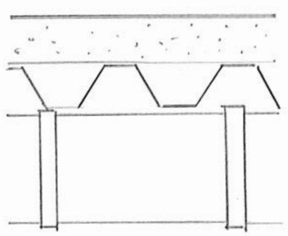
Problem 1.1 Solution

SITUATION: The cross-section of a roof is to be made of the materials shown.

SKETCH:



OBJECTIVE: Determine the unfactored dead load pressure for the roof cross-section

ASSUMPTIONS:
- The mechanical duct allowance adequately represents the self-weight of this 6" duct.

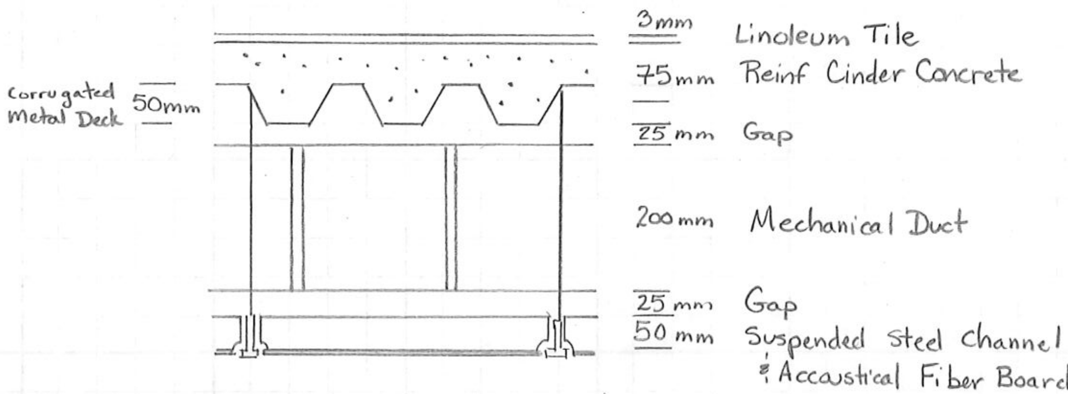
CALCULATIONS:
From Table on Dead Load Pressures of Common Construction Materials,

Single-ply waterproofing	0.7 psf		$30 \, N/m^2$
Polystyrene foam $(0.2 psf/in)(3 in) =$	0.6 psf	$(10 N/m^2/in)(3 in) =$	$30 \, N/m^2$
Metal deck, 18 gage	3.0 psf		$140 \, N/m^2$
Mechanical duct allowance	4.0 psf		$190 \, N/m^2$
	$\Sigma = 8.3$ psf		$\Sigma = 390 \, N/m^2$

$$\boxed{p_D = 8.3 psf = 390 \, N/m^2}$$

SITUATION: The cross-section of a floor is to be made of the materials shown.

SKETCH:



OBJECTIVE: Determine the unfactored dead load pressure for the floor cross-section

ASSUMPTIONS:
- The mechanical duct allowance adequately represents the self-weight of this 200mm duct.

CALCULATIONS:

From Table on Dead Load Pressures of Common Construction Matls and Table on Densities of Common Construction Matls

Linoleum or asphalt tile, 1/4 in (6mm) 50 N/m^2

For 3mm thick, linearly scale ⇒ 25 N/m^2

Reinforced Cinder Concrete $17\,kN/m^3\,(75mm)\left(\frac{m}{1000mm}\right)\left(\frac{1000N}{kN}\right) = 1275\,N/m^2$

Mechanical duct allowance 190 N/m^2

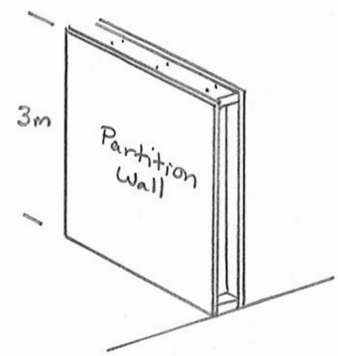
Suspended steel channel system 100 N/m^2

1590 N/m^2

$$\boxed{P_D = 1590\ N/m^2 = 33.25\,psf}$$

SITUATION: A partition wall is to be made of gypsum on both sides of wood studs.

SKETCH:



OBJECTIVE: Determine the unfactored line load generated by the self-weight of the partition wall.

CALCULATIONS:

From Table on Dead Load of Common Construction Materials (components)

Frame partitions - wood or steel studs, $\frac{1}{2}$in gypsum each side $p = 380\ N/m^2$

NOTE: p is force/unit of wall surface

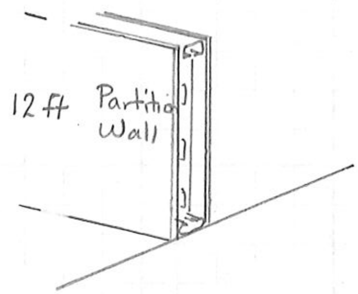
Convert to line load

$$w = p \times height = 380 \tfrac{N}{m^2}(3m)$$
$$= 1140 N/m$$

$$\boxed{w_D^{wall} = 1.140 kN/m}$$

SITUATION: A partition wall is to be made of gypsum on both sides of metal studs

SKETCH:



OBJECTIVE: Determine the unfactored line load generated by the self-weight of the partition wall

CALCULATIONS:

From Table on Dead Load of Common Construction Materials (components):

Frame partitions - Wood or steel studs, $\frac{1}{2}$ in gypsum each side $p = 8$ psf NOTE: p is force/unit wall surface

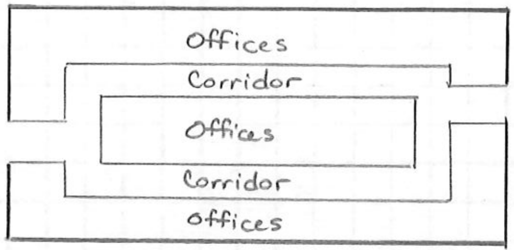
Convert to line load

$w = p \times \text{height} = 8\text{psf}\,(12\text{ft})$
$= 96\text{ plf}$

$$\boxed{w_D^{wall} = 96\text{ plf}}$$

SITUATION: Floor layout for an office building has offices around two corridors. The interior walls, partitions, are likely to be moved over time.

SKETCH:



OBJECTIVES:

a) Determine the minimum unfactored live loads for offices and corridors.

b) Propose a way to deal with corridors moving over time.

SOLUTION:

a) Minimum unfactored live loads:

Offices = 50 psf = 2.40 kN/m^2

Corridors = 80 psf = 3.83 kN/m^2

b) How deal with changing corridor locations:

Option: Design entire floor for highest live load, corridor.
- Might have significant cost implications

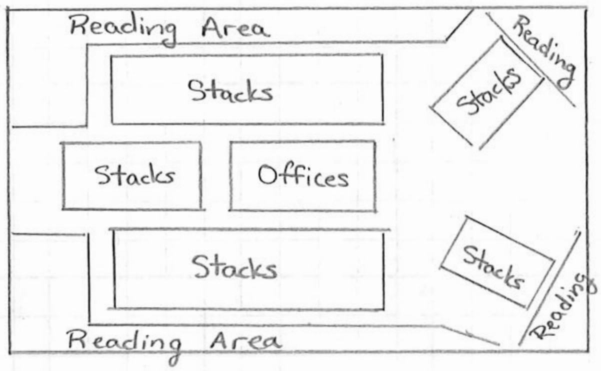
Option: Design all individual members for highest live load, corridor.
Design floor system for a weighted average live load. Since corridors will take less space than offices,

$0.25(80\ psf) + 0.75(50\ psf) = 57.5\ psf$

$0.25\ (2.40 kN/m^2) + 0.75\ (3.83 kN/m^2) = 3.47\ kN/m^2$

SITUATION: Floor layout for a library has offices, stack areas, reading areas, and corridors. The areas will likely be rearranged over time.

SKETCH:



OBJECTIVES:
a) Determine the minimum unfactored live loads for offices, stack areas, reading areas, and corridors.
b) Propose a way to deal with areas rearranging over time.

SOLUTION:
a) Minimum unfactored live loads:

Offices	=	50 psf	=	2.40 kN/m^2
Stacks	=	150 psf	=	7.18 kN/m^2
Reading	=	60 psf	=	2.87 kN/m^2
Corridor	=	80 psf	=	3.83 kN/m^2

b) How deal with possibility of rearranging areas:
Option: Design entire floor for highest live load ... stacks
- might have significant cost implications

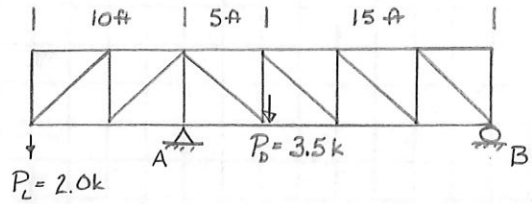
Option: Design all individual members for highest live load - stacks
Design floor system for a weighted average of the two highest loads: stacks and corridors.
For example,

$$0.75\,(150\,psf) + 0.25(80\,psf) = 132\,psf$$

$$0.75\,(7.18\,kN/m^2) + 0.25(3.83\,kN/m^2) = 6.34\,kN/m^2$$

SITUATION: A truss will carry live load over the edge of a roof.

SKETCH:

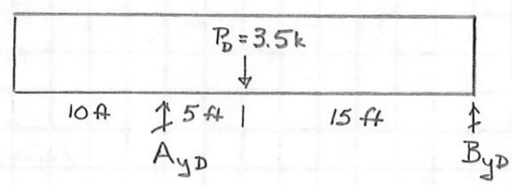


OBJECTIVES:

a) Find the design vertical reactions at A, up and down.
b) Find the design vertical reactions at B, up and down.

CALCULATIONS:

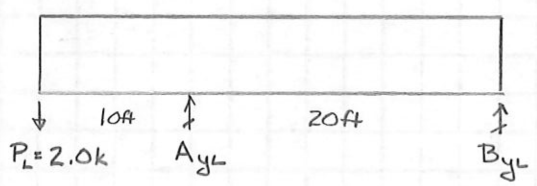
Dead load reactions



$$+\circlearrowleft \Sigma M_B = 0 = A_{yD}(20ft) - 3.5k(15ft)$$
$$A_{yD} = +2.62\ k\ (+\uparrow)$$

$$+\uparrow \Sigma F_y = 0 = 2.62k - 3.5k + B_{yD}$$
$$B_{yD} = +0.88k\ (+\uparrow)$$

Live load reactions



$$+\circlearrowleft \Sigma M_B = 0 = -2.0k(30ft) + A_{yL}(20ft)$$
$$A_{yL} = +3.0\ k\ (+\uparrow)$$

$$+\uparrow \Sigma F_y = 0 = -2.0k + 3.0k + B_{yL}$$
$$B_{yL} = -1.0k\ (+\uparrow)$$

a) Design vertical reactions at A

Both are positive, so use combinations that maximize both

Comb 1: $A_{yu} = 1.4A_{yD} = 1.4(2.62k) = +3.67\ k\ (+\uparrow)$

Comb 2: $A_{yu} = 1.2A_{yD} + 1.6A_{yL} = 1.2(2.62k) + 1.6(3.0k) = +7.94\ k\ (+\uparrow)$

Summary:

$A_{yu}^{+} = 7.94k\ (+\uparrow)$

A_{yu}^{-} = not possible

b) Design vertical reactions at B

Positive:

maximize dead since positive, minimize live since negative

Comb 1: $B_{yu} = 1.4\,B_{yD} = 1.4(0.88k) = +1.23k\,(+\uparrow)$

Comb 2: $B_{yu} = 1.2\,B_{yD} + 0\,B_{yL} = 1.2(0.88k) + 0(-1.0k) = +1.06k\,(+\uparrow)$

Negative:

minimize dead since positive, maximize live since negative

Comb 2: $B_{yu} = 1.2\,B_{yD} + 1.6\,B_{yL} = 1.2(0.88k) + 1.6(-1.0k) = -0.54k\,(+\uparrow)$

But dead might be less than anticipated

Comb 2a: $B_{yu} = 0.9\,B_{yD} + 1.6\,B_{yL} = 0.9(0.88k) + 1.6(-1.0k) = -0.81k\,(+\uparrow)$

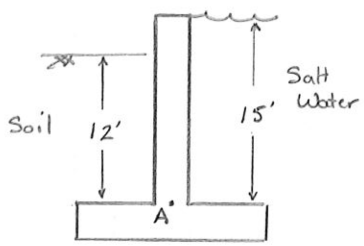
Summary:

$B_{yu}^{+} = 1.23^{k}\,(+\uparrow)$

$B_{yu}^{-} = -0.81^{k}\,(+\uparrow)$

SITUATION: A concrete retaining wall is used for a dolphin tank at a new zoo. The wall extends above the soil a few feet.

SKETCH:

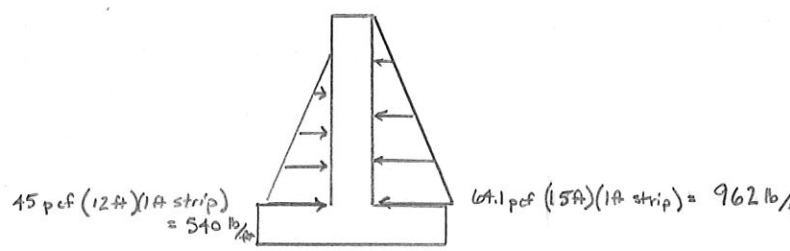


OBJECTIVE: Determine the peak design moments (↻ and ↺) at the base of the wall, A.

ASSUMPTIONS:
- Hydrostatic soil pressure is 45 pcf
- Hydrostatic pressure of salt water is 64.1 pcf

CALCULATIONS:

For a 1ft wide strip,



Fluid load effect, F:

$M_A^F = \frac{1}{2}(962 \text{ lb/ft})(15\text{ft})(5\text{ft}) = 36{,}070 \text{ lb·ft } (\circlearrowleft)$

Soil load effect, H:

$M_A^H = -\frac{1}{2}(540 \text{ lb/ft})(12\text{ft})(4\text{ft}) = -12{,}960 \text{ lb·ft } (\circlearrowleft)$

Design Moment, M_A^u:

Fluid loads are combined with D in load combinations
Soil loads have a factor range of 0.9 to 1.6 if permanent, or 0 to 1.6 if might be removed

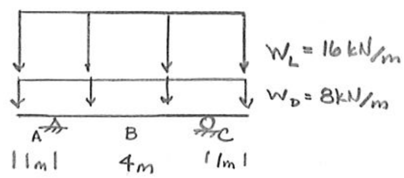
Considerations:
- Tank sometimes emptied, so factor on F can be zero
- Excavation of soil (e.g. for repairs) is possible, but can specify that tank is emptied before excavating. So no need to consider H removed while F acting

Comb 2 modified: $1.2M_A^F + 0.9M_A^H = 1.2(36.1 \text{ k·ft}) + 0.9(-13.0\text{ k ft}) = \boxed{+31.6 \text{ k ft } (\circlearrowleft)}$

Comb 2 modified again: $0M_A^F + 1.6M_A^H = 1.6(-13.0\text{ k ft}) = \boxed{-20.8 \text{ k ft } (\circlearrowleft)}$

SITUATION: A reinforced concrete beam is simply supported but overhangs the supports on both sides.

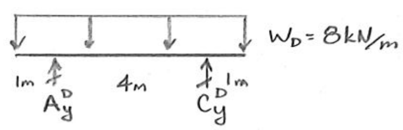
SKETCH:



OBJECTIVE:
Determine the design (maximum factored and combined value) moments, positive and negative, at midspan B.

ASSUMPTION:
The self wt of the concrete is uniform, so ignore the possibility of heavier dead load in some areas.

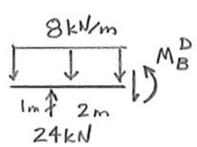
CALCULATIONS:
Dead load:



Find reaction:

$$\circlearrowleft + \Sigma M_C = 0 = A_y^D(4m) - 8\ kN/m(6m)(2m)$$

$$A_y^D = +24\ kN\ (+\uparrow)$$

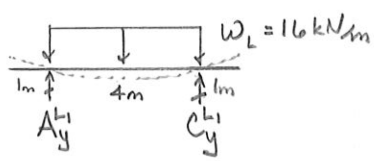


Find moment:

$$\circlearrowleft + \Sigma M_{cut} = 0 = 24\ kN(2m) - 8\ kN/m(3m)(1.5m) - M_B^D$$

$$M_B^D = +12\ kN\cdot m\ (\circlearrowright + \circlearrowleft)$$

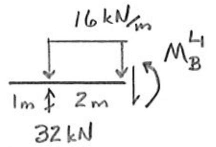
Live load:
Break live load into two patches because each causes different sign on M_B



Find reaction:

$$\circlearrowleft + \Sigma M_C = 0 = A_y^{L1}(4m) - 16\ kN/m(4m)(2m)$$

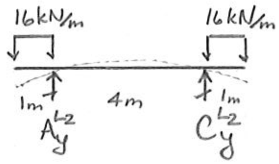
$$A_y^{L1} = +32\ kN\ (+\uparrow)$$



Find moment:

$$\circlearrowleft + \Sigma M_{cut} = 0 = 32\ kN(2m) - 16\ kN/m(2m)(1m) - M_B^{L1}$$

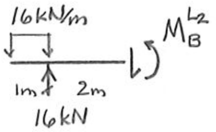
$$M_B^{L1} = +32\ kN\cdot m\ (\circlearrowright + \circlearrowleft)$$



Find reaction:

$$\circlearrowleft \Sigma M_C = 0 = A_y^{L2}(4m) + 16kN/m(1m)(0.5m) - 16kN/m(1m)(4.5m)$$

$$A_y^{L2} = +16\ kN\ (+\uparrow)$$



Find moment:

$$\circlearrowleft \Sigma M_{cut} = 0 = 16kN(2m) - 16kN/m(1m)(2.5m) - M_B^{L2}$$

$$M_B^{L2} = -8\ kN\cdot m\ (\circlearrowright +\circlearrowleft)$$

Combinations:

Comb1: $M_B^u = 1.4 M_B^D = 1.4(12\ kN\cdot m) = \quad = 16.8\ kN\cdot m$

Comb2: $M_B^u = 1.2 M_B^D + 1.6(M_B^{L1} + M_B^{L2}) = 1.2(12\ kN\cdot m) + 1.6(32\ kN\cdot m - 8\ kN\cdot m) = 52.8\ kN\cdot m$

Consider only L that causes +M

Comb 2a: $M_B^u = 1.2 M_B^D + 1.6 M_B^{L1} = 1.2(12\ kN\cdot m) + 1.6(32\ kN\cdot m) \quad = 65.6\ kN\cdot m$

Consider only L that causes −M

Comb 2b: $M_B^u = 1.2 M_B^D + 1.6 M_B^{L2} = 1.2(12\ kN\cdot m) + 1.6(-8\ kN\cdot m) \quad = 1.6\ kN\cdot m$

consider D being light since D causes +M

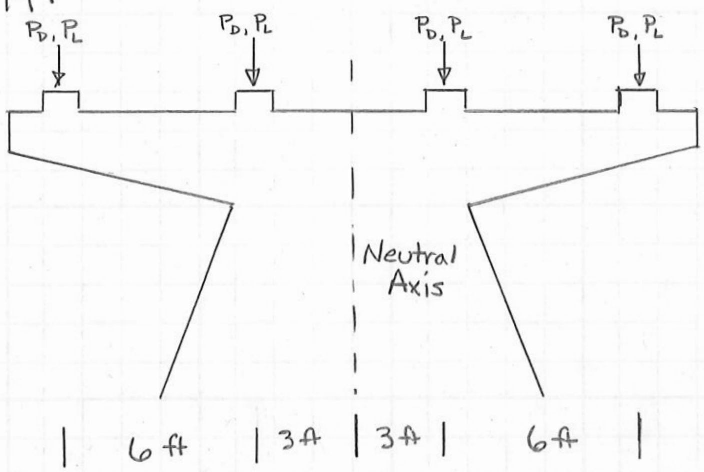
Comb 2c: $M_B^u = 0.9 M_B^D + 1.6 M_B^{L2} = 0.9(12\ kN\cdot m) + 1.6(-8\ kN\cdot m) \quad = -2.0\ kN\cdot m$

Design Moments:

$$\boxed{\begin{array}{l} +M_B^u = 65.6\ kN\cdot m\ (\circlearrowright +\circlearrowleft) \\ -M_B^u = -2.0\ kN\cdot m\ (\circlearrowright +\circlearrowleft) \end{array}}$$

SITUATION: A pier carrys four girders for a highway overpass.

SKETCH:



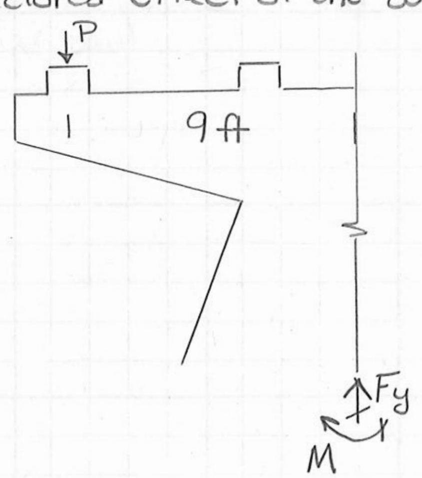
$P_D = 40\text{k}$

$P_L = 20\text{ k}$

OBJECTIVE: Using the ASCE 7 load factors and combinations, determine the design vertical force and design moment.

CALCULATIONS:

Unfactored effect of one outer load



$$+\uparrow \sum F_y = 0 = F_y - P$$

$$F_y = P$$

$$\therefore F_y^D = 40\text{k} \ (+\uparrow)$$

$$F_y^L = 20\text{k} \ (+\uparrow)$$

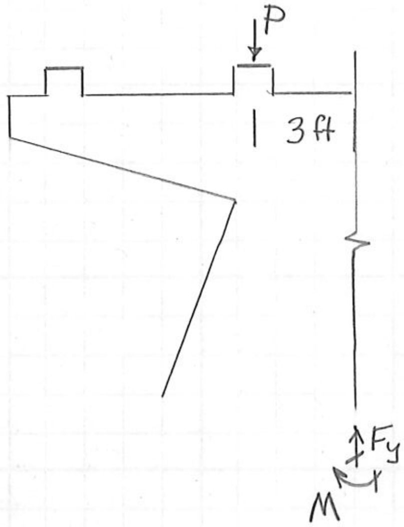
$$\circlearrowleft \sum M_{base} = 0 = M - P(9\text{ ft})$$

$$M = P(9\text{ ft})$$

$$\therefore M^D = 40\text{k}(9\text{ ft}) = 360 \text{ k}\cdot\text{ft} \ (\circlearrowleft)$$

$$M^L = 20\text{k}(9\text{ ft}) = 180 \text{ k}\cdot\text{ft} \ (\circlearrowleft)$$

Unfactored effect of one inner load



$$+\uparrow \Sigma F_y = 0 = F_y - P$$
$$F_y = P$$
$$\therefore F_y^D = 40\text{k} \ (+\uparrow)$$
$$F_y^L = 20\text{k} \ (+\uparrow)$$

$$+\circlearrowleft \Sigma M_{base} = 0 = M - P(3\text{ft})$$
$$M = P(3\text{ft})$$
$$\therefore M^D = 40\text{k}(3\text{ft}) = 120\text{kft} \ (+\circlearrowleft)$$
$$M^L = 20\text{k}(3\text{ft}) = 60\text{kft} \ (+\circlearrowleft)$$

Vertical reaction

All applied loads cause upward reaction, so maximize effects of the applied loads to maximize the upward reaction.

⇒ Apply all live loads
Use largest load factors

Comb 1: $F_y^u = 1.4(\Sigma F_y^D) = 1.4(40\text{k} + 40\text{k} + 40\text{k} + 40\text{k}) = 224\text{k}$

Comb 2: $F_y^u = 1.2(\Sigma F_y^D) + 1.6(\Sigma F_y^L)$
$= 1.2(40\text{k} + 40\text{k} + 40\text{k} + 40\text{k}) + 1.6(20\text{k} + 20\text{k} + 20\text{k} + 20\text{k})$
$= 320\text{k}$

Largest governs

$$\therefore \boxed{F_y^u = 320\text{k}}$$

Moment reaction

Left side loads create clockwise moment; right side loads create caunterclockwise moment.

Since live load can act anywhere, max moment will occur when live load is only on one side

Comb 2: $M^u = 1.2(\Sigma M^D) + 1.6(\Sigma M^L_{left})$
$= 1.2(360\,kft + 120\,kft - 120\,kft - 360\,kft)$
$+ 1.6(180\,kft + 60\,kft)$
$= 0 + 384\,kft$
$= 384\,kft$

Dead load might not be the same for each girder. Worst case would be heavy on one side & light on the other.

Comb 2a: $M^u = 1.2(\Sigma M^D_{left}) + 0.9(\Sigma M^D_{right}) + 1.6(\Sigma M^L_{left})$
$= 1.2(360\,kft + 120\,kft) + 0.9(-120\,kft - 360\,kft)$
$+ 1.6(180\,kft + 60\,kft)$
$= 576\,kft - 432\,kft + 384\,kft$
$= 528\,kft$

Largest governs

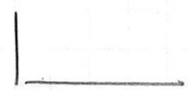
$\therefore \boxed{M^u = 528\,kft}$

Problem 1.11 Solution

SITUATION: Two steel beams are connected at the corner of a cantilevered balcony. The web, not flanges, of one is bolted to a web stiffener of the other

SKETCHES:

ELEVATION

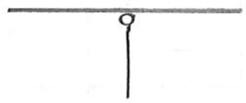


PLAN

IDEALIZATION: Pinned connection

SITUATION: A concrete beam - column connection was cast with notches on two sides. The beam is continuous over the column.

SKETCH:

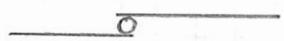


ELEVATION

IDEALIZATION: Pinned connection between continuous beam and column

SITUATION: A pivoting connection is used in a bridge girder to allow thermal expansion and contraction.

SKETCH:



ELEVATION

IDEALIZATION: Roller connection

SITUATION: Two pieces of bridge girder are spliced together

SKETCH:

(most common)

OR

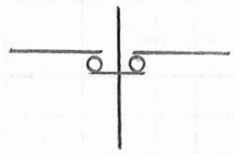
PLAN VIEW

IDEALIZATION: Rigid connection (also called splice in this situation)

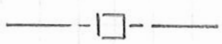
Problem 1.15 Solution

SITUATION: A reinforced concrete column has two corbels. On each corbel sits a girder on an elastomeric pad that allows expansion and contraction of the girder.

SKETCHES:



ELEVATION



PLAN

IDEALIZATION: Roller connection

SITUATION: A rocking element supports a bridge girder.

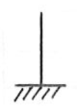
SKETCH:

IDEALIZATION: Roller support

SITUATION: A steel column is attached to a concrete pillar with bolts around the entire steel member perimeter.

SKETCHES:

Concrete as Support



ELEVATION

Concrete as Another Column

ELEVATION

IDEALIZATIONS:

Fixed Support

Rigid Connection

DISCUSSION:

If the concrete pillar is much stiffer than the steel column, the deformation of the concrete is much smaller than the steel member. In that case, we can consider the concrete to be immovable: fixed support.

If we are unsure of the relative stiffness or if we want to consider the deformation of the concrete, consider this to be a connection: rigid.

Problem 1.18 Solution

SITUATION: A railroad brigde girder rests on a support anchored to the concrete abutment.

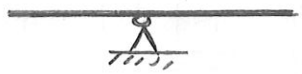
SKETCH:

ELEVATION

IDEALIZATION: Pin support

SITUATION: A short, rounded element supports a bridge girder

SKETCH:

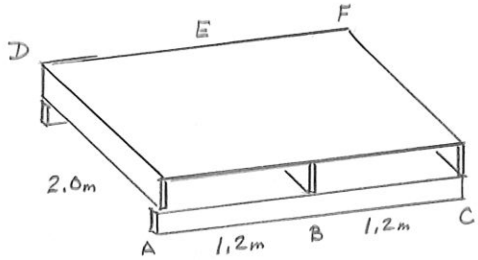


IDEALIZATION: Pin support

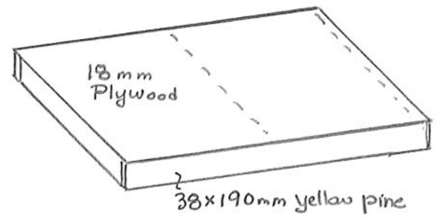
- slightly rounded top allows rotation
- short block anchored to concrete pier restrains horizontal movement

SITUATION: We are designing a platform for a theatrical stage. Two structural configurations are being considered.

SKETCHES:



Option 1



Option 2

OBJECTIVES:

For each structural configuration option,

a) Draw the idealized live loads on beam BE and girder ABC

b) Draw the idealized dead loads on beam BE and girder ABC

ASSUMPTIONS:

a) Floor diaphragm is much more flexible out of plane than the supporting beams & girders

b) Load goes to the nearest supporting member

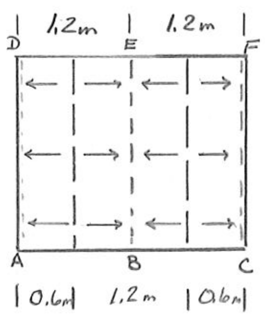
CALCULATIONS:

a) Live Load

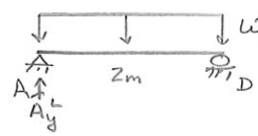
Since this is part of a theatrical stage,

$P_L = 7.18\ \mathrm{kN/m^2}$ (from Live Load Pressure table)

Option 1:



one-way action

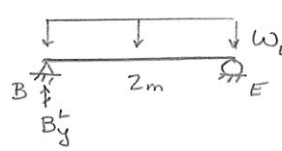


$w_L = 7.18\ \mathrm{kN/m^2}(0.6\mathrm{m}) = 4.31\ \mathrm{kN/m}$

$\circlearrowleft + \sum M_D = 0 = A_y^L(2\mathrm{m}) - 4.31\ \mathrm{kN/m}(2\mathrm{m})(1\mathrm{m})$

$A_y^L = 4.31\ \mathrm{kN}$

NOTE: CF is same & $C_y^L = 4.31\ \mathrm{kN/m}$



$w_L = 7.18\ \mathrm{kN/m^2}(1.2\mathrm{m}) = 8.62\ \mathrm{kN/m}$

$\circlearrowleft + \sum M_E = 0 = B_y^L(2\mathrm{m}) - 8.62\ \mathrm{kN/m}(2\mathrm{m})(1\mathrm{m})$

$B_y^L = 8.62\ \mathrm{kN}$