFy} := F_{Ay}$ $\quad F_{AFy} = 7.5 \cdot kip$ $\quad F_{AF} := \frac{F_{AFy}}{\sin(Angle_{DEH})}$ $\quad F_{AF} = 20.19 \cdot kip$

$F_{AB} := F_{AF} \cdot \cos(Angle_{DEH})$ $\quad F_{AB} = 18.75 \cdot kip$



The above signifies that elements BF and BG take no load. Element BF would take equal vertical load to element BG. However, element BG has a horizontal component. There is no room for a horizontal component at this point on joint B because FAB=FBC. So the only value that would be a possible as horizontal contribution by element BG to that joint is 0!

Proof through section method:

$22kip \cdot 10ft + 10kip \cdot 8ft - 8ft\,F_{AF} \cdot \cos(Angle_{BAF}) - 20ft \cdot F_{AF} \cdot \sin(Angle_{BAF}) = 0 \cdot k'$

$F_{AB} = 18.75 \cdot kip$ $\quad F_{AF} = 20.19 \cdot kip$ $\quad F_{DH} = 18 \cdot kip$ $\quad F_{GD} = 28.81 \cdot kip$ $\quad F_{DE} = 36.25 \cdot kip$ $\quad F_{HE} = 39.04 \cdot kip$ $\quad F_{CD} = 13.75 \cdot kip$

$F_{BC} = 13.75 \cdot kip$ $\quad F_{FG} := F_{AF} = 20.19 \cdot kip$ $\quad F_{CG} := 0kip$ $\quad F_{BG} := 0kip$ $\quad F_{BF} := 0kip$ $\quad F_{GH} = 39.04 \cdot kip$

b) Determining the dimensions the critical element should have based on Euler's formula if the element has a square cross section:

It so happens that the element with the highest load is also the longest element in the truss, and that is "GD". BG has the same length but is subject to no load. However, element "GD" is under tension, so buckling is not an issue for that element. Therefore, the issue is shifted to elements "GH" and "HE".

$L_{DH} = 4\,ft$ $\quad L_{AF} = 10.77\,ft$ $\quad E := 29000ksi$ $\quad P_{crit} = \dfrac{\pi^2 \cdot E \cdot I}{(K \cdot L_u)^2}$

By considering that the element will be of square cross section we can define that the moment of inertia will be:

$I = \dfrac{b\,h^3}{12}$ or $I = \dfrac{b^4}{12}$ Substituting to Euler's formula and solving for the dimension "b": $b = \left(\dfrac{12 \cdot K^2 \cdot L_u^{\,2} \cdot P_{crit}}{\pi^2 \cdot E}\right)^{\frac{1}{4}}$

The factor K has beeen removed as it takes a value of "1"

$b_{HE} := \left(\dfrac{12 \cdot L_{HE}^{\,2} \cdot F_{HE}}{\pi^2 \cdot E}\right)^{\frac{1}{4}}$ $\quad b_{HE} = 2.29 \cdot in$ $\quad Area := b_{HE}^{\,2} = 5.229 \cdot in^2$

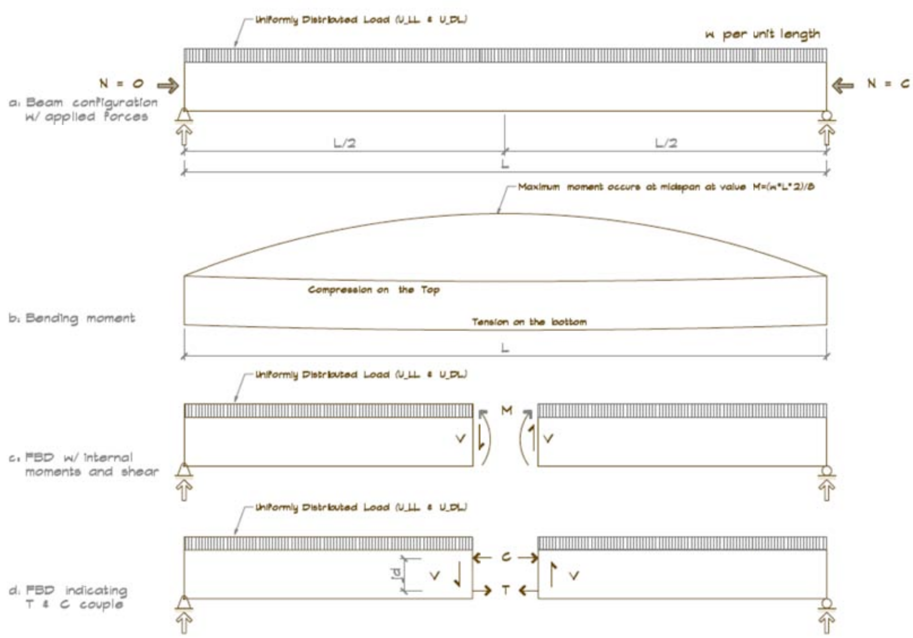
ANALYZING TRUSSES

c) Determining the dimensions the critical element should have based on Euler's formula if the element has a round cross section:

$I = \dfrac{\pi \cdot r^4}{4}$ ... Substituting... $r = \left(\dfrac{48 \cdot L_u^{\,2} \cdot P_{crit}}{\pi^3 \cdot E}\right)^{\frac{1}{4}}$ $\quad Dia_{HE} := 2\left(\dfrac{4 \cdot L_{HE}^{\,2} \cdot F_{HE}}{\pi^3 \cdot E}\right)^{\frac{1}{4}}$ $\quad Dia_{HE} = 2.61 \cdot in$ $\quad Area := \pi\left(\dfrac{Dia_{HE}}{2}\right)^2 = 5.351 \cdot in^2$

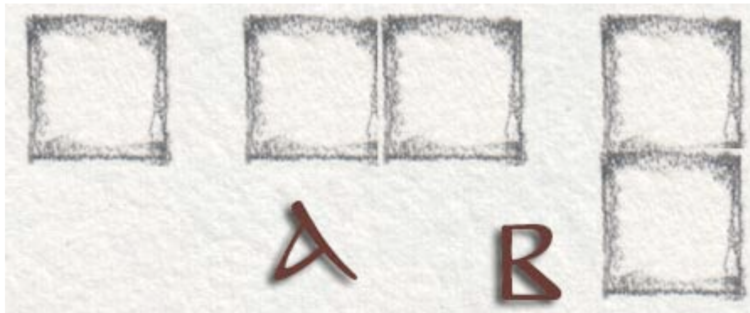
Bending Theory

Bending Theory



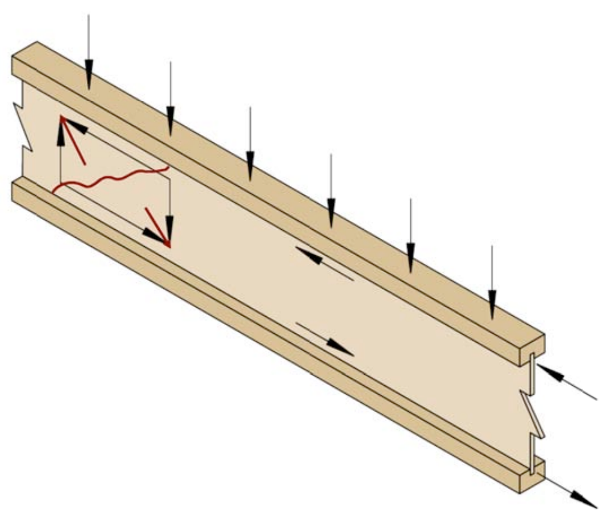
How Geometry Affects Structural Strength

- Which of the options on the right would you pick to double the size of a wooden beam?
  - Option A doubles the cross sectional area of the beam.
  - Option B also does that.
  - But the resistance the beam will develop depends on its Moment of Inertia:
    - Formula for "I" (Moment of Inertia): $I = \frac{b * h^3}{12} + A \cdot d^2$ *The A*d² component of the equation is applicable to parts that are not centered or to asymmetrical elements.*
    - Formula for "f_b" or "σ_b" flexural stress $f_b = \frac{M * c}{I}$ *Where "M" is the Moment, and "c" is the distance between the Neutral Axis and the extreme fiber.*
  - Option B has a larger "h" which is raised to the 3rd power!



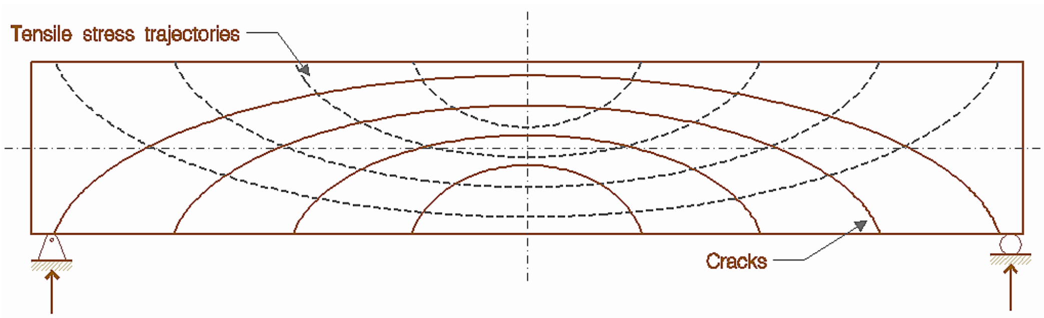
Material Behavior and Mechanics

- Take an infinitesimally small rectangular portion of this member and analyze the stresses developed:
  - The resultant will cause a diagonal tension.
  - Perpendicular to that diagonal will be the crack caused.
  - But what if the structural element is stronger on one side and weaker on the other.
  - Failure will occur "along the grain" not "against the grain" (see next slides)



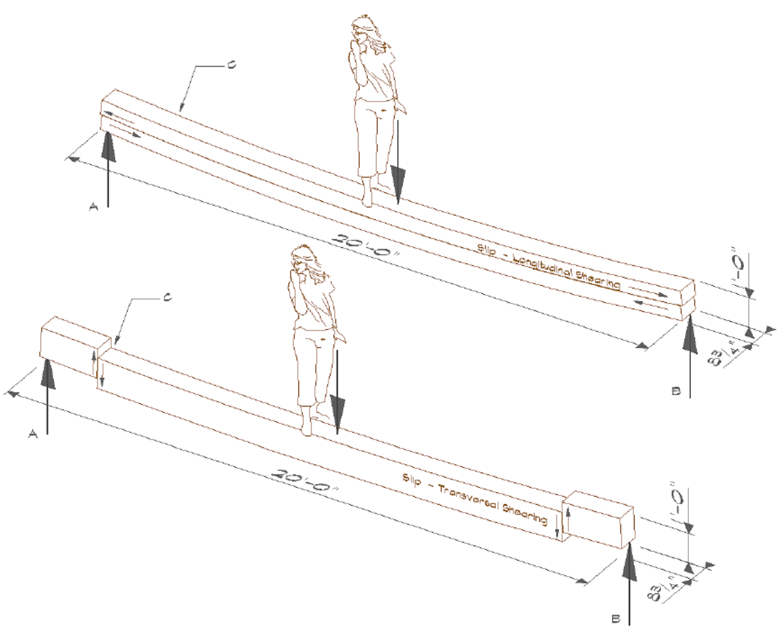
Material Behavior and Mechanics

- More specifically, the diagram below indicates the Tensile trajectories and the directions that cracks would occur due to shear in a beam



Material Behavior and Mechanics

Wood will shear more longitudinally along the weak direction of the fibers. The effect of transversal shear will be limited.



Shear to Moment

- There are standard formulae that can give us the <u>critical shear</u> and <u>moment</u> of most loaded beam configurations, but there are occasions where the location of loads does not help to use these standard formulae:

$$M_u = \frac{w \cdot L^2}{8}$$

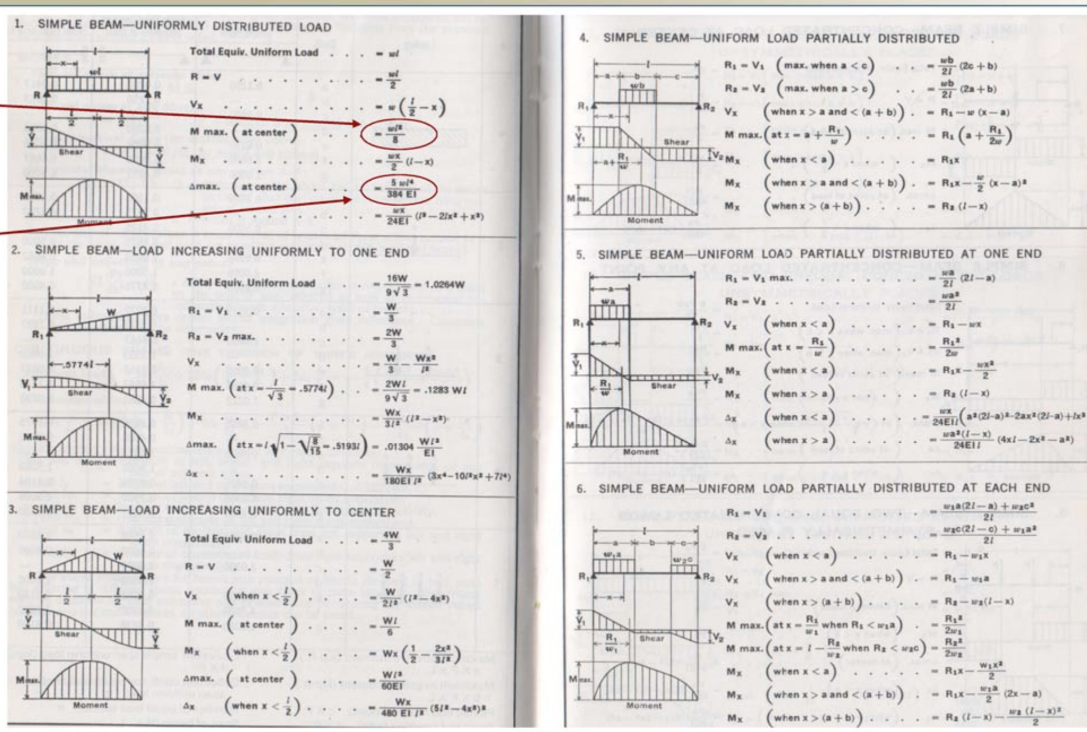
$$M_u = \frac{P \cdot L}{4}$$

- In those cases, the designer has to produce a Shear diagram and then a Moment diagram to find the critical shear and moment values

Standardized Load Configurations

$$M_u = \frac{w \cdot L^2}{8}$$

$$\Delta_{max} = \frac{5 \cdot w \cdot L^4}{384} E \cdot I$$

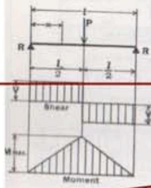


Standardized Load Configurations

$$M_u = \frac{P \cdot L}{4}$$

$$\Delta_{max} = \frac{P \cdot L^3}{48} E \cdot I$$

7. SIMPLE BEAM—CONCENTRATED LOAD AT CENTER



Total Equiv. Uniform Load . . . . . . $= 2P$

$R = V$ . . . . $= \frac{P}{2}$

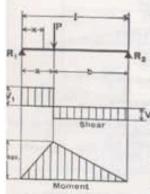
M max. (at point of load) . . . . $= \frac{Pl}{4}$

M_x (when $x < \frac{l}{2}$) . . . . . . $= \frac{Px}{2}$

Δmax. (at point of load) . . . . $= \frac{Pl^3}{48EI}$

Δx (when $x < \frac{l}{2}$) . . . . . . $= \frac{Px}{48EI}(3l^2 - 4x^2)$

8. SIMPLE BEAM—CONCENTRATED LOAD AT ANY POINT



Total Equiv. Uniform Load . . . . . . $= \frac{8Pab}{l^2}$

$R_1 = V_1$ (max. when $a < b$) . . . . . $= \frac{Pb}{l}$

$R_2 = V_2$ (max. when $a > b$) . . . . . $= \frac{Pa}{l}$

M max. (at point of load) . . . . . $= \frac{Pab}{l}$

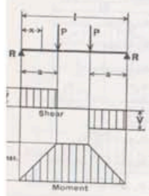
M_x (when $x < a$) . . . . . $= \frac{Pbx}{l}$

Δmax. $\left(\text{at } x = \sqrt{\frac{a(a+2b)}{3}} \text{ when } a > b\right) = \frac{Pab(a+2b)\sqrt{3a(a+2b)}}{27EIl}$

Δa (at point of load) . . . . . $= \frac{Pa^2b^2}{3EIl}$

Δx (when $x < a$) . . . . . $= \frac{Pbx}{6EIl}(l^2 - b^2 - x^2)$

9. SIMPLE BEAM—TWO EQUAL CONCENTRATED LOADS SYMMETRICALLY PLACED



Total Equiv. Uniform Load . . . . . . $= \frac{8Pa}{l}$

$R = V$ . . . . $= P$

M max. (between loads) . . . . . . $= Pa$

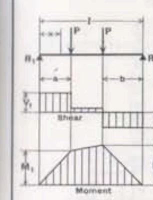
M_x (when $x < a$) . . . . . $= Px$

Δmax. (at center) . . . . $= \frac{Pa}{24EI}(3l^2 - 4a^2)$

Δx (when $x < a$) . . . . $= \frac{Px}{6EI}(3la - 3a^2 - x^2)$

Δx (when $x > a$ and $< (l-a)$) . . . $= \frac{Pa}{6EI}(3lx - 3x^2 - a^2)$

10. SIMPLE BEAM—TWO EQUAL CONCENTRATED LOADS UNSYMMETRICALLY PLACED



$R_1 = V_1$ (max. when $a < b$) . . . . . $= \frac{P}{l}(l - a + b)$

$R_2 = V_2$ (max. when $a > b$) . . . . . $= \frac{P}{l}(l - b + a)$

V_x (when $x > a$ and $< (l-b)$) . . $= \frac{P}{l}(b - a)$

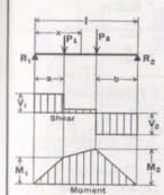
M_1 (max. when $a > b$) . . . . . $= R_1a$

M_2 (max. when $a < b$) . . . . . $= R_2b$

M_x (when $x < a$) . . . . $= R_1x$

M_x (when $x > a$ and $< (l-b)$) . . $= R_1x - P(x - a)$

11. SIMPLE BEAM—TWO UNEQUAL CONCENTRATED LOADS UNSYMMETRICALLY PLACED



$R_1 = V_1$ . . . . $= \frac{P_1(l-a) + P_2b}{l}$

$R_2 = V_2$ . . . . $= \frac{P_1a + P_2(l-b)}{l}$

V_x (when $x > a$ and $< (l-b)$) . . $= R_1 - P_1$

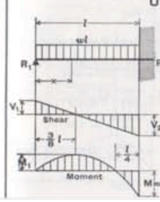
M_1 (max. when $R_1 < P_1$) . . . . $= R_1a$

M_2 (max. when $R_2 < P_2$) . . . . $= R_2b$

M_x (when $x < a$) . . . . $= R_1x$

M_x (when $x > a$ and $< (l-b)$) . . $= R_1x - P_1(x - a)$

12. BEAM FIXED AT ONE END, SUPPORTED AT OTHER—UNIFORMLY DISTRIBUTED LOAD



Total Equiv. Uniform Load . . . . . . $= wl$

$R_1 = V_1$ . . . . $= \frac{3wl}{8}$

$R_2 = V_2$ max. . . . . $= \frac{5wl}{8}$

V_x . . . . $= R_1 - wx$

M max. . . . . $= \frac{wl^2}{8}$

M_1 (at $x = \frac{3}{8}l$) . . . . $= \frac{9}{128}wl^2$

M_x . . . . $= R_1x - \frac{wx^2}{2}$

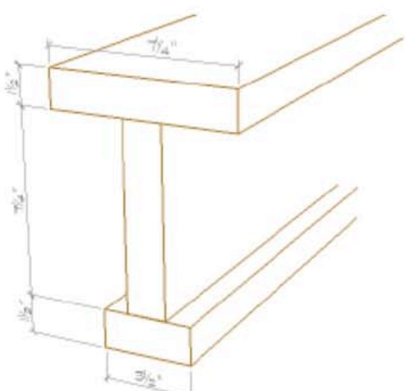
Δmax. (at $x = \frac{l}{16}(1 + \sqrt{33}) = .4215l$) . $= \frac{wl^4}{185EI}$

Δx . . . . $= \frac{wx}{48EI}(l^3 - 3lx^2 + 2x^3)$

Exercise

A wooden beam spanning 20' is designed with two 2x8 and a 2x4 elements as shown in the figure.

- Calculate the moment of inertia of the designed beam.
- If the wood can take a maximum stress of 1^{ksi} what would be the maximum uniformly distributed load that can be applied on this designed element?



1) Calculating the moment of inertia of the element:
a) Starting by calculating the center of gravity of the composed element:
Taking the bottom extreme fiber as reference:

$$C_g := \frac{\left[0.5 \cdot W_{24} \cdot (A_{24})\right] + \left[(W_{24} + 0.5 \cdot L_{28}) \cdot (A_{28})\right] + \left[(W_{24} + L_{28} + 0.5 \cdot W_{28}) \cdot (A_{28})\right]}{2(A_{28}) + (A_{24})}$$

$C_g = 6.04 \cdot in$

EXERCISE

b) Calculating the moment of inertia of each individual portion:

$d_1 := \left| C_g - \frac{W_{24}}{2} \right|$ $d_1 = 5.29 \cdot in$ $I_1 := \frac{L_{24} \cdot W_{24}^3}{12} + A_{24} \cdot d_1^2$ $I_1 = 147.7 \cdot in^4$

$d_2 := \left| C_g - \left(\frac{L_{28}}{2} + W_{24} \right) \right|$ $d_2 = 0.91 \cdot in$ $I_2 := \frac{W_{28} \cdot L_{28}^3}{12} + A_{28} \cdot d_2^2$ $I_2 = 56.67 \cdot in^4$

$d_3 := \left| C_g - \left(\frac{W_{28}}{2} + L_{28} + W_{24} \right) \right|$ $d_3 = 3.46 \cdot in$ $I_3 := \frac{L_{28} \cdot W_{28}^3}{12} + A_{28} \cdot d_3^2$ $I_3 = 132.5 \cdot in^4$

$I_{tot} := I_1 + I_2 + I_3$ $I_{tot} = 336.87 \cdot in^4$

EXERCISE

2) Calculating the maximum moment to be lead to the maximum uniformly distributed load:
Note: The formula for stress contains the moment and the distance "c" of the extreme fiber to N/A on the numerator. This beam is not symmetrical along the x-axis. The distance "c" on top is shorter than the distance "c" on the bottom. Therefore, the bottom will experience higher stress at any value of applied moment. The bottom fiber will fail first. Therefore it is the distance "c" on the bottom that needs to be used. That distance "c" happens to be the distance calculated above as Cg for center of gravity since the N/A is at the center of gravity of the composed element.

$f_b = \frac{M \cdot c}{I}$ $c := C_g$ $M := \frac{f_b \cdot I_{tot}}{c}$ $M = 4.65 \cdot k'$

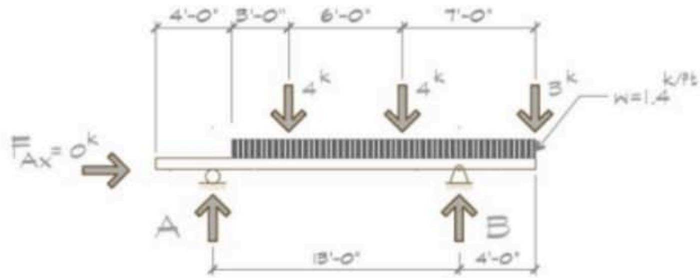
3) Calculating the applied load:

$M = \frac{w \cdot L^2}{8}$ $w := \frac{8 \cdot M}{L_b^2}$ $w = 93.01 \cdot \frac{lbf}{ft}$

Shear to Moment

- Example:

Construct a shear diagram for the following load distribution diagram:



Beam's Length $L_{beam} := 20ft$

Support Locations: $x_{RA} := 3ft$ $x_{RB} := 16ft$

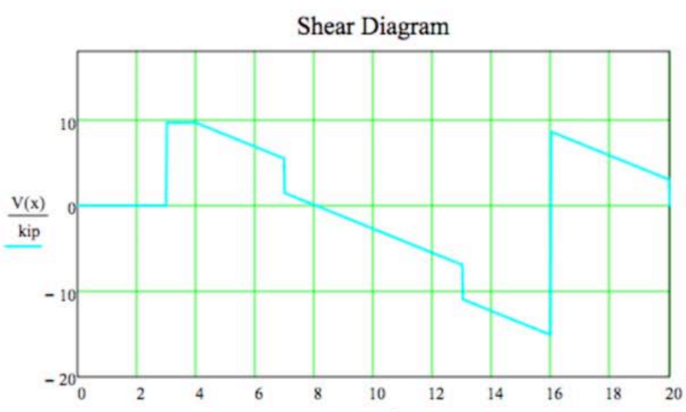
Other UDLs: $w_1 := 1.4\frac{kip}{ft}$ $w_2 := 0\frac{kip}{ft}$

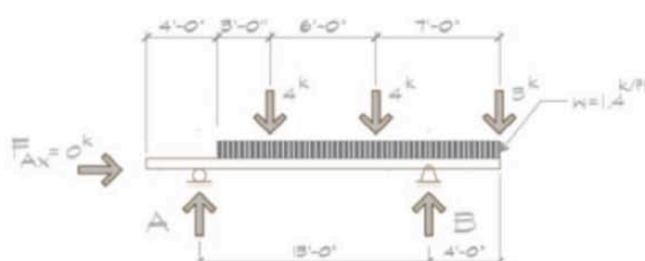
Beginning and end locations of uniformly distributed loads: $x_{Lw1} := 4f$ $x_{Rw1} := 20ft$ $x_{Lw2} := 0f$ $x_{Rw2} := 0ft$

Resultant locations of uniformly distributed loads:

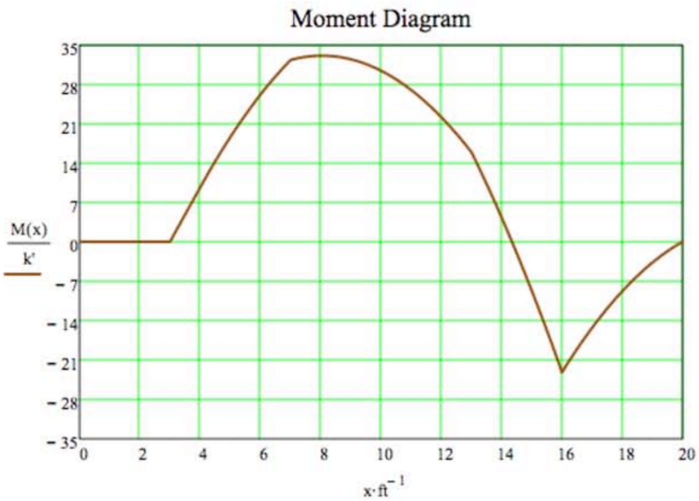
$$Res_{w1} := x_{Lw1} + \frac{x_{Rw1} - x_{Lw1}}{2} \quad Res_{w1} = 12\,ft$$

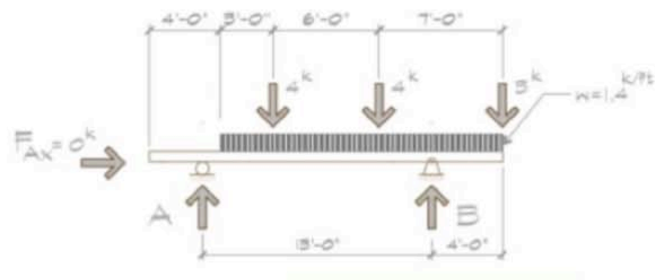
$$Res_{w2} := x_{Lw2} + \frac{x_{Rw2} - x_{Lw2}}{2} \quad Res_{w2} = 0$$





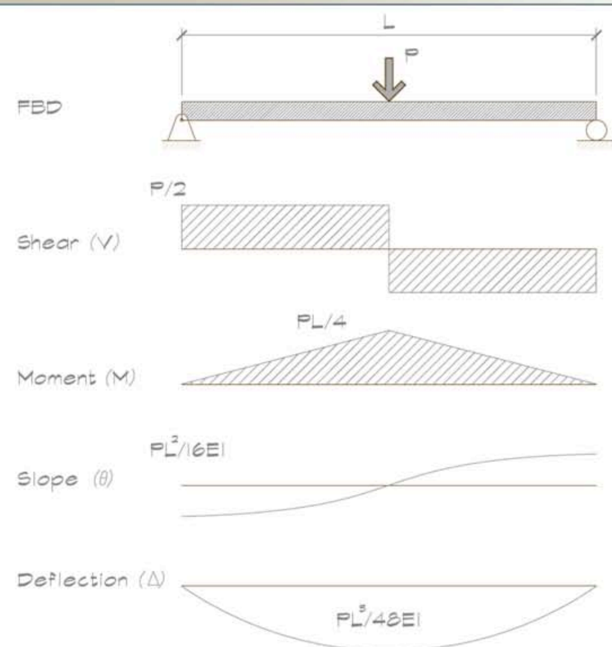
Shear to Moment





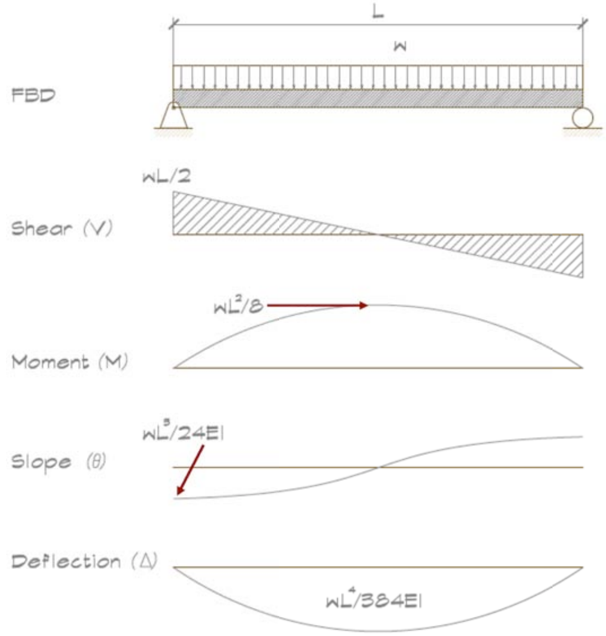
Deflection – Double Integral of Moment

- The deflection of a flexural beam
  - We have seen that the Moment diagram is the integral of the Shear diagram.
  - The Moment diagram is not something that we can observe by visually observing a flexural member. However, what is visually evident is the deflection that it is subjected to.
  - The actual relation of that physical effect can be produced through the double integration of the Moment Diagram.



Deflection – Double Integral of Moment

- The deflection of a flexural beam
  - Similar to the case of the simply supported beam with a point load in the middle, here we see the results of integration at all levels with a simply supported beam with uniformly distributed load.
  - These are very specific formulae but can be used in most situations, especially this scenario. However, it is quite probable to see them combined and/or adjusted according to the specificity of the design.



Limits of Deflection

- Deflection can be a failure
  - Granted the redundant statement that “if a Structure fails to do what it is supposed to do, it fails” the principle is applicable to deflection as well.
  - Some deflection should be anticipated, but there are limits given by codes that should not be surpassed:
  - The table gives the limits according to the American Concrete Institute Code 318-14/Table 24.2.2:

<table>
<tr><th>Member</th><th colspan="2">Condition</th><th>Deflection to be Considered</th><th>Limitation</th></tr>
<tr><td>Flat Roofs</td><td colspan="2" rowspan="2">Not supporting or attached to nonstructural elements likely to be damaged by large deflections</td><td>Immediate deflection due to max L_r, S, or R</td><td>L/180</td></tr>
<tr><td>Floors</td><td>Immediate deflection due to L</td><td>L/360</td></tr>
<tr><td rowspan="2">Roofs and Floors</td><td rowspan="2">Supporting or attached to non structural elements</td><td>Likely to be damaged by large deflections</td><td rowspan="2">That part of the total deflection occurring after attachment of nonstructural elements, which is the sum of the time-dependent deflection due to all sustained loads and immediate deflection due to any additional live load</td><td>L/480</td></tr>
<tr><td>Not likely to be damaged by large deflections</td><td>L/240</td></tr>
</table>

Lateral Loads

Some Hints on Lateral Loads

- These are just random hints pertaining to Lateral Loads for the ARE
  - Historic Buildings may need to be retrofitted to withstand seismic and wind loads because they were likely not designed for them initially
  - Wood connections have a special factor of safety (1.6) added to address lateral loads
  - It is unnecessary to design for Wind and Seismic loads as if they would act concurrently
  - Wind may cause substantial uplift to structures that may put elements such as columns that were in compression act in tension to hold the roof that uplifts
  - Negative pressure often governs over positive pressure. It is safer to design all openings for the higher value rather than anticipate that wind will always have the same direction
  - Smooth terrain produces laminar wind flow, whereas uneven/rough terrain generates turbulence
  - Most of the US receives a high wind value of 90^{mph}. Coastlines are much more prone to have higher wind values
  - Drift occurs due to both wind and earthquake. Drift should not exceed 0.002 x building height
  - Window Design Pressure (DP) needs to be determined for every project according to location so that windows will have withstand the anticipated pressures
  - Although there is a lot of hype about it, rain does not increase wind load dramatically

Seismic Loads

On Lateral Loads

- The effect of lateral loads depends upon the rotation that the force may generate:
  - Consider that you have a location where the resultant force is going to be applied,
  - And then consider that there will be a point that will remain stable.
  - That second point will be acting in a fashion similar to a hinge of a door.
  - The force acting on the center of mass will cause a torsional effect and the building will be rotating!

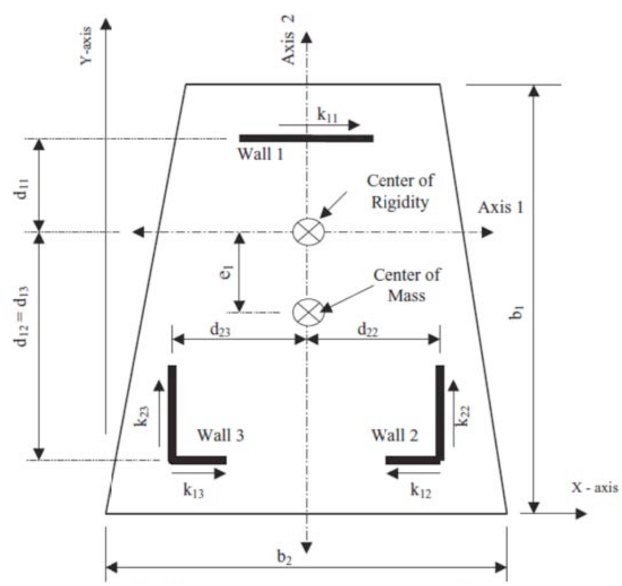
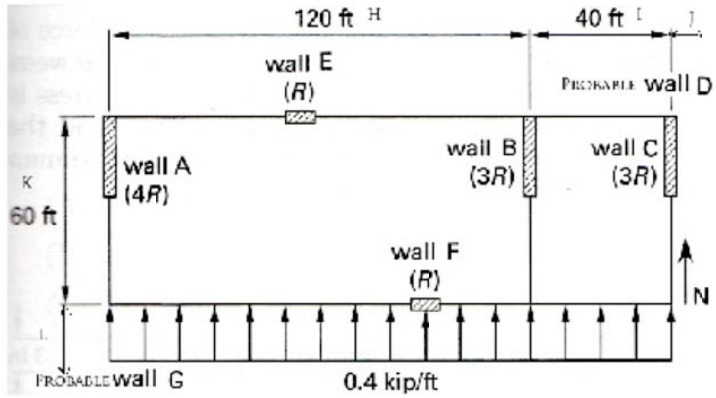


FIGURE 12.14-1 Notation Used in Torsion Check for Nonflexible Diaphragms

On Lateral Loads

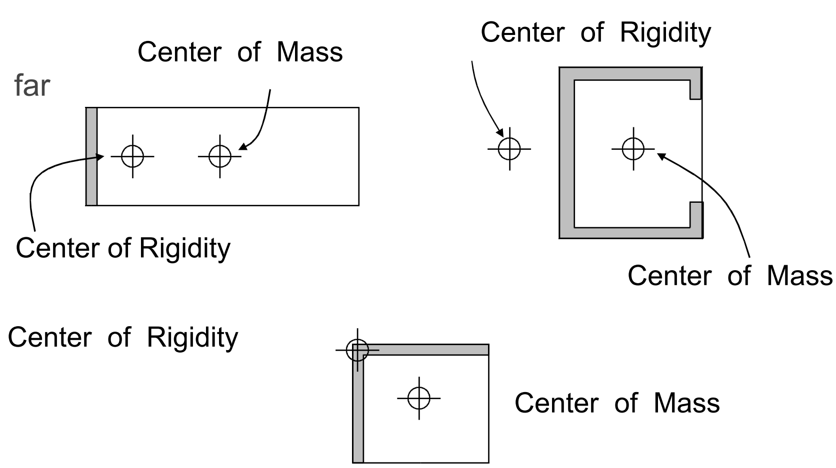
- Determining the center of gravity of a simple structure:
  - A problem like this will very likely be on the exam. However, the exam does not offer much time to go much deeper than this basic level.
  - Let's determine the center of Gravity of this along the x axis!
    - *Disregard the indicated wind load on the y axis. This is a graphic that was used for a different and more advanced problem.*



$$G_{c_x} = \frac{\sum R \cdot d}{\sum R} = \frac{4R \cdot 0^{ft} + 3R \cdot 120^{ft} + 3R \cdot 160^{ft}}{4R + 3R + 3R} = 64.62^{ft}$$

Problems with Plan Eccentricity

- The effect of lateral loads depends upon the rotation that the force may generate:
  - Plan eccentricity increases deformation demand on elements far from center of rigidity

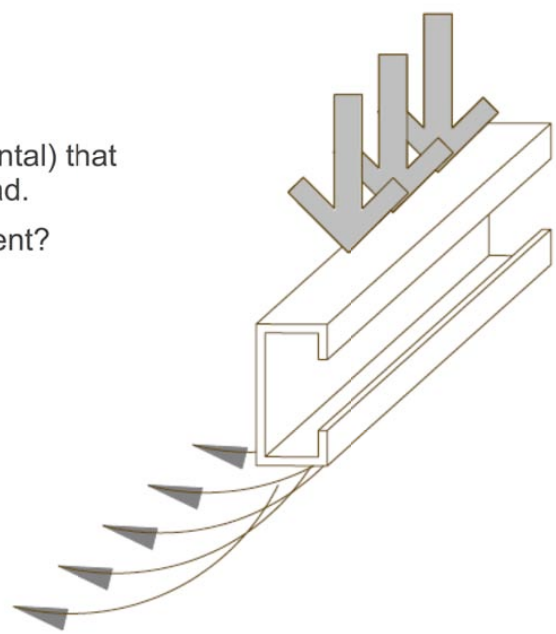


Lesson Learnt from Peru

- Photo, courtesy of Dr. R. Klingner:
  - The Embassy Hotel at Pisco Peru. A corner building.
  - Results of moment generated due to the distance of Center of gravity (mass) and center of shear (rigidity).

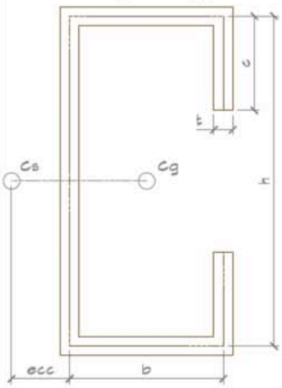
How Geometry Effects Behavior

- Center of Gravity vs Center of Shear:
  - Try to visualize a channel (C Section) spanning horizontally with the web vertical (flanges horizontal) that is subject to a uniformly distributed or a point load.
  - How do you envision the deflection of that element?



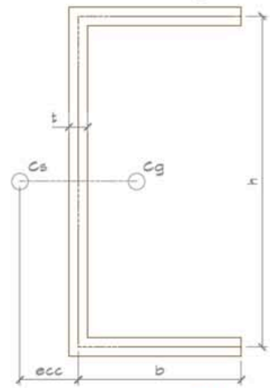
How Geometry Effects Behavior

- Center of Gravity vs Center of Shear:
  - Compare where the center of gravity is located with respect to the center of shear (also known as center of rotation or center of rigidity)
  - The distance between the point that receives the resultant force (Center of mass or Center of gravity) and the center of shear constitutes a moment-arm that will generate a rotation.



$$C_G=\frac{4b^2-8bt+4cb+t^2}{2(2b+2c+h)}$$

$$ecc=b\cdot\left[\frac{3h^2b+6h^2c-8c^3}{h^3+6h^2b+6h^{2c}+8c^3-12hc^2}\right]$$



$$C_G=\frac{(t-2b)^2}{2\cdot(2b+h)}$$

$$ecc=\frac{3t_f b^2}{h\cdot t_w+6b\cdot t_f}$$

Lateral Strength

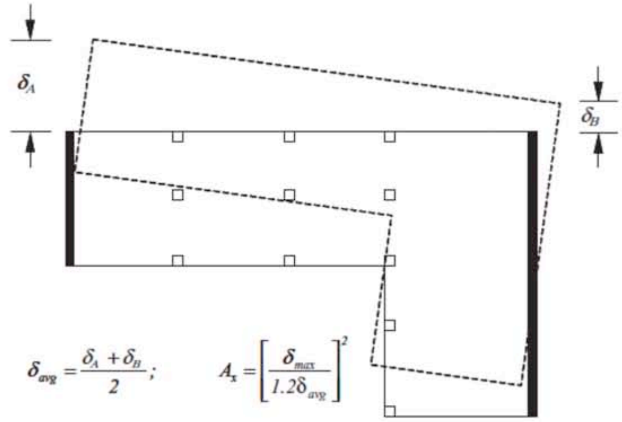
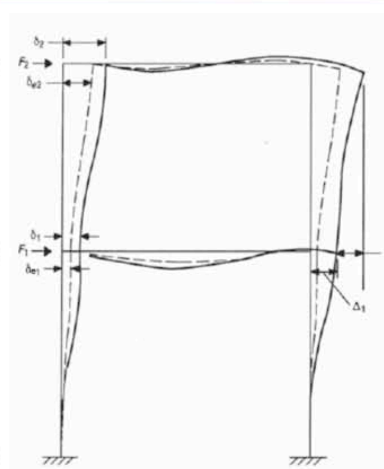


FIGURE 12.8-1 Torsional Amplification Factor, A_x



Story Level 2

- F_2 = strength-level design earthquake force
- δ_{e2} = elastic displacement computed under strength-level design earthquake forces
- δ_2 = $C_d\,\delta_{e2}/I_E$ = amplified displacement
- Δ_2 = $(\delta_{e2}-\delta_{e1})\,C_d/I_E \leq \Delta_a$ (Table 12.12-1)

Story Level 1

- F_1 = strength-level design earthquake force
- δ_{e1} = elastic displacement computed under strength-level design earthquake forces
- δ_1 = $C_d\,\delta_{e1}/I_E$ = amplified displacement
- Δ_1 = $\delta_1 \leq \Delta_a$ (Table 12.12-1)

- Δ_i = Story Drift
- Δ_i/L_i = Story Drift Ratio
- δ_2 = Total Displacement

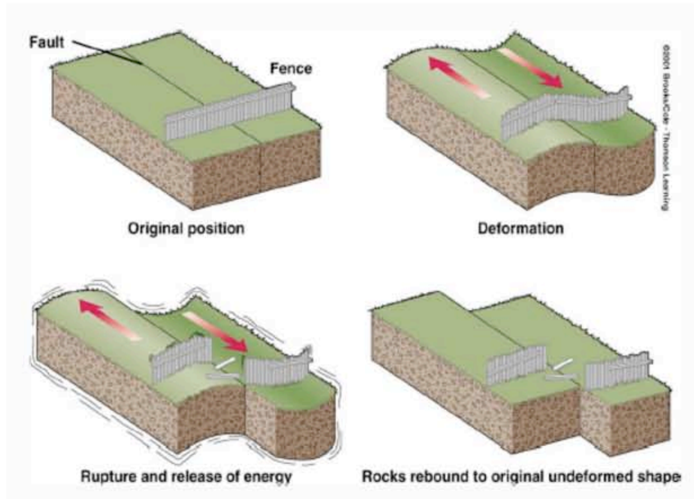
FIGURE 12.8-2 Story Drift Determination

- I don't believe that there will be anything that complex in the exam. You certainly do not have the time to do this!

Earthquake Basics

- What are Earthquakes?

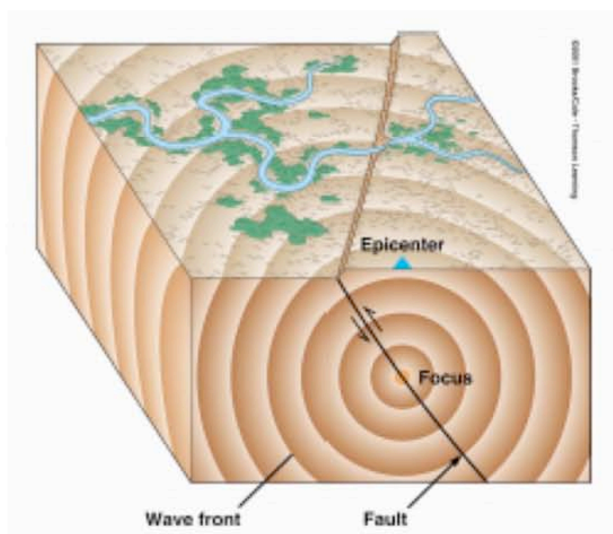
  - Movement of soil caused by shifting of tectonic plates.
  - Every material (including rocks and soil) have a certain elasticity and plasticity. Once it is surpassed an effect similar to that of fracture comes to the forefront!
  - The potential energy that is stored in the system will be released to kinetic energy, resulting in movement of the soil, liquefaction, and after effects such as tsunami, landslides, fires etc.



Earthquake Basics

- The geometry of Earthquakes?

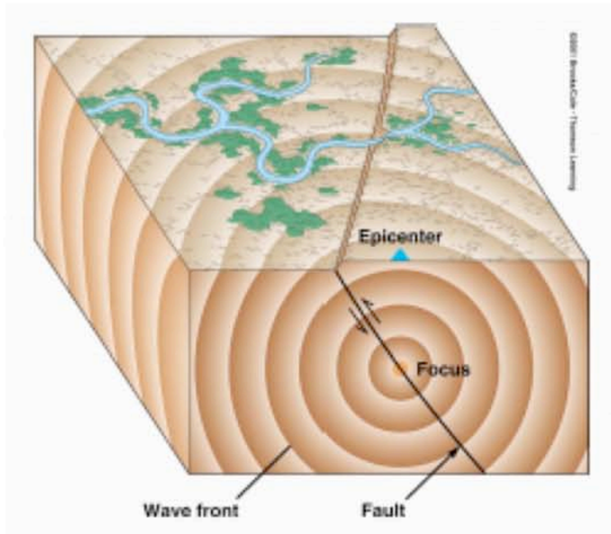
  - The "Focus" of the Earthquake is the point beneath the surface where the failure begins. You can see that as the actual "center" of the earthquake
  - The "epi-center" (which means point above the center) is the location directly above, on the surface of the earth that we shall likely experience the maximum energy emitted by the quake.



EARTHQUAKE BASICS

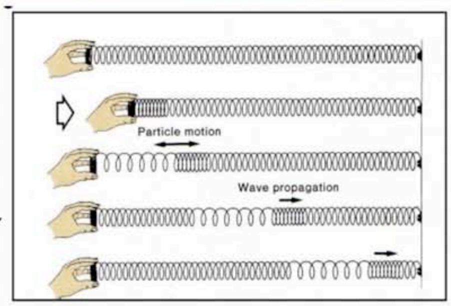
- The Earthquakes Waves
  - Earthquakes propagate in the form of waves. Those waves are the response of material substance to the energy fronts that were released after the effect that we consider as rupture (the point beyond the plastic region where materials break apart)

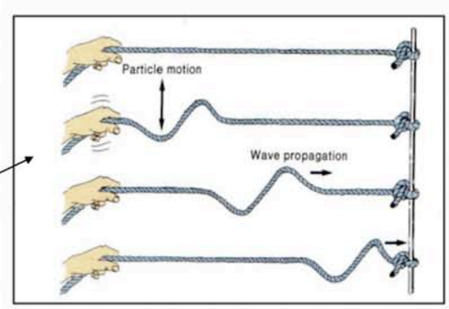
- There are two types of waves:
  - Body waves - P(rimary) & S(econdary)
  - Surface Waves - R(eileigh) & L(ove)



EARTHQUAKE WAVE PROPAGATION

- The P & S waves are "Body waves"
- P or primary waves
  - They are faster waves
  - They travel through solids, liquids, or gases (compressional wave action),
  - Material movement is in the same direction as wave movement
- S or secondary waves
  - They are slower than P waves
  - They travel through solids only
  - They are shear waves - move material perpendicular to wave movement

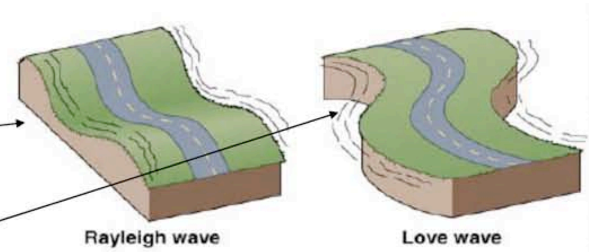




Earthquake Wave Propagation

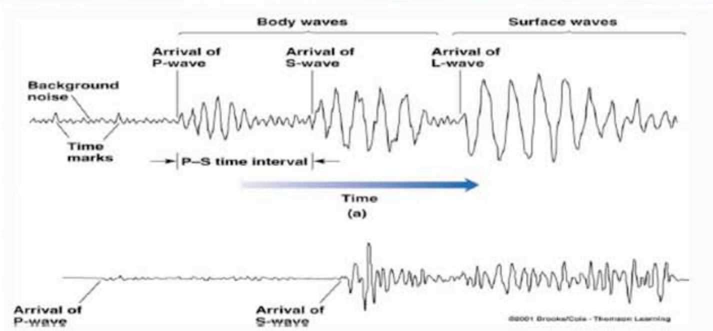
- The R & L waves are "Surface waves"
- Both types of Surface waves are particularly damaging to buildings.

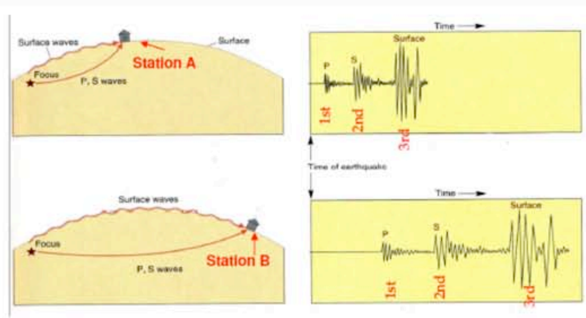
- Rayleigh waves
  - They are characterized by a modulation in terms of height
- Love waves
  - They modulate left and right along the length of their path



Earthquake Wave Propagation

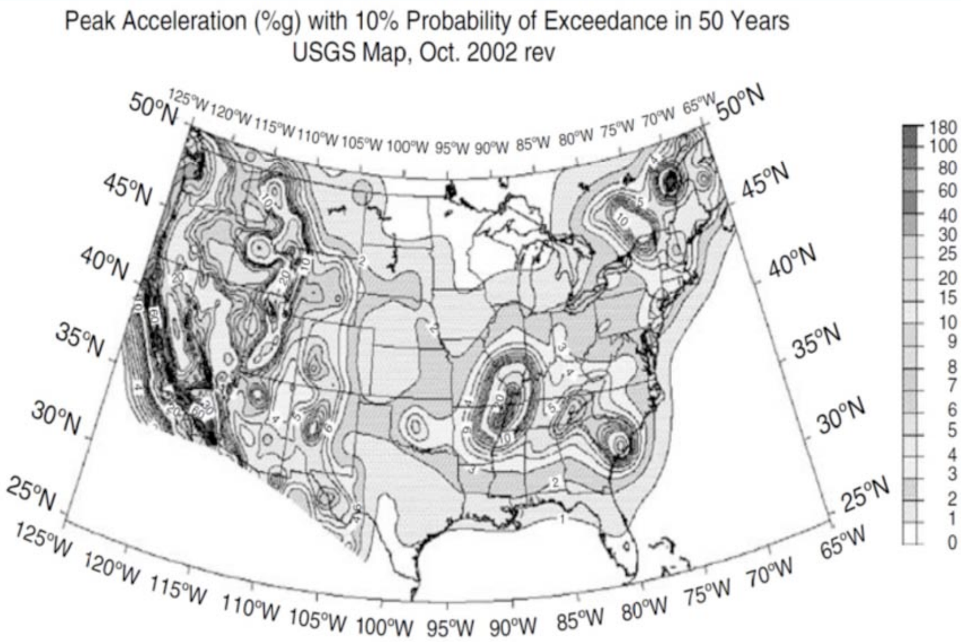
- Seismic wave behavior
- P waves arrive first, then S waves follow, and then the L and R will arrive
- The average speeds for all these waves is known
- After an earthquake, the difference in arrival times at a seismograph station can be used to calculate the distance from the seismograph to the epicenter.





Earthquake US Map

- The US Geological S
produced this map for t
  - It rates regions of the US a
  to how earthquake prone they



Estimating Earthquake Forces

- How to estimate the maximum forces generated by the earthquake?

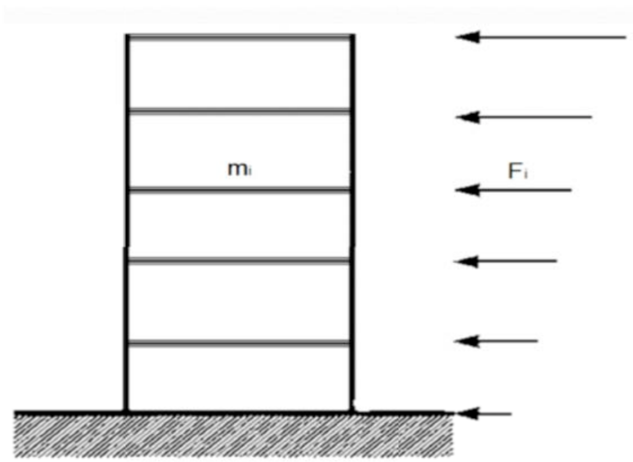
- For each specific site a Maximum Considered Earthquake (MCE) is defined. This is an event with a 2% probability of exceedence in 50 years or a Tr ~ 2500 years.
- There are three Methods to obtain seismic forces

1. Simplified Lateral Analysis (Equivalent Lateral Force analysis)
2. Spectral Analysis-Modal Response
3. Time History Analysis

- The American Society of Civil Engineers and its branch the Structural Engineering Institute produce a very useful reference that is coded as ASCE/SEI 7, which provides data for the calculation of seismic loads

Simplified Earthquake Analysis

- A simplified Method
  - It is a method where the seismic deformations are assumed to increase linearly according to the height of the building:

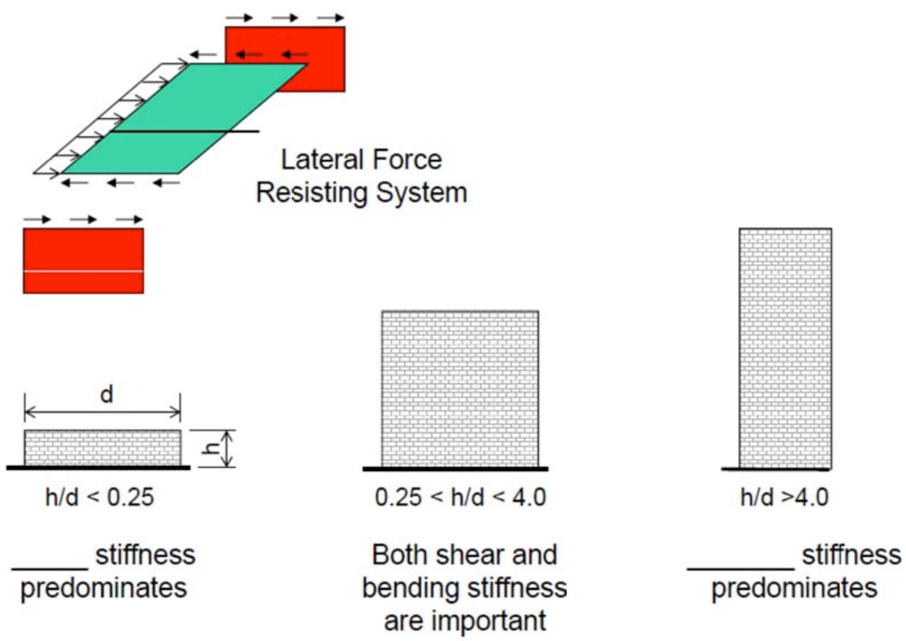


Estimating Earthquake Forces

- A step by step process:
  - For each specific site a Maximum Considered Earthquake (MCE) is defined. This is an event
  - Step1- Determination of maximum considered earthquake and design spectral response accelerations:
    1) Determine the mapped maximum considered earthquake MCE spectral response accelerations
    2) Determine the site class based on the soil properties
    3) Determine the maximum considered earthquake spectral response accelerations adjusted for site class effects "SM"
    4) Determine the 5% damped design spectral response accelerations "SD"
  - Step 2-Determination of seismic design category andImportance factor
  - Step 3-Determination of the Seismic Base Shear
  - Step 4-Vertical Distribution of Seismic Forces
  - But this reaches beyond the boundaries of what should be anticipated in the Architectural Record Examinations, so it may be optimum to move to another subject...

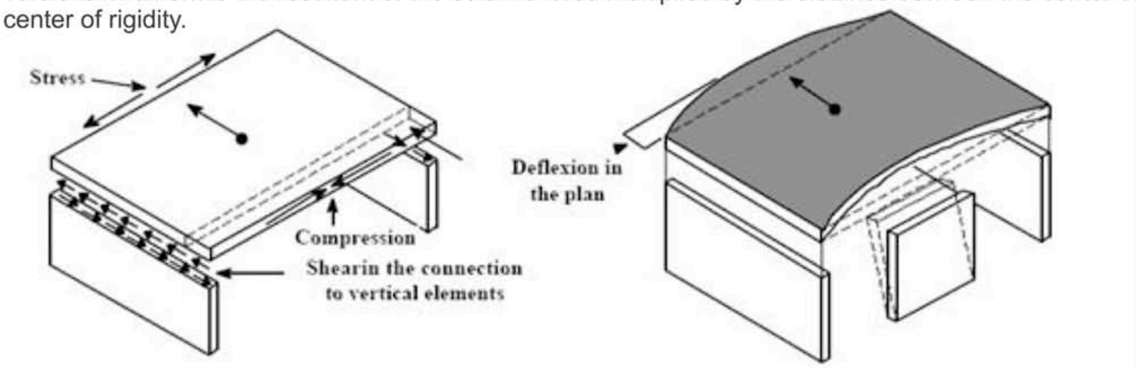
Wind Loads

How do Shear Walls work?



On Lateral Loads

- What is the difference between a rigid and a flexible diaphragm?
  - Plywood and un-topped steel decks may be considered flexible.
  - Concrete and concrete filled metal decks are considered rigid.
  - A flexible diaphragm moves more than 2 times the drift of the adjacent vertical bracing elements.
  - Seismic forces are distributed based on tributary areas on structures with flexible diaphragms like a beam with elastic qualities would.
  - Torsional moment is taken into account in structure with rigid diaphragms for the distribution of the seismic forces.
  - Torsional moment is the resultant of the seismic force multiplied by the distance between the center of mass and the center of rigidity.



Concentration of stress in rigid diaphragm

Deflection of stress in flexible diaphragm

On Lateral Loads

- The effect of lateral loads depends upon the rotation that the force may generate:
  - Consider that you have a location where the resultant force is going to be applied,
  - And then consider that there will be a point that will remain stable.
  - That second point will be acting in a fashion similar to a hinge of a door.
  - The force acting on the center of mass will cause a torsional effect and the building will be rotating!

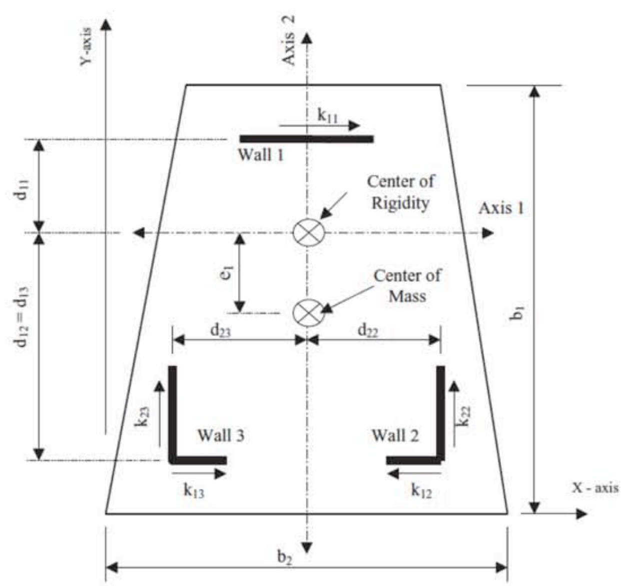


FIGURE 12.14-1 Notation Used in Torsion Check for Nonflexible Diaphragms

WIND LOADS

- What would be applicable codes?

WIND SPEED IN BALTIMORE

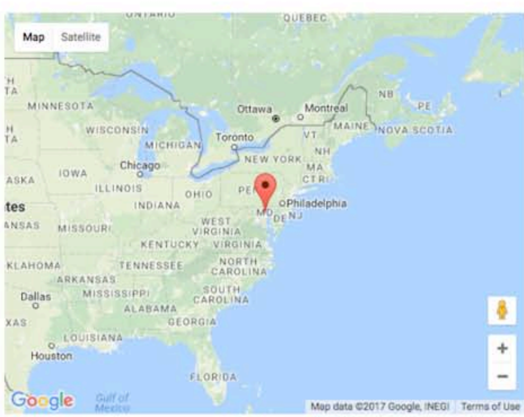
Search Results

Query Date: Sat Jun 03 2017
Latitude: 39.2904
Longitude: -76.6122

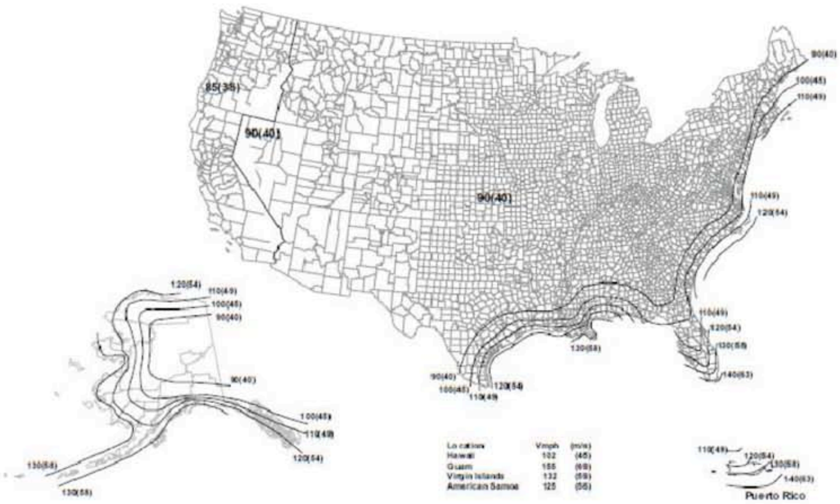
ASCE 7-10 Windspeeds (3-sec peak gust in mph*):

Risk Category I: 105
Risk Category II: 115
Risk Category III-IV: 120
MRI 10-Year:** 76
MRI 25-Year:** 84
MRI 50-Year:** 90
MRI 100-Year:** 96

ASCE 7-05 Windspeed:
90 (3-sec peak gust in mph)
ASCE 7-93 Windspeed:
70 (fastest mile in mph)

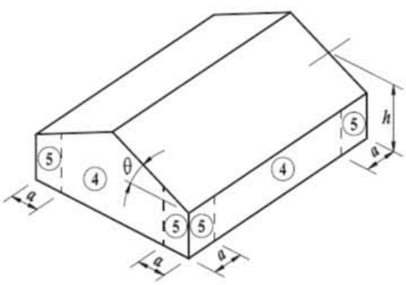


- The issue with the codes?
- 2012 IBC and 2012 IRC don't deal with this issue very well.
- Simplifications for the residential code make things confusing for the IBC!
- So what do you do?
    - 2012 IBC: multiply design pressures by 0.6
    - 2012 IRC: use Table R301.2(2) without any adjustments



Notes:
1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10 m) 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.

- Example
- SFD in Charleston, SC
- Suburban environment
- MRH less than 30 feet
- Window size = 20 square feet
- Find required DP using 2012 IRC
- Window in corner zone



- Solution:
- By extrapolation from the tables, p=29.28, i.e.
- Design Pressure =29.3^{psf}

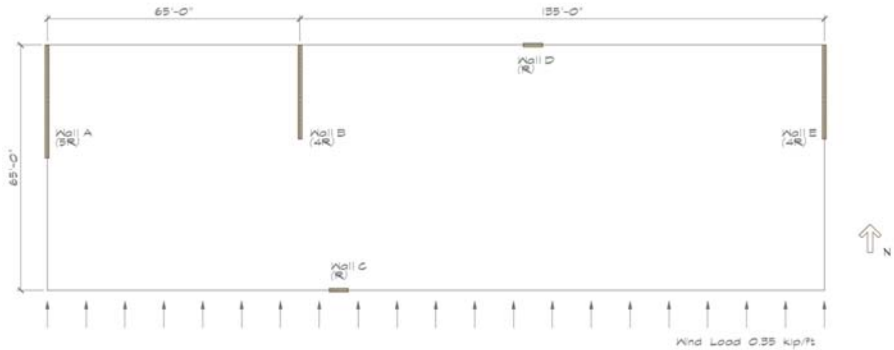
TABLE R301.2(2)
COMPONENT AND CLADDING LOADS FOR A BUILDING WITH A MEAN ROOF HEIGHT OF 30 FEET LOCATED IN EXPOSURE B (psf)[a, b, c, d, e]

	ZONE	EFFECTIVE WIND AREA (feet²)	BASIC WIND SPEED (mph-3-second gust) 85		90		100		105		110		120		125		130		140		145		150		170	
Wall	4	10	13.0	-14.1	14.6	-15.8	18.0	-19.5	19.8	-21.5	21.8	-23.6	25.9	-28.1	28.1	-30.5	30.4	-33.0	35.3	-38.2	37.8	-41.0	40.5	-43.9	52.0	-56.4
	4	20	12.4	-13.5	13.9	-15.1	17.2	-18.7	18.9	-20.6	20.8	-22.6	24.7	-26.9	26.8	-29.2	29.0	-31.6	33.7	-36.7	36.1	-39.3	38.7	-42.1	49.6	-54.1
	4	50	11.6	-12.7	13.0	-14.3	16.1	-17.6	17.8	-19.4	19.5	-21.3	23.2	-25.4	25.2	-27.5	27.2	-29.8	31.6	-34.6	33.9	-37.1	36.2	-39.7	46.6	-51.0
	4	100	11.1	-12.2	12.4	-13.6	15.3	-16.8	16.9	-18.5	18.5	-20.4	22.0	-24.2	23.9	-26.3	25.9	-28.4	30.0	-33.0	32.2	-35.4	34.4	-37.8	44.2	-48.6
	5	10	13.0	-17.4	14.6	-19.5	18.0	-24.1	19.8	-26.0	21.8	-29.1	25.9	-34.7	28.1	-37.6	30.4	-40.7	35.3	-47.2	37.8	-50.6	40.5	-54.2	52.0	-69.6
	5	20	12.4	-16.2	13.9	-18.2	17.2	-22.5	18.9	-24.8	20.8	-27.2	24.7	-32.4	26.8	-35.1	29.0	-38.0	33.7	-44.0	36.1	-47.2	38.7	-50.5	49.6	-64.9
	5	50	11.6	-14.7	13.0	-16.5	16.1	-20.3	17.8	-22.4	19.5	-24.6	23.2	-29.3	25.2	-31.8	27.2	-34.3	31.6	-39.8	33.9	-42.7	36.2	-45.7	46.6	-58.7
	5	100	11.1	-13.5	12.4	-15.1	15.3	-18.7	16.9	-20.6	18.5	-22.6	22.0	-26.9	23.9	-29.2	25.9	-31.6	30.0	-36.7	32.2	-39.3	34.4	-42.1	44.2	-54.1

Rigid Diaphragm subject to Wind Loads

A rigid diaphragm structure is subjected to wind loads as indicated in the diagram. The windl loads are transferred to the shear walls. Each of the shear walls carry relative rigidity as indicated in the diagram

•Determine the forces on Shear walls A and E



Shear Walls R values $R_A := 5$ $R_B := 4$ $R_E := 4$ $R_C := 1$ $R_D := 1$

w load $w := .35 \frac{kip}{ft}$ Total Length: $L_{EW} := 200ft$ Length AB: $L_{AB} := 65ft$ Length BE: $L_{BE} := 135ft$ Length N/S: $L_{NS} := 65ft$

Solution:

1) Calculating the moment resultant force:

$V := w \cdot L_{EW}$ $\qquad V = 70 \cdot kip$

2) Calculating the center of rigidity of the system (using the West wall as reference):

$$\chi := \frac{0ft \cdot R_A + L_{AB} \cdot R_B + L_{EW} \cdot R_E}{R_A + R_B + R_E} \qquad \chi = 81.54 \cdot ft$$

$$\upsilon := \frac{L_{NS}}{2} = 32.5\ ft$$ (By observation of symmetry)

3) Calculating the torsional moment:

$$M := V \cdot \left(\frac{L_{EW}}{2} - \chi\right) \qquad M = 1292.31 \cdot k'$$

3) Calculating the polar moment of inertia for the walls that resist the torsional moment:

$$J := R_A \cdot \chi^2 + R_B \cdot (\chi - L_{AB})^2 + R_E \cdot (\chi - L_{EW})^2 + 2\left(\frac{L_{NS}}{2}\right)^2 \qquad J = 92581.73 \cdot ft^2$$

4) Calculating the maximum lateral force resisted by walls A and E (adding the contribution of each of these walls to direct shear V and the resistance due to torsional moment :

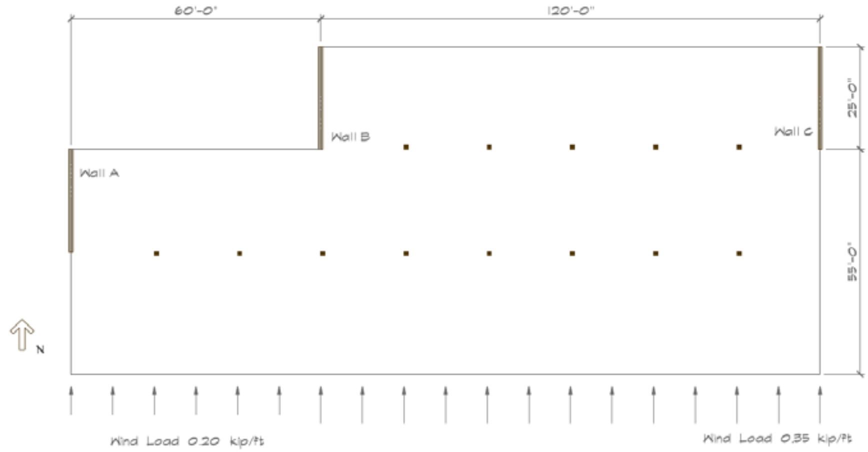
$$V_A := \frac{(R_A \cdot V)}{R_A + R_B + R_E} + \frac{M \cdot R_A \cdot \chi}{J} \qquad V_A = 32.61 \cdot kip$$

$$V_E := \frac{(R_E \cdot V)}{R_A + R_B + R_E} + \frac{M \cdot R_E \cdot (L_{EW} - \chi)}{J} \qquad V_E = 28.15 \cdot kip$$

Flexible Diaphragm subject to Wind Loads

A single story commercial buildingcarries a roof that consists of wood joists, supported by timber beams and sheathed with a nailed and blocked plywood diaphragm. NS lateral forces are indicated in the diagram. The plywood shear wals carry the dimensions shown in the diagram and below

• Determine the axial compression and tension forces in the Shear walls B.
Disregard accidental torsion as it may be required by code.



Wall height $h := 14\text{ft}$

w load $w_1 := .2 \dfrac{\text{kip}}{\text{ft}}$ $w_2 := .33 \dfrac{\text{kip}}{\text{ft}}$ Length AB: $L_{AB} := 60\text{ft}$ Length BC: $L_{BC} := 120\text{ft}$ Length N/S: $L_{NSwall} := 25\text{ft}$

Solution:

1) Calculating the resultant force:

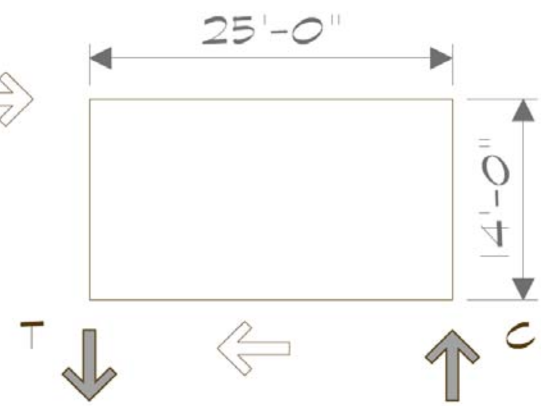
$$V := w_1 \cdot \frac{L_{AB}}{2} + w_2 \cdot \frac{L_{BC}}{2} \qquad V = 25.8 \cdot \text{kip}$$

2) Calculating the Compressive and Tensile reactions:

$$M := V \cdot h \qquad M = 361.2 \cdot \text{k'}$$

$$T := \frac{M}{L_{NSwall}} \qquad T = 14.45 \cdot \text{kip}$$

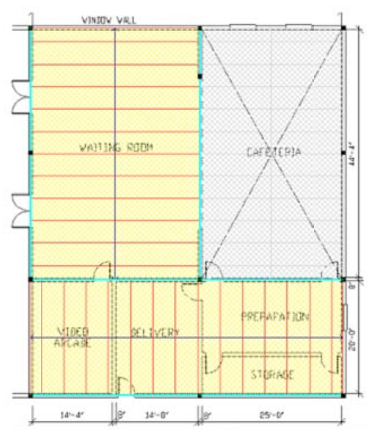
$$C := T = 14.448\,\text{kip}$$



On the Vignette

3.1 Vignette

- There's isn't really much to say...:
  - Just don't use the load bearing walls,
  - Don't span joists more than 35', preferably 30' max
  - Beams are to be used to
    - Carry joists,
    - Under Clerestory
    - To carry upper wall when adjacent to a lower roof
  - Pretty much, consider what the load path is.
    - If there is an interruption of the path that will be wrong,
    - If you have structure supporting nothing, that will also be wrong!



4.0 Vignette

- Let's see, what is this beam doing here?