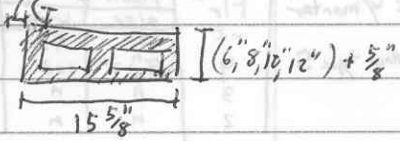
Test 9-1?? C 90 CMU
C 62 Brick

Masonry

CMU weight: light: $w < 105\,pcf$ low strength
medium: $105 < w < 125$ ↓
typ. 110 pcf
Norm: $125\,pcf < w <$ high strength

$\frac{3}{8}$" mortar



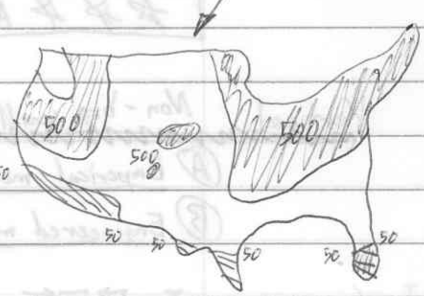
shrinkage: drying shrinkage allow $= 0.065\%$

Brick weathering: C = % water in cold water
B = % water in cold + boiling water
$\frac{C}{B} \geq 1.0$ is bad (No F/T ability)

	normal weathering C	normal weathering C/B	moderate weathering C	moderate weathering C/B	special weathering C	special weathering C/B
C62	–	–	22%	.88	17%	.77
C216			22%	.88	17%	.77

	WI <50	50–500	>500
Vert Earth	MW	SW	SW
Vert none	MW	SW	SW
horizontal Earth	SW	SW	SW
horizontal none	MW	SW	SW

Weather index $= \frac{\#\text{ of inches of rain}}{\text{year}}$ (# of F/T cycles)



IRA = internal rate of absorption $= (W_1 - W)\frac{30\,in^2}{A_{net}} < 30$ grams
W_1 = brick weight @ 1 min in $\frac{1}{8}$" water   W = weight of dry brick

Mortar PC : L : S
M 1 : 0 : 3 } req'd below grade } req'd on exterior
S 1 : ½ : 4.5
N 1 : 1 : 6
O 1 : 2 : 9

Types PC : L : S , mortar cement
Masonry Cement

Grout	PC	Fine ag	corse ag (< 3/8")	lime
Fine grout	1	3	—	0.1
coarse grout	1	3	2	0.1

low lift: grout @ each level
high lift: grout @ each story

Reinf
#4–6 typ. used grade 60 (F_y = 60 ksi)
ladder ~~truss~~ type } crack control
truss type } horizontal flexure

Prism strength
$f'_m = \frac{\text{failure load}}{\text{net area}} \cdot (\text{correction factor})$ 1500 psi typ

Allowable comp. strength
$\frac{A_n}{A_g} \approx 0.4$ table $\frac{P}{A_{net}} = 2000\,psi = \frac{P}{0.4A_g} \Rightarrow \frac{P}{A_g} = 0.4(2000) = 800\,psi$ block
if want 1000 psi block $\frac{P}{A_g} = 1000\,psi = \frac{P}{\left(\frac{A_n}{.4}\right)} \Rightarrow \frac{P}{A_n} = \frac{1000}{.4} = 2500\,psi$

Ⓐ see emperical design for brg walls $h/t \le \frac{20}{18}$

check compressive stresses in block

$F_c = \frac{P}{A} \le F_{c\,allow}$

Area

Table 6 p. A-4

8" hollow = 35.5 plf/ft

8" grouted = 85.8 plf/ft (≈ Table 3 p. A-3)

12" hollow = 48.5 plf/ft tall

Ⓑ Engineered Method wall h = 10'

Axial Bearing wall (trib area) = 10'

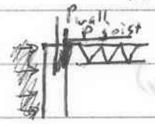
(DL+LL) trib

Table 5.4.2

block w/ mortar	Flr #	plank size	plank weight	Floor DL	DL tot	LL tot	total (plf) load on wall	wall load above	wall weight	total wall load	total wall stress	allowable comp. stress
8" hollow type S	rf	8"	75	15	90	30	1200 plf	0 plf	355 plf	1555	→ $\frac{}{A}$ =	< 65 psi handout
	4	8"+2"	100	20	120	40	1600 plf					
	3	"	"	"	"	"	"					
	2	"	"	"	"	"	"					

trib = 10'

* all mortar is PCL mortar



$\frac{P}{A_{net}} = 2000psi \Rightarrow \frac{P}{A_g} = 0.4(2000) = 800psi$

Block type $\frac{A_n}{A_g} \approx 40\%$

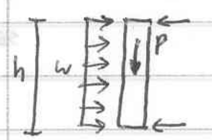
Control Joint: Ⓐ Emperical Method	Ⓑ Engineered method
- spacing: 25' or 1.5h (smaller)	find CCC: $CCC = \frac{0.00065}{2} + 0.00025 + 0.0000055(\Delta T)$
- @ pt. of weakness	CCC ≤ 0.001 | 0.001 < ccc ≤ .0015
- ½ spacing @ corner	$\ell = 25'$, $\ell = 2.5h$ } min | $\ell = 20'$, $\ell = 2.0h$ } min
- provide $As_{min} = 0.25\ in^2/ft_{height}$ as hor. reinf	

*** Alternate: if $As_{min} \ge (0.002 \cdot A_{wall\,net})$ no control joint req'd (usually in seismic D, E, F)*

Vertical Non-brg wall (wind on ext. walls)

Ⓐ Emperical method: see emperical design for non-brg walls $h/t \le \frac{18}{36}$

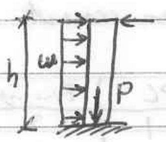
Ⓑ Engineered method: $f_{t\,net} = \frac{M}{S} - \frac{P}{A} \le F_{T\,allow}$ (P = dead load only)



simple

$P = \frac{1}{2}h$ (wall weight psf)

$M = \frac{wh^2}{8}$



proped cantilever (typ. construction, void if not fixed ie: flashing sep.)

$P = h$ (wall weight psf)

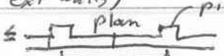
$M = \frac{w\ell^2}{8}$

* for S, A see apendix A table 2

Teck 18-1A

horizontal Non-brg wall (wind on ext walls)

Ⓐ Emperical method: $\frac{\ell}{t} \le$



see emperical design for non-brg walls $\frac{\ell}{t} \le \frac{18}{36}$

watch out for control joints: 1.5(h), 25' } min

Ⓑ Engineered method: $f_t = \frac{M}{S} \le F_{T\,allow}$

section properties off of 1' vertically

$M = \frac{w\ell^2}{8}$

$M = \frac{w\ell^2}{8}$

rein f. @ 8" increments

grade 60 $F_s = 24000$ psi allow

$F_m = \frac{1}{3} f'_m$ allow

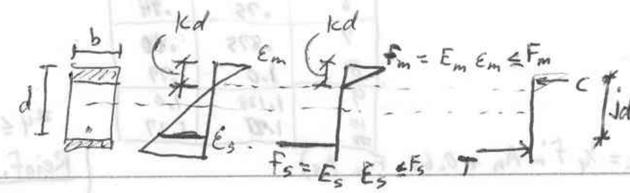
$F_b = \frac{1}{3} f'_m$

$E_m = 900 f'_m$

$n = 21.5$ ← for $f'_m = 1500$ psi

$E_{brick} = 750 f'_m$

$E_s = 29000000$ psi



Non-brg. wall

solid, or assumed solid

flexure

$T = A_s f_s$

$C = \frac{1}{2} b (kd) f_m$

$M_s = A_s f_s jd$ ($M_{max}: f_s = F_s$)

or $M_m = \frac{1}{2} b_j k d^2 f_m$ ($M_{max}: f_m = F_m$)

$k = -(pn) + \sqrt{(pn)^2 + 2(pn)}$

rect. solid $j = 1 - \frac{k}{3}$

$f_s = \frac{M}{A_s jd}$

$f_m = \frac{2M}{b_j k d^2}$

given b, d, A_s, M find j, k

for 8" CMU $d = 3.81"$

$n = \frac{E_s}{E_m} = \frac{29 \times 10^6 psi}{900(1500 psi)} = 21.5$

★ check $kd \leq t_{face}$; if ok, face shell bedded only needed (can assume fully grouted)

$t_{face} = 1.25"$ for 8" CMU

comp b_{eff} { bar spacing; 6 · wall thickness; 72" } min

bar spacing	A_s (in^2/ft)	pn	k	j	kd	M_s	M_m	
# @ 24"					$<t_f$	—		over reinf ($M_m < M_s$)
					$<t_f$		—	under reinf ($M_s < M_m$)
					$>t_f$			?

go to partially grouted

Partially grouted ($kd > t_{face}$)

T-beam flexure (needs either • running bond • hor. bond bm < 48" oc)

long approach (solve for $x_T f_{m_1} = x_c f_{m_1}$)

try a $kd > t_f$; $k = \frac{kd}{d}$

$f_{m_2} = \left(\frac{kd - t_f}{kd}\right) f_{m_1}$ leave as variable

$f_s = n\left(\frac{1-k}{k}\right) f_{m_1}$

$A_f = \frac{t_f \cdot b_{eff}}{b_{eff}} = t_f \cdot \frac{12"}{ft} = \frac{in^2}{ft}$

$b_w = \frac{\text{web width}}{b_{eff}} = \frac{in}{ft}$

$A_s = \frac{A_s}{b_{eff}} = \frac{in^2}{ft}$

$C_f = \frac{1}{2}(f_{m_1} + f_{m_2})\underbrace{(t_f)(\frac{12"}{ft})}_{A_f}$

$C_w = \frac{1}{2} b_w (kd - t_f) f_{m_2}$

$T = A_s f_s$

$C = T$

$C_f + C_w = T$

$(x_c) f_{m_1} = (x_T) f_{m_1}$

$M = A_s f_s jd$

(interpolate between 2 tries to get kd)

if $(x_c) \neq (x_T)$ & if $(x_c) > (x_T)$ kd moves up. (closer to t_f)

Short approach (ignore web contribution (non-brg only))

(Flexure only)

$f_{m_2} = \left(1 - \frac{t_f}{kd}\right) f_{m_1}$

$f_s = n\left(\frac{1-k}{k}\right)(\frac{1}{3} f'_m) \leq 24{,}000$ psi

if true → overreinf $f_{m_1} = \frac{1}{3} f'_m$

if false → $f_{m_1} = \frac{24000(k)}{n(1-k)}$ (under reinf.)

$p' = \frac{A_s}{b_{eff} d}$

$k = \frac{(p'n) + \frac{1}{2}\left(\frac{t_f}{d}\right)^2}{\left(p'n + \left(\frac{t_f}{d}\right)\right)}$

arm length

$jd = d - \left(\frac{2 f_{m_2} + f_{m_1}}{3(f_{m_2} + f_{m_1})}\right) t_f$

$kd \geq t_f$

$M_s = A_s f_s jd$ use → if over reinf. use $f_s = \left(\frac{1-k}{k}\right)(\frac{1}{3} f'_m) n$; if $f_s > 24000$ psi use $f_s = 24000$ psi

✓ over reinf or under reinf.

$M_m = \frac{1}{2}(f_{m_1} + f_{m_2}) A_f jd$ ($\frac{12"}{ft} \cdot t_f$) $f_{m_1} = \frac{1}{3} f'_m$

$M_s = A_s F_s jd$ $F_s = 24000$

$M_m = \frac{1}{2}(f_{m_1} + f_{m_2}) A_f jd$ $f_{m_1} = \frac{1}{3} f'_m$

$F_s = 24000\,psi$ for $f_y = 60\,ksi$

$F_b = \frac{1}{3} F'_m$

$E_m = 900\, F'_m$

$E_s = 29000\,000\,psi$

$n = 750 F'_m$

$$\frac{M}{S} = \frac{My}{I}$$

Bearing walls

unreinforced

tension: $f_t = \frac{M}{S} - \frac{P}{A} \le F_t = 0$ (dead load only)

2.2.3.1 compression: $\frac{f_a}{F_a} + \frac{f_b}{F_b} \le 1.0$ (1.33 for wind)

P+M from chart below

$f_a = \frac{P}{A}$ = psi

$f_b = \frac{M}{S} = \frac{My}{I}$ = psi

$$F_a = \frac{1}{4} f'_m \cdot R$$

$\left(\frac{h}{r}\right) < 99 \quad R = 1 - \left(\frac{h}{r} \cdot \frac{1}{140}\right)^2$

$\left(\frac{h}{r}\right) > 99 \quad$ if $P \le \frac{1}{4} P_e = \frac{1}{4}\left[\frac{\pi^2 E_m I}{h^2}\left(1 - 0.577\frac{e}{r}\right)^3\right]$

then $R = \left(\frac{70}{(h/r)}\right)^2$ otherwise change walls

reinforced CMU Brg walls

2.3.3.2.2 Compressive stress in masonry $\le 0.33 f'_m$ (F_b)

$f_a < F_a$

$e_{min} = 0.1t$

$$M_{max} = (A_s F_s + P)\, jd = A'_s F_s\, jd$$

$$A'_s = A_s + \frac{P}{F_s}$$

non-dim interaction diagram 12-24

$\frac{P}{F_b bt}$, $\frac{Pe}{F_b bt^2}$ } check tension controlled & comp. controlled charts, find P_{max}

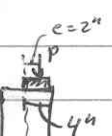
find $A_s = \rho b t$

$g = 0$ ∴ 1 layer of steel

$$p'n = \frac{A'_s}{bd} n$$

$$K = \frac{p'n + \frac{1}{2}\left(\frac{t_f}{d}\right)^2}{p'n + \left(\frac{t_f}{d}\right)}$$

if $kd \le t_f \Rightarrow j = 1 - \frac{k}{3}$



load	mid-ht press P	$M = P(e)$	mid-ht suction P	M
DL_{self}	$45_{psf} \cdot \frac{h}{2} =$	0	same as press	0
DL_{net}	$(DL - uplift)\, trib_{roof}$	$\frac{1}{2}(-P)(e)$	$(DL - uplift) \cdot trib_{roof}$	$\frac{1}{2}(-P)(e)$
DL_{TOT}	$DL\,(trib_{roof})$	$\frac{1}{2}(-P)(e)$	same as press	same as press
LL	$LL\,(trib_{roof})$	$\frac{1}{2}(-P)(e)$	"	"
WL	0	$\frac{(wL)(12''/ft)(h)^2}{8}$	"	$\frac{(WL)(12''/ft)(h^2)}{8}$

	Load combo	Pressure		suction	
$DL_{self} + DL_{TOT}$	D+L				
$DL_{self} + DL_{net}$	D_n+L+W				
	D_n+W				

} plot points

tek 14-19A find $A_s = \frac{in^2}{ft}$: find moment capacities on non-brg charts > M_{max} above (tek 14-19A)

pick $A_s = \frac{in^2}{ft}$

verify $P_{max} = \frac{1}{4} A_n F'_m R > P_{max}$ from above

check cases

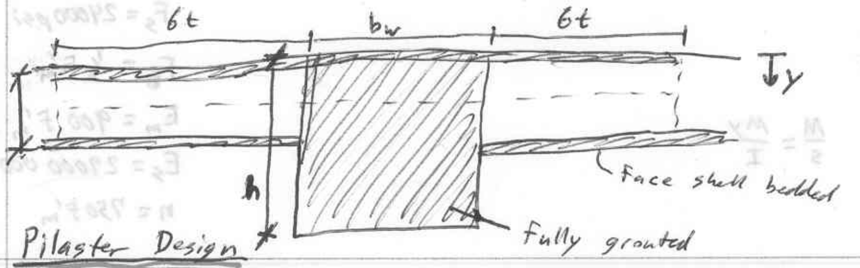
$A'_s = A_s + \frac{P}{F_s} \Rightarrow p'n = \frac{A_s'}{bd} \cdot n$

$f_b = \frac{2 M_{max}}{\left(\frac{12''}{ft}\right) jkd^2} \le F_b = \frac{1}{3} f'_m$

$K = \frac{p'n + \frac{1}{2}\left(\frac{t_f}{d}\right)^2}{\left(p'n + \frac{t_f}{d}\right)} \longrightarrow kd \le t_f \quad j = 1 - \frac{k}{3}$

$f_s = \frac{M_{max}}{A'_s\, jd} \le F_s = 24000\,psi$

text interaction 12-24 $g = 0$ (1 layer of steel) check ρ for tension controlled & comp controlled



$F_s = 24000\,psi$ for $f_y = 60\,ksi$
$F_b = \frac{1}{3} f'_m$
$E_m = 900\ f'_m$
$E_s = 29\,000\,000\ psi$
$n = 21.5$ for $f'_m = 1500\,psi$
$e_{min} = 0.1t$

Pilaster Design

effective flange projection = 6t (running bond)

find A + I

Part	A (in^2)	y (in)	Ay (in^3)	I_o (in^4)	$y-\bar{y}$ (in)	$A(y-\bar{y})^2$ (in^4)
Wall	$\frac{A_{block}}{ft} \cdot 12t$	$\frac{1}{2}t$	$A \cdot y$	$12t\,(I_{o\,face})$ table 14-1	$y-\bar{y}$ (see below)	$A(y-\bar{y})^2$
Pilaster	$b_w \cdot h$	$\frac{1}{2}h$	$A \cdot y$	$\frac{1}{12} b_w h^3$	$y-\bar{y}$	$A(y-\bar{y})^2$
sum	$\Sigma =$		$\Sigma =$	$\Sigma =$		$\Sigma =$

A_{TOT}

$$\bar{y} = \frac{Ay}{A_{TOT}} \qquad I_{TOT} = I_o + A(y-\bar{y})^2$$

$$r = \sqrt{\frac{I_{TOT}}{A_{TOT}}}$$

if unreinf $A_s = 0$

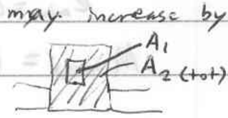
$\frac{h}{r} > 99 \quad P_a = \left(\frac{70}{(h/r)}\right)^2 \left(\frac{1}{4} f'_m A_n + 0.65 F_{sc} A_s\right)$

$\frac{h}{r} < 99 \quad P_a = \left(1 - \left(\frac{(h/r)}{140}\right)^2\right)\left(\frac{1}{4} f'_m A_n + 0.65 F_{sc} A_s\right)$

similiar to brg walls

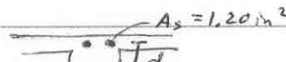
$\frac{1}{4} f'_m$ (allow brg stress)

may increase by $\sqrt{\frac{A_2}{A_1}} \le 2$



reinforced pilaster

① Tension in flange (suction)



$d = h - t_f - 1.5''$ — cover + ½ bar ϕ

$pn = \frac{A_s}{b_w d} \cdot n$

$k = -pn + \sqrt{(pn)^2 + 2pn}$

$j = 1 - \frac{k}{3}$

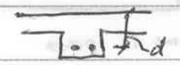
$M_s = A_s F_s\, jd$

$M_m = \frac{1}{2} b\, j\, k\, d^2 F_b$

$\Big\}\ min = M_{max}$ — if $M_s > M_m$ over reinf; else under reinf

same as non-brg wall shortcut 2

② Tension in web (pressure)



$d = h - t_f - 1.5''$

$pn = \frac{A_s}{b_w d}\, n$

$\frac{t_f}{d} =$

$k = \frac{pn + \frac{1}{2}\left(\frac{t_f}{d}\right)^2}{pn + \left(\frac{t_f}{d}\right)} \Rightarrow kd \le t_f$ if true, use eq's above

else ($kd > t_f$); $jd = d - \frac{(2 f_{m2} + f_{m1})}{3(f_{m2} + f_{m1})}$

$f_{m1} = \frac{1}{3} f'_m$

$f_{m2} = \left(\frac{kd - t_f}{kd}\right) f_{m1}$

$M_s = A_s F_s\, jd$

$M_m = \frac{1}{2}(f_{m1} + f_{m2})\underbrace{(12t + b_w)(t_f)}_{A_f}(jd)$

$\Big\}\ min = M_{max}$ — if $M_s > M_m$ over reinf; else under reinf.

interaction diagrams: short cut #1 (reinf. suction cases + web compression) if symmetric reinf. then web in comp is crit.

- pilaster interaction reduces to rect. col. interaction

$A_{tot} =$ ___ $\Big|\ g = \frac{(h - 2(t_f))}{h} =$ ___ $= 0.65 \Rightarrow$ use 0.6 interaction diagram — solve for pn that encompass all of them

short cut #2 straight line approx $\frac{f_a}{F_a} + \frac{f_b}{F_b} \le 1.0$ ok + $f_t \le F_t$ ok

see brg wall on back

$e_{min} = 0.1t$

$A_g = b \cdot h$

$A_n = A_g - A_{holes}$

bar	ϕ	Area
4	.5	.20
5	.625	.31
6	.75	.44
7	.875	.60
8	1.0	.79
9	1.128	1.0
10	1.27	1.27

$F_{sc} = 24000$ psi for $f_y = 60$ksi

$F_b = \frac{1}{3} f'_m$

$E_m = 900 f'_m$

$E_s = 29000000$ psi

$\#4 \le$ bar size $\le \#11$

Columns $(P_c = \frac{1}{4} f'_m A_n + 0.65\ F_{sc} A_s)$

calculate $\left(\frac{h}{r}\right)$

$r = \sqrt{\frac{I}{A}}$ (Tek 14-1A)

h = unsupported height

column max comp. load (axial)

if $(\frac{h}{r} > 99)$ $P_c = \left(\frac{70}{(\frac{h}{r})}\right)^2 \left(\frac{1}{4} f'_m A_n + 0.65\ F_{sc} A_s\right) \le \frac{1}{3} f'_m$ (R; masonry, steel)

if $(\frac{h}{r} < 99)$ $P_c = \left(1 - \left(\frac{h}{r(140)}\right)^2\right)\left(\frac{1}{4} f'_m A_n + 0.65\ F_{sc} A_s\right) \le \frac{1}{3} f'_m$

min ecc $= 0.1t$ $P_c = R\left(\frac{1}{4} f'_m A_n + 0.65\ F_{sc} A_s\right)$

Reinf. stl. (8" modules)

$p_{min} = \frac{A_s}{A_g} = 0.0025 < p = \frac{A_s}{A_g} < 0.04$

$p_{max} = 0.0400$

ties (min size = $\frac{1}{4}$")

spacing min: $16\ d_{bar}$; $48\ (d_{tie})$; least col. dimension

design trial size $A_n = \frac{P_c}{\frac{1}{4} f'_m} = \_\_\ in^2 \overset{\sqrt{}}{=} (\_\_\ in)^2$ ← col. dimensions for a square column (round to 8")

$A_{s_{req}} = \frac{\left(\frac{P_c}{R} - \frac{1}{4} f'_m A_n\right)}{0.65\ F_{sc}} = \_\_\ in^2$ to get bar sizes divide $A_{s_{req}}$ up $A_{size} \ge \frac{A_{s\,req}}{\#cores}$

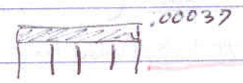
ie 4

$0.0025 < p = \frac{A_s}{A_g} < 0.04$; if $p > .04$ increase col. size ; if $p < .04$ increase steel size

interaction diagram

case 1: $M = 0$

$P_c = F_b(A_g - A_{sc}) + F_b(A_{sc}) \cdot n$

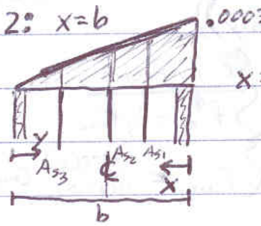


For tables cases 2-5:

element	Area involved	strain	Avg. stress	Force	ecc from ℄	Moment
Masonry	$x \cdot b$	$\frac{(\frac{1}{3} f'_m)}{E_m} = .00037$ typ; E_m = 900 f'm	$\cdot$strain$\cdot E_m$	$A_m \cdot$ stress	$\frac{2}{3}x - \frac{1}{2}t$ (changes)	$M_m = P_m \cdot e_m$
A_{s1}	3bars(A_{bar})	similar	$E_s \cdot$strain (t or c)	$A_{s1} \cdot$ stress	9"	$M_1 = P_1 \cdot e_1$
A_{s2}	2bars(A_{bar})	triangles	t = − stress	$A_{s2} \cdot$ stress	0	$M_2 = P_2 \cdot e_2$
A_{s3}	3bars(A_{bar})	"	c = + stress	$A_{s3} \cdot$ stress	−9"	$M_3 = P_3 \cdot e_3$

$\Sigma: P =$ $\Sigma: M =$

case 2: $x = b$

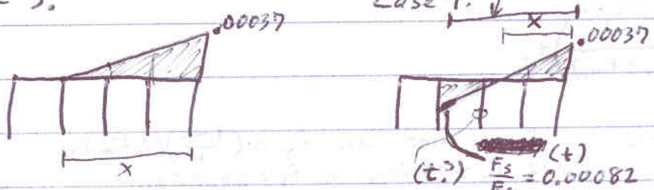


$x = b$ $\epsilon_{s3} = .00037\left(\frac{y}{x}\right)$ ← = b (pt where triangle hits 0)

$x = \frac{37}{(37+82)} \cdot d$

case 3:

case 4:



case 5:

$P = 0$

$M_o = M_s = A_{s_3} F_s\, j d$

A_{s3} = 1 layer only (3 bars this layer)

d_3 = dist. from comp face

Find $pn = \frac{A_s}{bd} n$

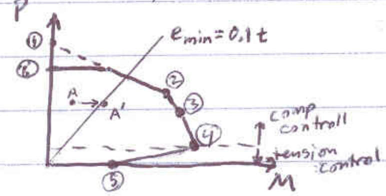
$K = -pn + \sqrt{(pn)^2 + 2pn}$

$j = 1 - \frac{K}{3}$

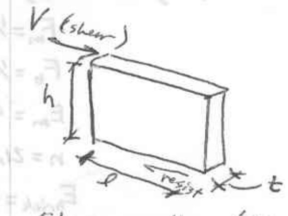
case 6: $P_{max} = P_c = R\left(\frac{1}{4} f'_m A_n + .65\ F_{sc} A_s\right)$

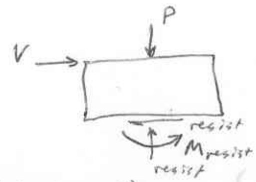
M = any value

text p. 12-12



gr. 60 $F_s = 24000$ psi
$F_m = \frac{1}{3} f'_m$
$F_b = \frac{1}{3} f'_m$
$E_m = 900 f'_m$
$n = 21.5 \longleftrightarrow$ for $f'_m = 1500$ psi
$E_s = 29000000$ psi
$E_{brick} = 750 f'_m$





* watch control joint spacing
$\pm 200'$ shear wall
$= (10)\ 20'$ shear walls

Shear walls (3 types, but start @ 1)

① **Unreinforced**

$M = V \cdot h$

flexure: $f_t = \frac{M}{S_{wall}} - \frac{P}{A_{wall}} = \frac{6Vh}{t\ell^2} - \frac{P}{t\ell} \leq F_{t_{allow}} = 0$ (P = DL + DL self)

shear: $f_v = 1.5\frac{V}{bd} = 1.5\left(\frac{V}{t\ell}\right) \leq F_{v_{allow}} = \min \begin{cases} 120\text{ psi} \\ 1.5\sqrt{f'_m} \\ v + 0.45\left(\frac{P}{A_{wall}}\right) \end{cases}$

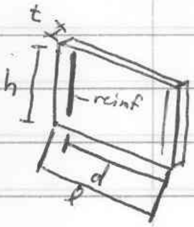
✓ brg: $\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1.0$ (see brg wall section + load combo's)

~~M = V·h~~

$v = 37$ psi (running bond w/ partial/no grout)
$\rightarrow v = 60$ psi (grouted)
$v = 15$ psi (ungrouted stack bond)

* if any item fails move on to next wall type *

② **Masonry reinf. in flexure, unreinf. in shear** (ignore P)



flexure: $d = \ell\left(\frac{12''}{ft}\right) - \frac{1}{2}(8'')$ ← 8" block

$A_{s_{req}} = \frac{M}{F_s\, d\, (j)} = \_\_\_\_ \leq (\ )^{\#}\text{ bars} = A_s = \_\_\_\_$ (j: assume 0.9)

$pn = \frac{A_s}{td}(n)$

$k = -pn + \sqrt{(pn)^2 + 2(pn)}$

$j = 1 - \frac{k}{3}$ (check to see if A_s is ok) → $f_s = \frac{M}{A_s jd} \leq F_s$, $f_b = \frac{2M}{bkjd^2} \leq F_b$

shear: $\frac{M}{Vd} = \_\_\_\_$

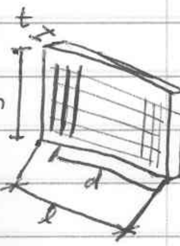
$\frac{M}{V}$ = shear span

$f_v = \frac{V}{b_w d} = \frac{V}{td} \leq F_v$

for $\frac{M}{Vd} < 1.0$ $F_v = \frac{1}{3}\left(4 - \frac{M}{Vd}\right)\sqrt{f'_m} \leq \left(80 - 45\left(\frac{M}{Vd}\right)\right)$ psi

for $\frac{M}{Vd} \geq 1.0$ $F_v = \sqrt{f'_m}$ ($f'_m = 1500 \Rightarrow (39\text{ psi})$) ≤ 35 psi

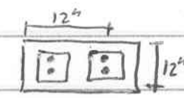
③ **Masonry reinf. in flexure and reinf. in shear** (ignore P)



Flexure: $d = \ell\left(\frac{12''}{ft}\right)$ − Avg. reinf. layer (8" increments for 8" blocks, 6" for 12" blocks)

check block size $f_v = \frac{V}{td} \leq F_v$ (see above)

if fails, • increase f'_m or • increase block size (* recommended) → 12" blocks can do double bars → new d value?



$A_{s_{req}} = \frac{M}{F_s\, d\, (j)} = \_\_\_\_ \leq (\ )^{\#}\text{ bars} = A_s$ (j: assume 0.9)

$pn = \frac{A_s}{td}(n)$

$k = -pn + \sqrt{(pn)^2 + 2(pn)}$

$j = 1 - \frac{k}{3}$ (check to see if A_s is ok) → $f_s = \frac{M}{A_s jd} \leq F_s$, $f_b = \frac{2M}{bkjd^2} \leq F_b$

shear: (flexure d)

$A_{horizontal} = \frac{V(s)}{F_s \cdot d} = \_\_\_\_ \leq (\ )^{\#}\text{ bars} = A_s$ in **bond beams** (spacing $s \leq \begin{cases} 24'' \\ h/2 \end{cases}$ (8" increments); 2 max)

$f_v = \frac{V}{bd} = \frac{V}{td} \leq F_v$

for $\frac{M}{Vd} < 1.0$ $F_v = \frac{1}{2}\left(4 - \frac{M}{Vd}\right)\sqrt{f'_m} \leq \left(120 - 45\left(\frac{M}{Vd}\right)\right)$ psi

for $\frac{M}{Vd} \geq 1.0$ $F_v = 1.5\sqrt{f'_m}$ (56 psi) ≤ 75 psi gr. 60 $F_s = 24000\,psi$

$F_m = \frac{1}{3} f'_m$

$F_b = \frac{1}{3} f'_m$

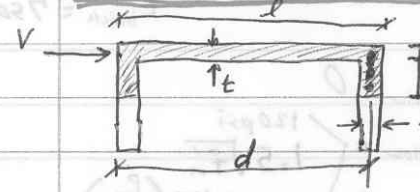
$E_m = 900 f'_m$

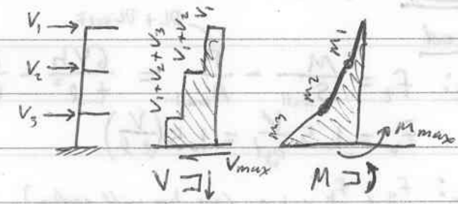
$n = 21.5 \rightarrow$ for $f'_m = 1500$

$E_{brick} = 750 f'_m$

$E_s = 29{,}000{,}000\,psi$

Flanged Shear wall (non-bearing)





design for max V & M_{max}

$$d = l(12''/ft) - \tfrac{1}{2}(t)$$

$$A_{s_{req}} = \frac{M}{F_s d (j)} = \_\_\_\_ \le (\ )^{\#}\text{bars} = A_s$$

j — assume 0.9

$$pn = \frac{A_s}{b_F\, d}$$

$$k = \frac{pn + \frac{1}{2}\left(\frac{t_F}{d}\right)^2}{\left(pn + \left(\frac{t_F}{d}\right)\right)}$$

$kd = \_\_\_\_ \ge t_F$ ∴ ok, it is a T-beam

$$jd = d - \left(\frac{2 f_{m_2} + f_{m_1}}{3(f_{m_2} + f_{m_1})}\right) t_F$$

$f_s = n\left(\frac{1-k}{k}\right)\left(\frac{1}{3} f'_m\right) < 24000\,psi \rightarrow$ if true: $f_{m_1} = \frac{1}{3} f'_m$

$f_{m_1} \longrightarrow$ if False: $f_{m_1} = \dfrac{24000(k)}{n(1-k)}$ (under reinf)

$$f_{m_2} = \left(1 - \frac{t_F}{kd}\right) f_{m_1}$$

$$M_s = A_s F_s (jd) \ge M_{max}$$

$$M_m = \tfrac{1}{2}(f_{m_1} + f_{m_2})(b_F \cdot t_F)(jd) \ge M_{max}$$

shear check: $\frac{M}{Vd} = \ ?$

$$f_v = \frac{V}{t_F\, d} \le F_v$$

for $\frac{M}{Vd} \le 1$ $\quad F_v = \frac{1}{3}\left(4 - \frac{M}{Vd}\right)\sqrt{f'_m} \le \left(80 - 45\frac{M}{Vd}\right) psi$

for $\frac{M}{Vd} \ge 1$ $\quad F_v = \sqrt{f'_m} \quad (39) \le 35\,psi$

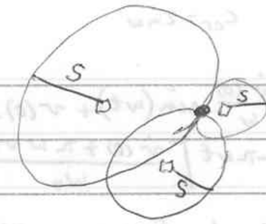
Bar size	diameter	Area
3	0.375"	0.11 in^2
4	0.5"	0.20 in^2
5	0.625"	0.31 in^2
6	0.750"	0.44 in^2
7	0.875"	0.60 in^2
8	1.00"	0.79 in^2
9	1.128"	1.0 in^2
10	1.27"	1.27 in^2
11	1.41"	1.56 in^2
14	1.693"	2.25 in^2
18	2.257"	4.0 in^2

Earthquake

$g = 32.2\ ft/s^2$
$g = 386.4\ in/s^2$

EQ difference
1. Peak Amplitude
2. duration of strong motion
3. Frequency content $= \frac{1}{T}$

(10) waves
- ↔ P : compressional 19000 ft/s
- S : shear waves 10000 "
- R/L : 9000 "



(11) locating the Epicenter $S = \dfrac{t_s - t_p}{\left(\frac{1}{v_s} - \frac{1}{v_p}\right)}$

(12) Magnitude

$M = \log_{10}\left(\frac{A}{A_o}\right)$ $A_o = 0.001\,mm$

A = peak amplitude 100 km from Epicenter

(13) energy release

$E = 10^{(11.8 + 1.5M)}$ in erg $= 10^{-7}$ Joules

(14) length of active Faults

$L = 10^{(1.02M - 5.77)}$

(15) (PGA) peak ground acceleration

$PGA = 15\frac{ft}{s^2} = \frac{15\ ft/s^2}{32.2\ ft/s^2} = 0.47g$ (47% g)

(16) intensity

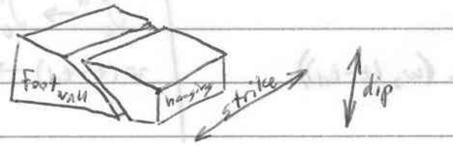
(MMI) modified mercalli intensity $= I \to XII$

MMI	PGA
VI	$0.08g \to 0.15g$
VIII	$0.24g \to 0.45g$
X	$0.6g \to 0.8g$

(17) vertical peak ground acc. (PGA_{vert})

$PGA_{vert} = \frac{1}{3} PGA$

Faults



footwall controls direction

left strike slip

(20) frequency of occurance (#/yr)

$N(\#/yr) = 10^{(a - bM)} = \frac{\#\text{ of EQ of M magnitude}}{yr}$

(21) Probability of occurance $\frac{\#\ M(\text{magnitude})}{y\ (\text{years})}$

$N = cye^{-\frac{M}{\beta}}$

$P = K\Delta$

$\omega = \sqrt{\frac{K}{m}} = \frac{2\pi}{T}$

$T = \frac{2\pi}{\omega} = \frac{1}{f}$

$f = \frac{1}{T} = \frac{\omega}{2\pi}$

$\eta = \frac{c}{C_{cr}} = \frac{\delta}{2\pi}$

free vibration

$\eta = \frac{c}{C_{cr}} = \frac{\delta}{2\pi} = \frac{\ln\left(\frac{v_1}{v_2}\right)}{2\pi}$ $C_{cr} = 2m\omega$

$B = \frac{\omega}{\omega_n}$

$\eta = \frac{P_o \sin\theta}{2B\rho k}$

$C_{cr} = 2m\omega$

springs parallel: $K_{eq} = k_1 + k_2$

springs series: $\frac{1}{K_T} = \frac{1}{k_1} + \frac{1}{k_2}$

Column CASE	Δ	K	beam case	Δ	K
P, h, IΔ	$\frac{Ph}{AE}$	$\frac{AE}{h}$	P	$\frac{Pl^3}{48EI}$	$\frac{48EI}{l^3}$
P, h, θ, flex.	$\frac{Ph^3}{3EI}$, $M_m = Pl$	$\frac{3EI}{h^3}$	w	$\frac{5wl^4}{384EI}$	$\frac{384EI}{5l^3}$
P, trans, rigid diaphragm	$\frac{Ph^3}{12EI}$, $M_m = \frac{Pl}{2}$	$\frac{12EI}{h^3}$	P	$\frac{Pl^3}{192EI}$	$\frac{192EI}{l^3}$
w	$\frac{wh^4}{8EI}$	$\frac{8EI}{h^3}$	w	$\frac{wl^4}{384EI}$, $M = \frac{wl^2}{12}$	$\frac{384EI}{l^3}$

Brace case	Δ	K
l, P, θ	$\frac{Pl}{AE\cos^2\theta}$	$\frac{AE}{l}\cos^2\theta$

smaller mass → higher frequency
larger stiffness → higher frequency

eq. of motion $m\ddot{v}(t) + c\dot{v}(t) + k v(t) = P(t)$ $\begin{cases} P(t) = 0 & \text{free vibration} \\ P(t) = P_0 \sin(\omega t) & \text{harmonic} \end{cases}$

$c_{cr} = 2m\omega$

SDOF

Undamped $v(t) = \frac{\dot{v}(0)}{\omega}\sin(\omega t) + v(0)\cos(\omega t)$

damped (circular) $v(t) = e^{-\xi\omega t}\left[\frac{\dot{v}(0) + \xi\omega v(0)}{\omega_D}\cdot\sin(\omega_D t) + v(0)\cos(\omega_D t)\right]$

damped circular frequency $\omega_D = \omega\sqrt{1-\xi^2}$ see notes

Harmonic - SDOF

undamped $v(t) = \underbrace{\left[\frac{\dot{v}(0)}{\omega_n} - \frac{P_0}{K}\cdot\frac{\beta}{1-\beta^2}\right]\sin(\omega_n t) + v(0)\cos(\omega_n t)}_{v_c(t)\ \text{(free vibration) (transient)}} + \underbrace{\frac{P_0}{K}\cdot\frac{1}{1-\beta^2}\sin(\omega t)}_{v_p(t)\ \text{(steady state)}}$

$\beta = \frac{\omega}{\omega_n}$ frequency ratio

$v_{st} = \frac{P_0}{K}$ static displacement $R(t) = \frac{v(t)}{v_{st}} = \frac{1}{(1-\beta^2)}\left(\sin(\omega t) - \beta\sin(\omega_n t)\right)$ response ratio

damped $v(t) = \underbrace{e^{-\xi\omega_n t}\left[\left(\frac{\dot{v}(0)}{\omega_n} + v(0)\xi\right)\sin(\omega_n t) + v(0)\cos(\omega_n t)\right]}_{v_c(t)} + \underbrace{\frac{P_0}{k}\left[\frac{1}{(1-\beta^2)^2 + (2\xi\beta)^2}\right]\left[(1-\beta^2)\sin(\omega t) - 2\xi\beta\cos(\omega t)\right]}_{v_p(t)}$

$v_p(t) = \rho\sin(\omega t - \theta)$

$\xi = \frac{P_0\sin\theta}{2\beta\rho k}$ $\rho = \frac{P_0}{k}\frac{1}{\sqrt{(1-\beta^2)^2 + (2\xi\beta)^2}}$ $DA = \frac{\rho}{(P_0/k)} = \frac{1}{\sqrt{(1-\beta^2)^2 + (2\xi\beta)^2}}$

@ resonance $\beta = 1$ $DA = \frac{1}{2\xi}$

Rect. Impulse $(t_d < T_n)$ $(v(0) = 0)$

undamped $v(t - t_d) = \frac{1}{m\omega}\left[\int_0^{t_d} p(t)\,dt\right]\sin(\omega(t - t_d))$ w/ $v(0) = 0$

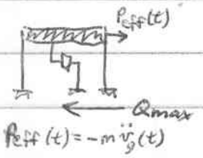
assumes $v(0) = 0$

damped $v(t - t_d) = \frac{1}{m\omega_D}\left[\int_0^{t_d} p(t)\,dt\right]e^{-\xi\omega_D(t - t_d)}\sin(\omega_D(t - t_d))$

$\omega_D = \omega\sqrt{1-\xi^2}$

total response
$t_d \to \tau$
$\int_0^{t_d} \to \int_0^{t}$
$v(t - t_d) \to v(t)$

Ground displacement motion



$P_{eff}(t) = -m\ddot{v}_g(t)$

$D = S_d$ = spectral displacement $= v_{max}(t)$ $S_d = Q_{max}\left(\frac{1}{m\omega_n^2}\right)$

$V = S_{pv}$ = spectral pseudo-velocity $= V_{max}(t)$ $S_{pv} = \omega_n S_d$

$A = S_{pa}$ = spectral pseudo-acceleration $= \ddot{v}_{tot\,max}(t)$ $S_{pa} = \omega_n^2 S_d$ (units are $\underline{X}\cdot g$)

$Q_{max} = W\left(\frac{A}{g}\right)$

note: A is in units of g, so the g's cancel out

watch charts (ie. if chart is $\ddot{v} = 1g$ & have $\ddot{v} = .5g$ interpolated values are $A_1 = \frac{1}{2}A_{Table}$)

Diaphram	Distribution
rigid	stiffness
Flexible	trib. area

Load combinations

1.0L if LL > 100psf

5. $1.2D + 1.0E + 0.2S + 0.5L$ $\qquad E = \rho Q_E + 0.2 S_{DS} \cdot D$ (only w/ load combo 5)

7. $0.9D + 1.0E + 1.6H$ $\qquad E = \rho Q_E - 0.2 S_{DS} \cdot D$ (only w/ load combo 7)

ground pressure

redundancy ($1.0 \le \rho_x \le 1.5$)

for SDC = A, B, or C

$\rho = 1.0$

for SDC = D, E, or F

$\rho = \rho_{x\,max}$ out of all floor levels

$$1.0 \le \rho_x = 2 - \frac{20}{r_{max_x}\sqrt{A_x}} \le 1.5$$

A_x = floor area in ft^2

Q_E = member force

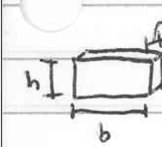
reliability factor

$r_{max_x} = \dfrac{(V_{element})_{max}}{V_{story}}$	
Braced frame: $r_{max_x} = \dfrac{(V_{brace})_{max}}{V_{story}}$	$V_{brace} = (\text{axial load})\left(\frac{w}{\ell}\right)$
*Moment frame: $r_{max_x} = \dfrac{(V_{col_1} + V_{col_2})_{max}}{V_{story}}$	adjacent cols *use $0.7V_{col}$ if columns are common to 2 bays.
Shear wall: $r_{max_x} = \dfrac{\left((V_{wall})\left(\frac{10}{\ell_{wall}}\right)\right)_{max}}{V_{story}}$	ℓ_{wall} = wall length (ft) $\frac{10}{\ell_w} \le 1.0$

$$\Delta = \frac{Ph^3}{12EI} + 1.2\frac{Ph}{AG}$$ conc. $G = 0.4E$

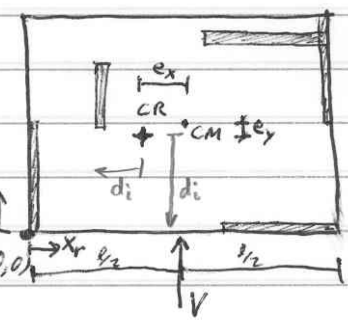
Shear in building due to torsion & direct load (EQ loading)

rigidity $R_{fix} = \frac{k}{E} = \frac{1}{\left(\frac{h}{b}\right)^3 + 3\left(\frac{h}{b}\right)}$ w/ 4 = $R_{cant.}$



$A = bt$

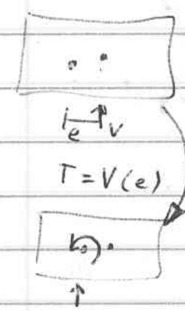
$I = \frac{tb^3}{12}$



center of mass – @ center of building

center of rigidity: $X_r = \dfrac{\sum R_{iy} X_i}{\sum R_{iy}}$

$Y_r = \dfrac{\sum R_{ix} Y_i}{\sum R_{ix}}$



$T = V(e)$

$e_{min} = \pm 0.05\ell$ $\qquad e_x = X_{cm} - X_{CR}$

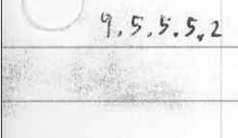
$e_y = Y_{cm} - Y_{CR}$

torsional moment of inertia $\quad J = \sum(R_i (d_i)^2)$ $\qquad T = V(e \pm e_{min})$

Direct shear: $V_i = \dfrac{R_i}{\sum R_i} V_{story}$

Torsional shear: $V_{T,i} = \dfrac{T_i \cdot d_i \cdot R_i}{J}$ (check if you add or subtract e_{min})

$T_i = V(e \pm e_{min})$



$T_{resist}\uparrow \downarrow V_{resist} \Rightarrow$ subtract e_{min}

$T_{resist}\downarrow \downarrow V_{resist} \Rightarrow$ add e_{min}

Total $V_i = V_{i,direct} \pm V_{i,torsional}$

table 9.5.2.3.2 check plan irregularity $V_{TOT,i}$ R δ $\qquad V = R\delta \Rightarrow \delta = \frac{V}{R}$

item 1a: if $\Delta_{max} \ge 1.2\left(\frac{\Delta_1 + \Delta_2}{2}\right)$ then irregular, if $> 1.4\left(\frac{\Delta_1 + \Delta_2}{2}\right)$ then extreme irregularity

9.5.5.5.2 if irregularity: Amplification Factor (SDC C, D, E, F)

$$1.0 \le A_x = \left(\frac{\delta_{max}}{1.2(\delta_{avg})}\right)^2 \le 3.0$$ (ignore if light framed)

$e_{min_2} = A_x \cdot e_{min}$

$V_{T,i} = \dfrac{V(e \pm e_{min_2})\, d_i\, R_i}{J}$ $\Rightarrow$ new higher $V_{TOT,i} = V_{dir,i} + V_{T,i}$

Component Design

$$F_p = \frac{0.4\, a_p\, S_{DS}\, W_p}{\left(R_p/I_p\right)}\left(1 + 2\left(\frac{z}{h}\right)\right) \qquad 0.3\, S_{DS}\, I_p\, W_p \le F_p \le 1.6\, S_{DS}\, I_p\, W_p$$

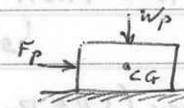
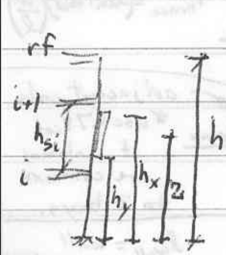


table 9.6.2.2 — a_p = component amplification factor $1.0 \le a_p \le 2.5$

R_p = component response modification factor

z = avg height in structure of attachment point of component

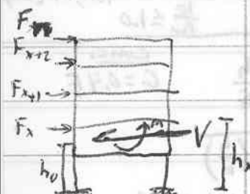
h = avg roof height W_p = weight of component I_p = importance factor



relative displacement requirement

$$D_p = \delta_{xA} - \delta_{yA} \le (h_x - h_y)\frac{\Delta_{a\,allow}}{h_{si}}$$

δ @ level x determined by elastic analysis & multiplied by C_d



Overturning moment

$$M_0 = \sum^n \left(F_x (h_x - h_0)\right)$$

$$V_0 = \sum^n F_x$$

story drift

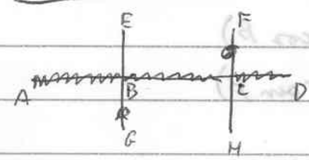
table 9.5.2.8

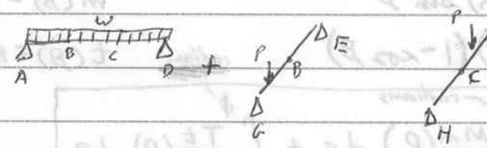
$$\Delta_x = \delta_{x+1} - \delta_x \le \Delta_{allow} \approx \frac{h}{400}$$

$$\delta_x = \frac{C_d\, \delta_{xe}}{I}$$ elastic analysis

AE 439

Grid structure (consist. def.)



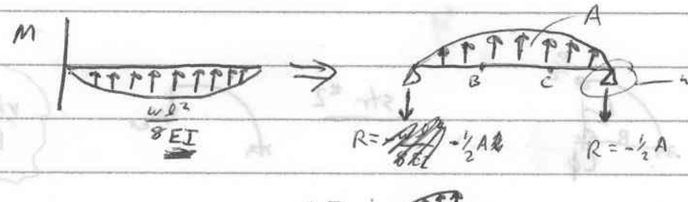


Real	Conj.
pin roller	Pin
Fixed	Free
Free	Fixed

$\Delta c_1 + \Delta c_2 = 0$

$\Delta b_1 + \Delta b_2 = 0$

use conjeguate beam



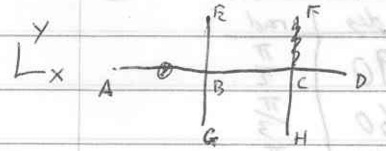
watch end conditions

$R = -\frac{1}{2} A$ $\quad R = -\frac{1}{2} A$

$\Delta_B = \frac{M_B}{EI} = \Delta_B$ (Find internal moment @ pt. $= \Delta_B$)

Grids (by slope deflection)

use modified eq. if possible



(no torsion)

$$Y\begin{cases} M_{BA} = \\ M_{BC} = \\ M_{CB} = \\ M_{CD} = \end{cases} \qquad X\begin{cases} M_{BE} \\ M_{BG} \\ M_{CF} \\ M_{CH} \end{cases} \qquad \text{pt. } B \begin{cases} V_{BA} \\ V_{BC} \\ V_{BE} \\ V_{BF} \end{cases} \qquad \text{pt. } C \begin{cases} V_{CB} \\ V_{CD} \\ V_{CG} \\ V_{CH} \end{cases}$$

use L for each member

each member is independent

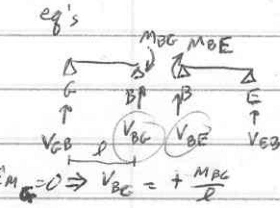
equalib. eq. (Known)

$$\begin{cases} \Sigma M_{BX} = 0 = M_{BE} + M_{BG} \\ \Sigma M_{BY} = 0 = M_{BA} + M_{BC} \\ \Sigma M_{CX} = 0 = M_{CB} + M_{CD} \\ \Sigma M_{CY} = 0 = M_{CH} + M_{CF} \\ \Sigma F_{BZ} = 0 = V_{BA} + V_{BC} + V_{BE} + V_{BG} \\ \Sigma F_{CZ} = 0 = V_{CB} + V_{CH} + V_{CD} + V_{CF} \end{cases}$$

slope deflection eq's

no torsion

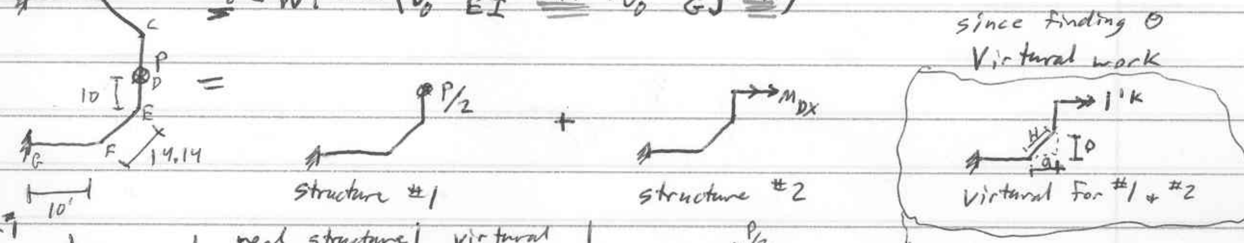
Joint reaction eq's



$\Sigma M_G = 0 \Rightarrow V_{BC} = + \frac{M_{BG}}{\ell}$

Awning beams (use symmetry) $\boxed{\theta_1 + \theta_2 = 0}$

$$\theta_i = \Sigma W_i = \Sigma\left(\int_0^x \frac{M m}{EI}\,dx + \int_0^x \frac{T t}{GJ}\,dx\right)$$

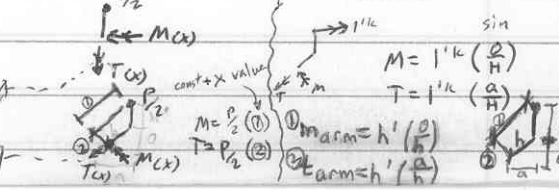


since finding θ Virtural work

Virtural For #1 + #2

str #1

memb	X	real structure M	real structure T	virtural m	virtural t
DE	0→10'				
EF	0→14.14'				
FG	0→10'				



const + x value

$M = \frac{P}{2}(①)$

$T = \frac{P}{2}(②)$

① $m_{arm} = h'\left(\frac{a}{H}\right)$

② $t_{arm} = h'\left(\frac{a}{h}\right)$

$M = 1'^k\left(\frac{a}{H}\right)$ sin

$T = 1'^k\left(\frac{a}{H}\right)$

* M @ pt $E = M_{str\,1} + M_{str\,2}$ ← real components

(Radians)

Curved beam in plan (const radius)

pt load

$$\begin{cases} M(\beta) = -P(R)\sin\beta \\ T(\beta) = P(R)(1-\cos\beta) \end{cases} \qquad \begin{aligned} m(\beta) &= \cancel{M_{EX}}\,1^{\prime K}(\cos\beta) \\ t(\beta) &= \cancel{M_{EX}}\,1^{\prime K}(-\sin\beta) \end{aligned}$$

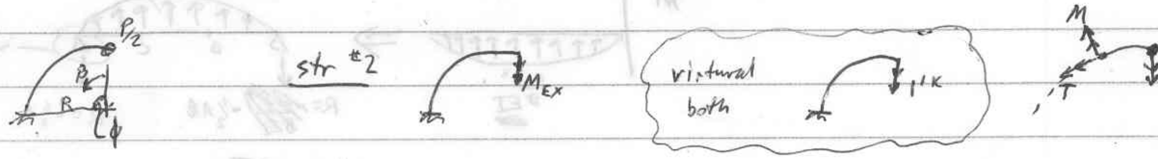
if M_{EX} just multiply virtual by M_{EX}

$$\boxed{\theta_c = w_i = \int_0^{\phi \leftarrow \text{radians}} \frac{Mm}{EI}\underline{(R)}\,\underline{d\beta} + \int_0^{\phi} \frac{Tt}{GJ}\underline{(R)}\,\underline{d\beta}}$$

radians

real

str #1 $P/2$ β R ϕ str #2 M_{EX} virtual both $1^{\prime K}$ M T



Calculus

$\int_0^{\phi} \sin\beta\, d\beta = -\cos\phi + 1$	$\int_0^{\phi} \sin^2\beta\, d\beta = \frac{1}{2}(\phi - \sin\phi\cos\phi)$
$\int_0^{\phi} \cos\beta\, d\beta = \sin\phi$	$\int_0^{\phi} \cos^2\beta\, d\beta = \frac{1}{2}(\phi + \sin\phi\cos\phi)$
$\int_0^{\phi} \sin\beta\cos\beta\, d\beta = \frac{\sin^2\phi}{2}$	

	member	β	real M	real T	virtual m	virtual t
Str #1	EA	$0 \to \frac{\pi}{2}$				

deg	rad
90	$\frac{\pi}{2}$
60	$\frac{\pi}{3}$
45	$\frac{\pi}{4}$
30	$\frac{\pi}{6}$

$\beta = \angle$ @ pt B in radians

@ pt B $\quad M_B = M_{str\#1} + M_{str\#2}$ (real, real) $\quad$ ($M_{@\text{ a point}}$ = sum of the real components at that pt.)

$\quad T_B = T_{str\#1} + T_{str\#2}$

Real	Conjugate
Pin	Pin
Fixed	Free
Free	Fixed
Roller	Pin

Virtural Work $\left[\curvearrowright + \quad \downarrow^{+} \right]$

member	actual F_i	virtual f_i	in l	in^2 A	ksi E	f_iF_il/AE
1						
2						
					$\Sigma=$	

$1^k = \Delta$

axial: $1^k\Delta = \Sigma^{in} \dfrac{f_i F_i l_i}{A_i E_i}$

Flexural: $1^K\Delta = \int_0^l \dfrac{m_i M_i}{E_i I_i} dx$ Deep beams: $1^K\Delta = \int_0^l \dfrac{K_i v_i V_i}{A_i G_i} dx$

$G_i = \dfrac{E}{2(1-\nu)}$ $\nu =$ 0.15 for conc, 0.29 for steel

$K_i =$ shape factor $K_i =$ 1.2 for rectangles, 1.0 for W-sections w/ $A_i = A_w$

Torsional: $1^K\Delta = \int_0^l \dfrac{t_i T_i}{K_i G_i}$ note: torsion is constant throughout member; E or G can use super position

Slope Deflection $\curvearrowright +$

see handout

Find known conditions @ joint 3 $\Sigma M = 0 = M_{34} + M_{31}$

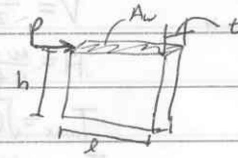
Lateral Force resist (add Δ not k's)

$ww = \left(\dfrac{.8}{.8+.5}\right) p^{lf}$

shear walls $\Delta = \dfrac{Ph^3}{3EI} + \dfrac{2.78\,Ph}{t\,l\,E}$

$F = K\Delta$

Flexure $\left(\dfrac{h}{l} > 3\right)$ shear $\left(\dfrac{h}{l} < 0.3\right)$



$\dfrac{h}{l} < 0.3$ short wall

$\dfrac{h}{l} > 3$ tall wall

$I = \frac{1}{12} t l^3$

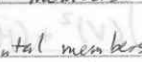
tall walls $\left(\frac{h}{l} > 3\right)$	if const t+E	if all tall/short	$.3 < \frac{h}{l} < 3$ (Both)	if (t + E + h const)
$K = \dfrac{3E}{h^3}\left(\dfrac{t l^3}{12}\right)$	$K = \dfrac{l^3}{4h^3}$	$K = l^3$	$K = \dfrac{1}{\frac{Ph^3}{3EI} + \frac{2.78h}{tlE}}$	$\frac{Et}{h}$ $K = \dfrac{1}{\left(\frac{4h^2}{l^3} + \frac{2.78}{l}\right)}$
short walls $\frac{h}{l} < 0.3$				
$K = \dfrac{t l E}{2.78h}$	$K = \dfrac{l}{2.78h}$	$K = l$		

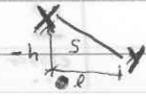
Stiffness analysis

$K_{xx}^{\,y} = K_{yy}^{\,x} = -K_{xy} = -K_{yx}$

vertical members $K_{xx}^{\,y} = \begin{bmatrix} 0 & 0 \\ 0 & 1 \end{bmatrix} \dfrac{AE}{L}$

horizontal members $K_{xx}^{\,y} = \begin{bmatrix} 1 & 0 \\ 0 & 0 \end{bmatrix} \dfrac{AE}{L}$

if  $K_{xx}^{\,y} = \begin{bmatrix} \left(\frac{l}{s}\right)^2 & \left(\frac{lh}{s^2}\right) \\ \left(\frac{lh}{s^2}\right) & \left(\frac{h}{s}\right)^2 \end{bmatrix} \dfrac{AE}{L}$

if  $K_{xx}^{\,y} = \begin{bmatrix} \left(\frac{l}{s}\right)^2 & -\left(\frac{lh}{s^2}\right) \\ -\left(\frac{lh}{s^2}\right) & \left(\frac{h}{s}\right)^2 \end{bmatrix} \dfrac{AE}{L}$

$\begin{matrix} 0 = \\ \cancel{R_{1y}} = \\ 0 = \\ -5 = \end{matrix} \begin{Bmatrix} P_{1x} \\ P_{1y} \\ P_{2x} \\ P_{2y} \end{Bmatrix} = AE \begin{bmatrix} \Sigma K_{11} & K_{12} \\ K_{21} & \Sigma K_{22} \end{bmatrix} \begin{Bmatrix} \Delta_{1x} \\ \Delta_{1y} \\ \Delta_{2x} \\ \Delta_{2y} \end{Bmatrix} \cancel{= 0}$

member force

$\begin{Bmatrix} f_{xy} \\ f_{yx} \end{Bmatrix} = \begin{bmatrix} k_{xx} & -k_{xy} \\ -k_{yx} & k_{yy} \end{bmatrix} \begin{bmatrix} \Gamma_{xy} & 0 \; 0 \\ 0 \; 0 & \Gamma_{xy} \end{bmatrix} \begin{Bmatrix} \Delta x_1 \\ \Delta y_1 \\ \Delta x_2 \\ \Delta y_2 \end{Bmatrix}$

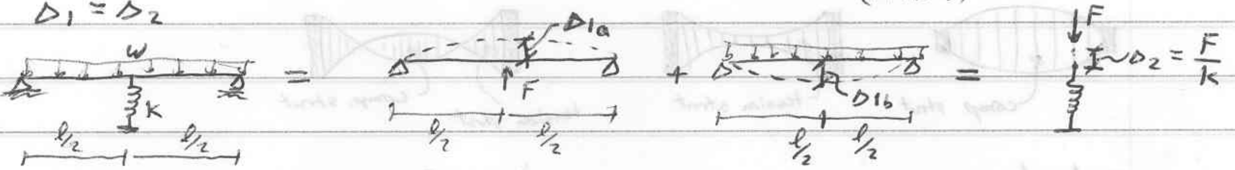
$k = \dfrac{AE}{L}$ $\Gamma_{xy} = \left[\frac{l}{s} \quad \frac{h}{s}\right]$

Elastic Supports
(Consistent Deformation)

$K_{cable} = \frac{AE}{\ell}$ $\left(\frac{K}{in}\right)$

$K_{col} = \frac{AE}{\ell}$ (in inches)

$\Delta_1 = \Delta_2$



$+\downarrow \quad \Delta_1 = \Delta_{1b} - \Delta_{1a} = \Delta_2 = \frac{F}{K}$

$\frac{5w\ell^4}{384EI} - \frac{F\ell^3}{48EI} = \frac{F}{K}$

$F = \_$

$M_{@\ell/2} = \frac{w\ell^2}{8} - \frac{F\ell}{4}$

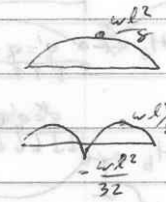
steel manual equations @ each location

dist load w/ values shown above

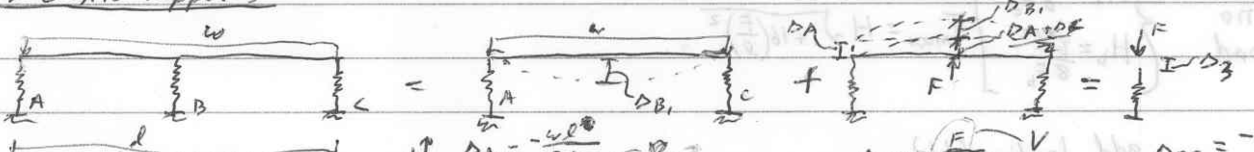
let $r = \frac{K}{(EI/\ell^3)}$

$F = \frac{w\ell}{2}\left(\frac{5}{4} \cdot \frac{r}{(r+48)}\right)$

$M_{@\frac{\ell}{2}} = \frac{w\ell^2}{8}\left(1 - \frac{5}{4}\,\frac{r}{(r+48)}\right)$



3 elastic supports



$+\uparrow \quad \Delta_{A_1} = \frac{-w\ell}{2K}$ R_c

$\Delta_{C_1} = \left(-\frac{w\ell}{2}\right)/K$

$\Delta_{B1} = -\frac{5w\ell^4}{384EI} - \frac{\left(\frac{w\ell}{2K}\right)\cdot 2}{2}$

avg reaction force displacement

$\Delta_{2A} = \frac{F}{2K}$ V

$\Delta_{C2} = \left(\frac{F}{2}\right)/K$

$\Delta_{B2} = \frac{F\ell^3}{48EI} + \frac{\left(\frac{F}{2K}\right)\cdot 2}{2}$

$\Delta_{B3} = -\frac{F}{K}$

if $r = \frac{K}{(EI/\ell^3)}$

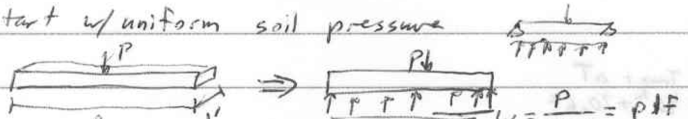
r	exterior support F_A	int support	middle support
typ → 1	.195P	.202P	.206P
10	.159P	.218P	.250P
100	.015P	.270P	.430P
1000	-.488P	.210P	.676P
10000	-.013P	.051P	.925P

rigid bm, Flex. support (good)

flex. support beam (bad)

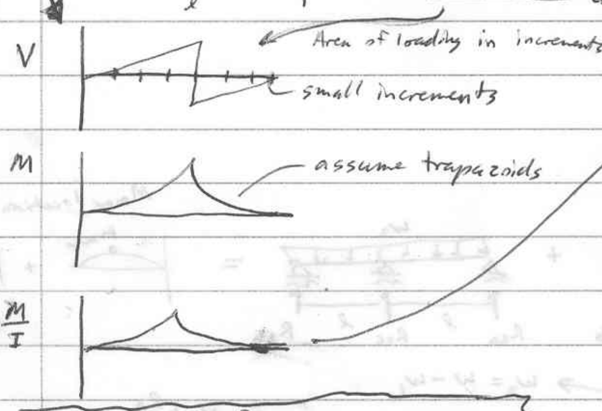
Elastic foundation iterations ($\sigma = \Delta \cdot E_{soil}$)

start w/ uniform soil pressure

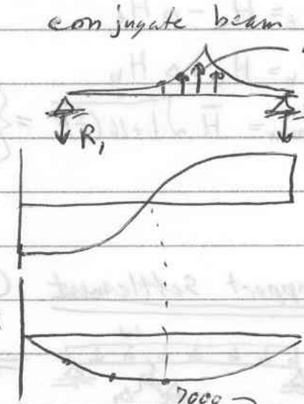


$w = \frac{P}{\ell(1')} = plf$

break into smaller sections (ie 1', 2', ...)

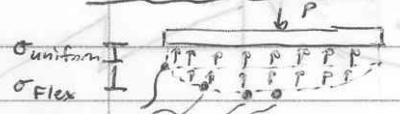


conjugate beam



$\sigma_{Flex} = \frac{E_{soil}}{E_c}\left(\Delta_{Flex} \cdot E_c\right)$

$= \frac{E_{soil}}{E_c}(7000)$



$\sigma_{uniform} + \sigma_{Flex}$ @ each point

Kips $\quad \Sigma\sigma_{Flex} = 2\,\Sigma\left(A_{Flex1} + A_{Flex2} + A_{Flex3}\right)$ up to ℄ (2) = double if symmetrical

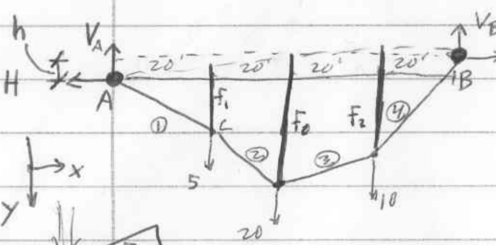
$\sigma_{uniform} = \frac{P - \Sigma\sigma_{Flex}}{\ell} = 2.11$ klf $\rightarrow w$

Iteration #2 (Avg of start + finish @ each point)

$\frac{w_1 + w_2}{2}$

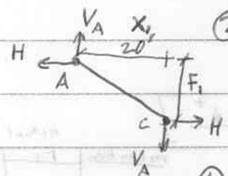
effective range ~~[illegible]~~ $0.03 \le n \le 0.15$

Tension Structures (Point loads) $\left(n = \frac{f}{\ell}\right)$



③ $T_{max} = \sqrt{(V_B)^2 + (H)^2}$

max reaction (shear)

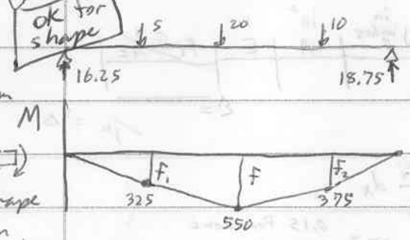


② $\Sigma M_c = 0$

$H = \frac{V_A \cdot X_1}{f_1}$

cable str shape same idea w/ simple beam moment diagram.

ok for shape



deflected shape is M diagram from simple beam

$f_1 = \frac{325}{550} f \le f$

$f_2 = \frac{375}{550} f \le f$

① solve for f

$L_{TOT} = \ell_1 + \ell_2 + \ell_3 + \ell_4 =$

Σ given $= \sqrt{(20')^2 + \left(\frac{325}{550}f\right)^2} + \sqrt{(20)^2 + \left(f - \frac{325}{550}f\right)^2} + \ldots$

$(x)^2 + (f_1)^2$ $\quad (x)^2 + (f - f_1)^2$

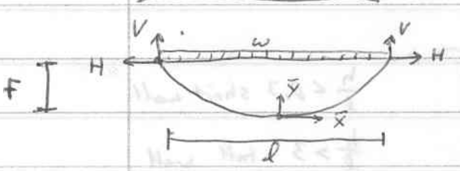
④ second order effects

member	x	y	$\ell_n = \sqrt{x^2+y^2}$	$T = H\left(\frac{\ell_n}{x}\right)$	elongation $\Delta = \frac{T \ell_n}{AE}$
①	20'	f_1			
②	20'	$f - f_1$			
③	20'	$f - f_2$			
④	20'	f_2			

$\Sigma =$ $\approx L_{TOT}$;ok

$\Sigma = \Delta_{TOT}$ $\Rightarrow \ell_{initial} = L_{TOT} - \Delta_{TOT}$ (ℓ_{end})

Tension Structures (distributed loads)



$V = \frac{w\ell}{2}$

$H = \frac{w\ell^2}{8f}$

$T_{max} = \sqrt{(V)^2 + (H)^2} =$

$T_{max} = \frac{w\ell}{8n}\sqrt{16n^2+1}$

$\bar{y} = \frac{4f\bar{x}^2}{\ell^2}$

iterative process to find f

$\left(L_{initial} = \ell\left[1 + \frac{8}{3}n^2 - \frac{32}{5}n^4 + \ldots\right]\right)$ $\quad n_i = \frac{f_i}{\ell}$

approximation $\quad L_{final} \Rightarrow n_f = \frac{f_f}{\ell}$

$f_0 = f_{initial}$ (unloaded cable sag)

$n = \frac{f}{\ell}$

$H = \frac{w\ell^2}{8f_n}$

$\Delta L = \frac{H\ell}{AE}\left[1 + \frac{16}{3}n^2\right]$

$\Delta f = \frac{15(\Delta L)}{16n(5 - 24n^2)}$

$f_{TOT} = f_{initial} + \Delta f = f_0 + \Delta f_n$

if $f_{TOT} \ne f_{initial}$ (unloaded) (f_0)

$f_n = f_{TOT}$

$T_{max} = \sqrt{(V)^2 + (H)^2}$

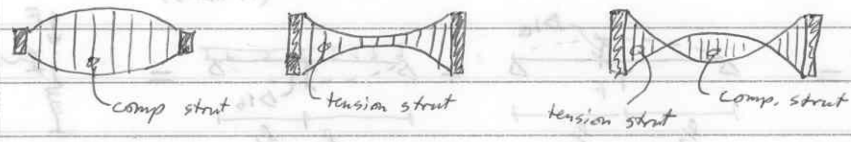
cycle	starting f	Δf	ending f	conservative? ($f_{end} > f_{start}$)	$f_{end} = f_{start}$?
1	$f_0 = 20'$	$\Delta f = 2.81$	$f_0 + 2.81 = 20 + 2.81 = 22.81$	yes, I could stop	no
2	$f_1 = 22.81$	$\Delta f = 2.19$	$f_0 + 2.19 = 20 + 2.19 = 22.19$	no	~~[illegible]~~
3	$f_2 = 22.19$	$\Delta f = 2.31$	$f_0 + 2.31 = 20 + 2.31 = 22.31$	yes, I could stop	closer
...	...	...	...	...	...

$L_{initial}, L_{final}$

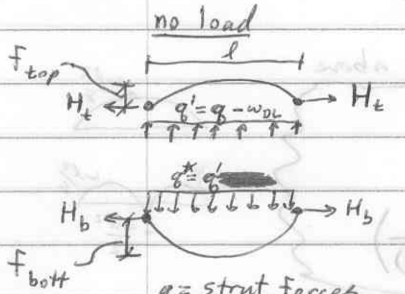
if yes, then you have found $f_{actual} = f_{start} = f_{end}$

assumptions
1. Both cables parabolic shape
2. struts are continuous diaphragm
3. struts exert uniform load

Combined tension Structures



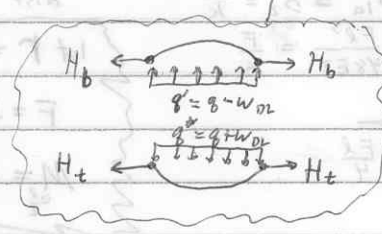
no load



A_t, f_t

A_b, f_b

same equations for all different labels



q = strut forces

no Live load
$$\left\{\begin{aligned} H_t &= \frac{q' l^2}{8 f_t} \\ H_b &= \frac{q^* l^2}{8 f_b} \end{aligned}\right] \quad T_{max} = H\sqrt{1+16\left(\frac{f}{l}\right)^2}$$

add loading (LL)

$$\Delta q = w_{LL}\left(\frac{A_b f_b^{\,2}}{A_t f_t^{\,2} + A_b f_b^{\,2}}\right)$$

$$\Delta H_t = \frac{(q' - \Delta q) l^2}{8 f_t}$$

$$\Delta H_b = \frac{(\Delta q) l^2}{8 f_b}$$

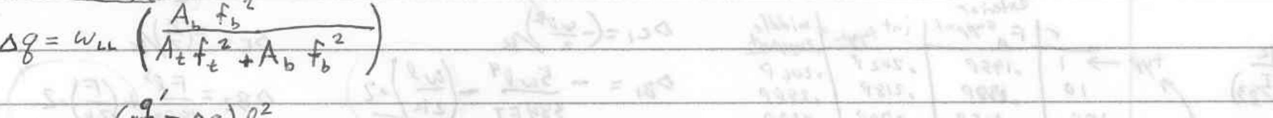
$$\Delta T = \Delta H\sqrt{1+16\left(\frac{f}{l}\right)^2}$$

$$\bar{H}_t = H_t - \Delta H_t$$

$$\bar{H}_b = H_b + \Delta H_b$$

$$\bar{T}_{max} = \bar{H}\sqrt{1+16\left(\frac{f}{l}\right)^2} = \left\{\begin{aligned} \bar{T}_t &= \overset{T_{max} + \Delta T}{40^k + 20.6^k} \\ \bar{T}_b &= T_{max} - \Delta T \end{aligned}\right.$$

Support settlement ($w = w_1 + w_2$)



$$\Delta_1 = \frac{5 w_1 l^4}{384 EI}$$

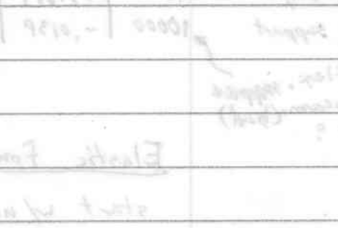
solve for w_1 → $w_2 = w - w_1$

solve for Reactions

solve for reactions

add sheares

$R_A = R_{1A} + R_{2A}$ $\quad R_c = R_{1c} + R_{2c}$ $\quad R_b = R_{1b} + R_{2b}$



compare w/ beam w/o settlement issue