Ryan-Biggs Associates, P.C.
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Hybrid Masonry

Calculation Outline for Type I Hybrid Masonry Walls

1. Layout the building.
2. Determine fenestration.
3. Locate masonry control joints in wall panels and at columns.
4. Determine lateral stiffness for each wall panel.
5. Determine lateral stiffness of each wall.
6. Determine lateral loads - wind and seismic. Develop load combinations.
7. Distribute lateral loads to walls and wall panels.
8. Design the steel framing.
9. Design wall panels for in-plane shear.
10. Design wall panels for out-of-flexure
11. Summarize the results.

Prepared by:
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David T. Biggs, P.E.
Morgan E. Holbrook, E.I.T.
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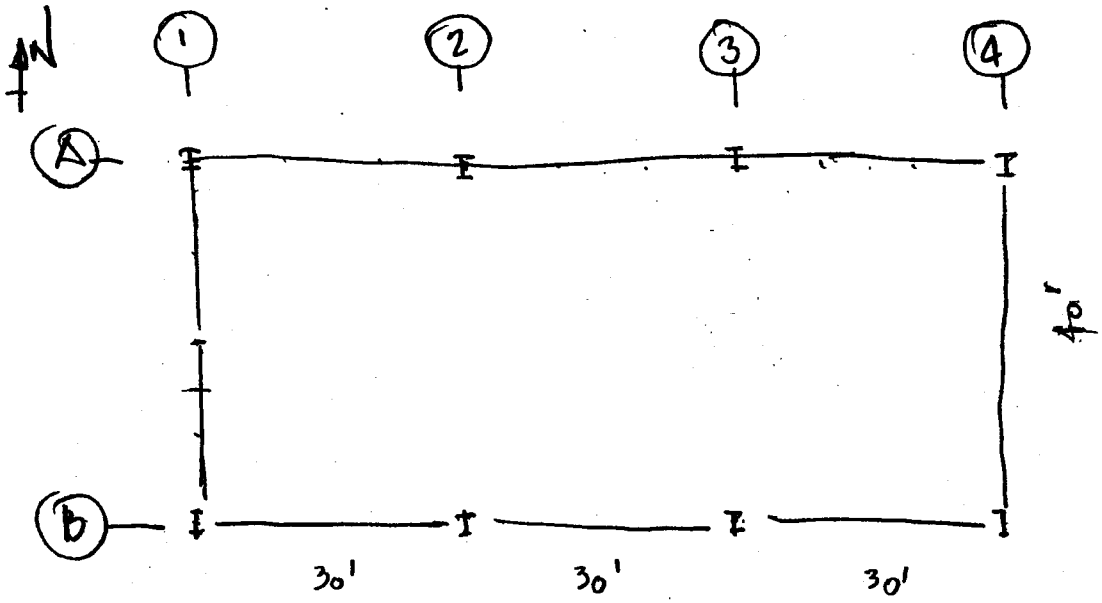
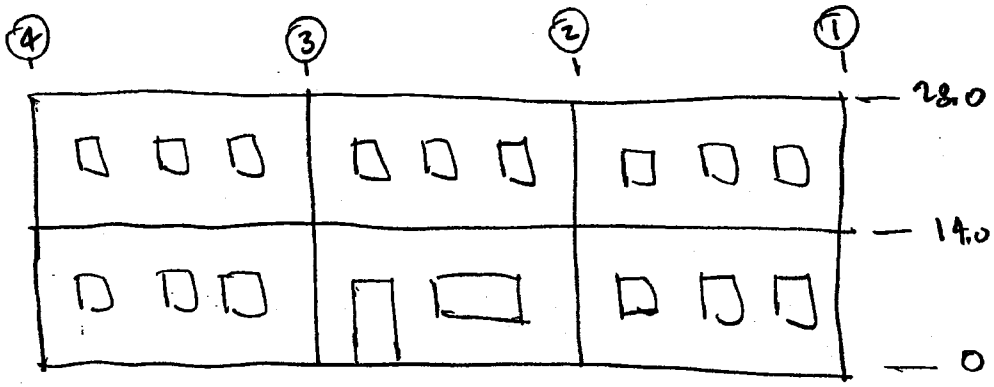
PROJECT NUMBER: 9126 DATE: 10/1/07 PAGE: P1

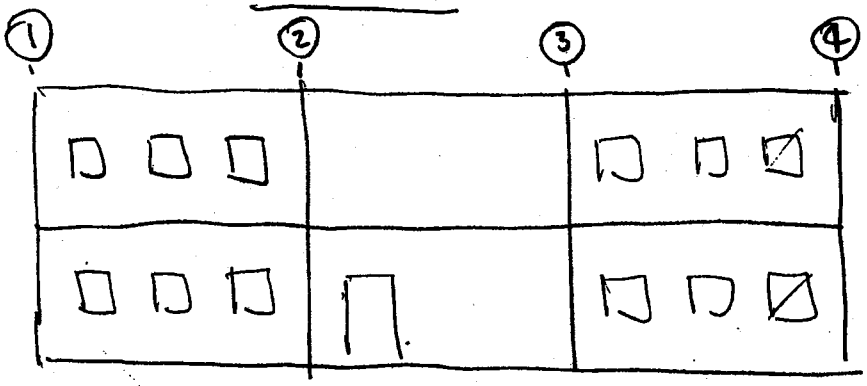
PROJECT: EXAMPLE HYBRID BY: DTB

SUBJECT: TYPE I





NORTH WALL – LONG WALL



SOUTH WALL – LONG WALL

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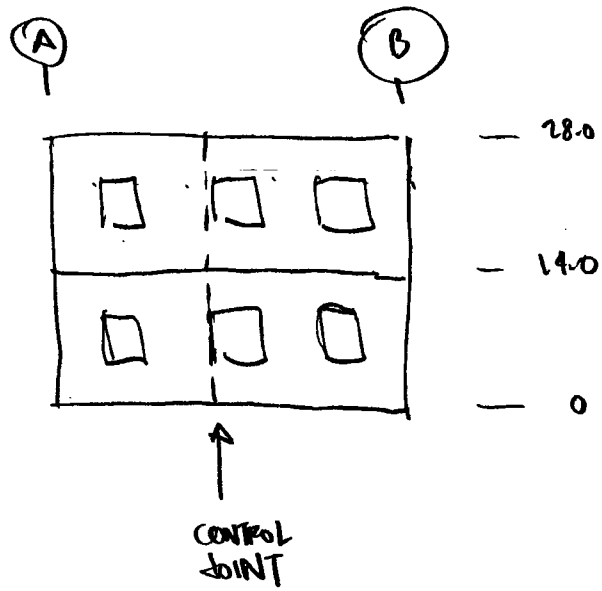
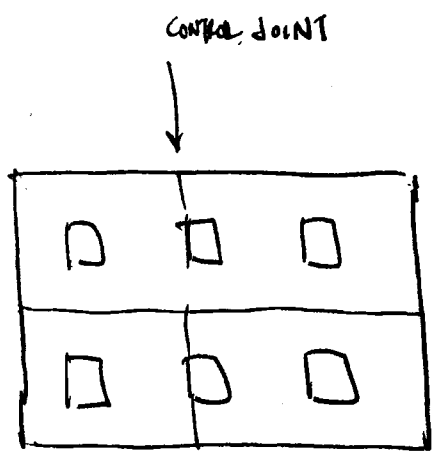
PROJECT NUMBER: 9128 DATE: 11/1/07 PAGE: P2

PROJECT: EXAMPLE HYBRID BY: DTB

SUBJECT: TYPE I



WEST WALL



EAST WALL

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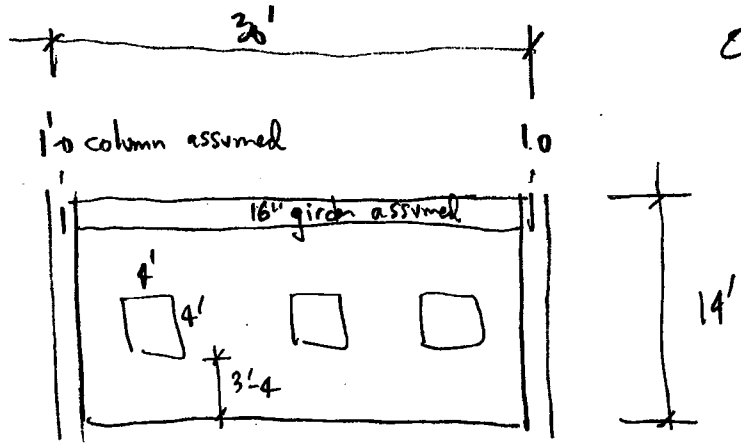
PROJECT NUMBER: 9128 DATE: 11/1/07 PAGE: L1

PROJECT: EXAMPLE HYBRID BY: DRB

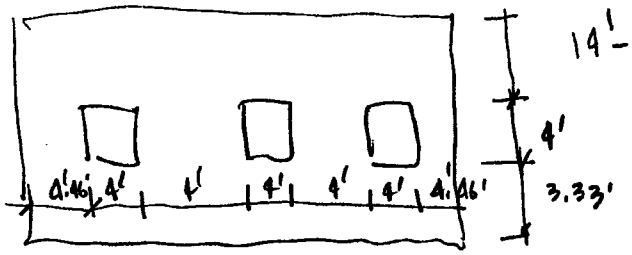
SUBJECT: TYPE I



Ordinary (Type I) Hybrid

1/2" joints @ columns & girders

CMU



Masonry Dimensions

$14' - 16' - \frac{1}{2}'' = 14 - 1.37' = 12.63'$

$- \; 7.33$

5.33

$30' - 1' - 1'' = 28.92'$

GENERAL FORMULAS TO DEVELOP STIFFNESS

$$\Delta_c = \frac{Ph^3}{3E_m I} + \frac{1.2Ph}{E_v A} \Rightarrow \frac{Pl}{E_m t}\left[4\left(\frac{h}{l_w}\right)^3 + 3\left(\frac{h}{l_w}\right)\right]$$ MDG 9.2-3

$$\Delta_F = \frac{Pl}{E_m t}\left[\left(\frac{h'}{l_w}\right)^3 + 3\left(\frac{h'}{l_w}\right)\right]$$

$$K_{wall} = \frac{1}{\Delta}$$

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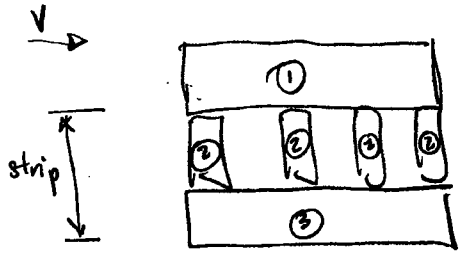
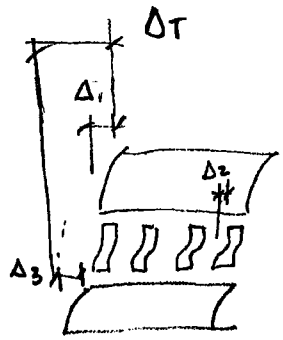
PROJECT NUMBER: 9128 DATE: 11/1/07 PAGE: L2

PROJECT: EXAMPLE HYBRID BY: DJB

SUBJECT: TYPE I

3 WINDOW PANEL





① Δ_1 as cantilever : $4\left(\frac{h}{l}\right)^3 + 3\left(\frac{h}{l}\right) = 4\left(\frac{5.33}{28.92}\right)^3 + 3\left(\quad\right)$

$= 4(.0062) + 3(.184)$

$= .025 + .552 \doteq 0.577$

② Δ_2 as fixed: $\left(\frac{h}{l}\right)^3 + 3\left(\frac{h}{l}\right) \doteq \left(\frac{4}{4}\right)^3 + 3\left(\frac{4}{4}\right) = 1 + 3 = 4.0$ $\quad K = \frac{1}{4}$

③ Δ_3 as fixed : $\left(\frac{3.33}{28.92}\right)^3 + 3\left(\quad\right) \doteq (.115)^3 + 3(.115)$

$= (.0015) + 3(.115)$

$= .0015 + .345 \doteq 0.347$

$\Delta_{wall} = \Delta_{solid(c)} - \Delta_{strip(c)} + \Delta_{2,3(f)}$

$\Delta_{strip(c)} = 4\left(\frac{7.33}{28.92}\right)^3 + 3\left(\frac{7.33}{28.92}\right) \doteq .065 + .760 \doteq .825$

$\Delta_{2(f)} = \frac{1}{1/4 + 1/4 + 1/4 + 1/4} = 1.0$

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PROJECT NUMBER: 9128 DATE: 11/1/07 PAGE: L3

PROJECT: EXAMPLE HYBRID BY: DMB

SUBJECT: TYPE I

$$\Delta_{2,3}(f) = \Delta_{solid\ 2,3}(f) - \Delta_{strip}(f) + \Delta_2(f)$$

$$\Delta_{solid\ 2,3}(f) = \left(\frac{7.33}{28.92}\right)^3 + 3\left(\frac{7.33}{28.92}\right) \doteq .016 + .760 \doteq 0.776$$

$$\Delta_{strip}(f) = \left(\frac{4.0}{28.92}\right)^3 + 3\left(\frac{4}{28.92}\right) \doteq .003 + .415 \doteq .418$$

$\Delta_2(f) = 1.0$ see pt2

$$\Delta_{2,3}f = 0.776 - .418 + 1.0 = 1.358$$

$$\Delta_{wall} = 1.649 - .825 + 1.358 \doteq 2.182 \qquad K_{wall} = \frac{1}{\Delta} = 0.458$$

$$\Delta_{solid} = 4\left(\frac{12.66}{28.92}\right)^3 + 3\left(\frac{12.66}{28.92}\right) \doteq .336 + 1.313 \doteq 1.649$$

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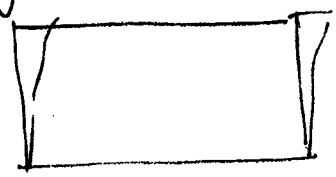
PROJECT NUMBER: 9128 DATE: 11/1/07 PAGE: L4

PROJECT: EXAMPLE HYBRID BY: DTB

SUBJECT: TYPE I

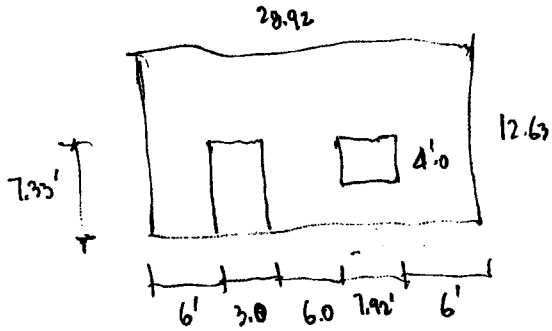
PANEL W/O WINDOW



$$\Delta_c = 4\left(\frac{12.66}{28.92}\right)^3 + 3\left(\quad\right) = 1.649 \quad (p.L3)$$

$$K_c = \frac{1}{\Delta_c} = .606$$

PANEL W/ DOOR OPENING & WINDOW



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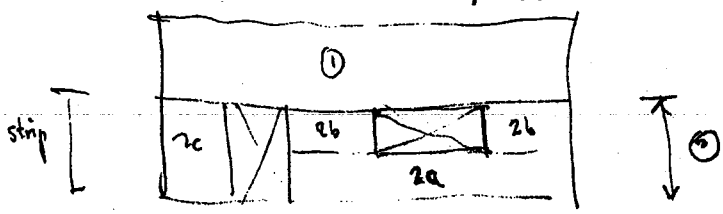
$$\Delta_{wall} = \Delta_c - \Delta_{strip(c)} + \Delta_{2a,2b,2c}(f)$$

$$\Delta_{2a,2b,2c}(f) = \frac{1}{K_{2a,2b,2c}(f)}$$

$$K_{2a,2b,2c}(f) = K_{2a}(f) + K_{2b,2c}(f)$$



$$K_{2b,2c}(f) = \frac{1}{\Delta_{2b,2c}(f)}$$

$$\Delta_{2b,2c}(f) = \Delta_{solid\,2b}(f) - \Delta_{strip\,2b}(f) + \Sigma\Delta_{2b}(f)$$

$\Delta_c = 1.649$ (pL4) $\qquad \Delta_{strip(c)} = 0.825$ (pL4)

$$\Delta_{solid\,2b}(f) = \left(\frac{7.33}{19.92}\right)^3 + 3\left(\frac{7.33}{1992}\right) \doteq .050 + 1.104 \doteq 1.154$$

$$\Delta_{strip\,2b}(f) = \left(\frac{4}{19.92}\right)^3 + 3\left(\frac{4.0}{1992}\right) \doteq 0.008 + 0.602 \doteq 0.610$$

$$\Delta_{2b}(f) = \left(\frac{4.0}{6}\right)^3 + 3\left(\frac{4}{6}\right) = (.666)^3 + 3\left(\frac{2}{3}\right) = .30 + 2.0 = 2.30$$

$$\Sigma\Delta_{2b}(f) = \frac{1}{2.3} + \frac{1}{2.3} = \frac{1}{1.15} \doteq 0.870$$

$$\Delta^\circ = \frac{1}{.870} = 1.149$$

$$\Delta_{2b,2c}(f) = 1.154 - 0.610 + 1.149 = 1.693$$

$$K_{2b,2c}(f) = \frac{1}{\Delta} = \frac{1}{1.693} \doteq 0.591$$

$$K_{2c}(f) = \frac{1}{\Delta_{2c}(f)} = \frac{1}{\left(\frac{7.33}{6}\right)^3 + 3\left(\frac{7.33}{6}\right)} \doteq \frac{1}{1.823 + 3.665} \doteq \frac{1}{5.488} = 0.182$$

$$K_{2a,2b,2c}(f) \doteq 0.591 + 0.182 \doteq 0.773 \qquad \Delta_{2a,2b,2c}(f) = 1.294$$

$$\Delta_{wall} = 1.649 - 0.825 + 1.294 = 2.118$$

$$K_{wall} = 0.472$$

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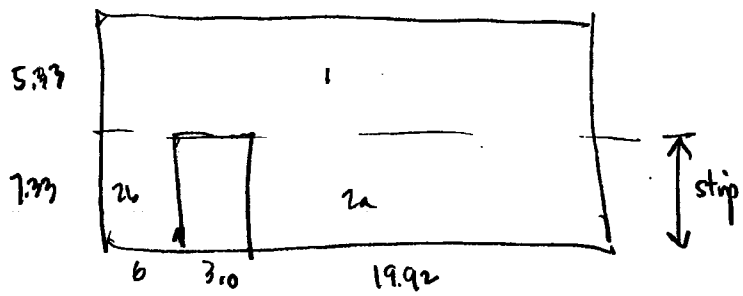
PANEL W/ DOOR ONLY



$$\Delta = \Delta_{solid(c)} - \Delta_{strip(c)} + \Delta_{2a,2b(f)}$$

$\Delta_{solid(c)} = 1.649$ (p.L4)
$\Delta_{strip(c)} = 0.825$ (p.L4)

$$\Delta_{2a,2b(f)} = \frac{1}{K_{2a,K2b}(f)}$$

$$K_{2a,2b}(f) = K_{2b}(f) + K_{2a}(f) = \frac{1}{\Delta_{2b(f)}} + \frac{1}{\Delta_{2a(f)}}$$

$$\Delta_{2b}(f) = \left(\frac{7.33}{6}\right)^3 + \left(\frac{7.33}{6}\right)3 \doteq 1.823 + 3.665 \doteq 5.488$$

$$\Delta_{2a}(f) = \left(\frac{7.33}{19.92}\right)^3 + 3\left(\frac{7.33}{19.92}\right) \doteq 0.050 + 1.104 \doteq 1.154$$

$$K_{2a,2b}(f) = \frac{1}{5.488} + \frac{1}{1.154} \doteq 0.182 + 0.867 \doteq 1.049 \qquad \Delta_{2a,2b(f)} = 0.954$$

$$\Delta_{wall} = 1.649 - 0.825 + 0.954 \doteq 1.778 \qquad K_{wall} = 0.562$$

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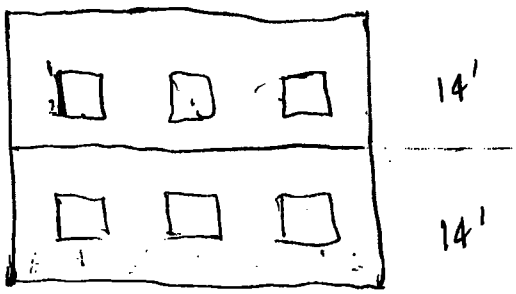
PROJECT NUMBER: 9128 DATE: 11/1/07 PAGE: 51

PROJECT: EXAMPE HYBRID BY: DTB

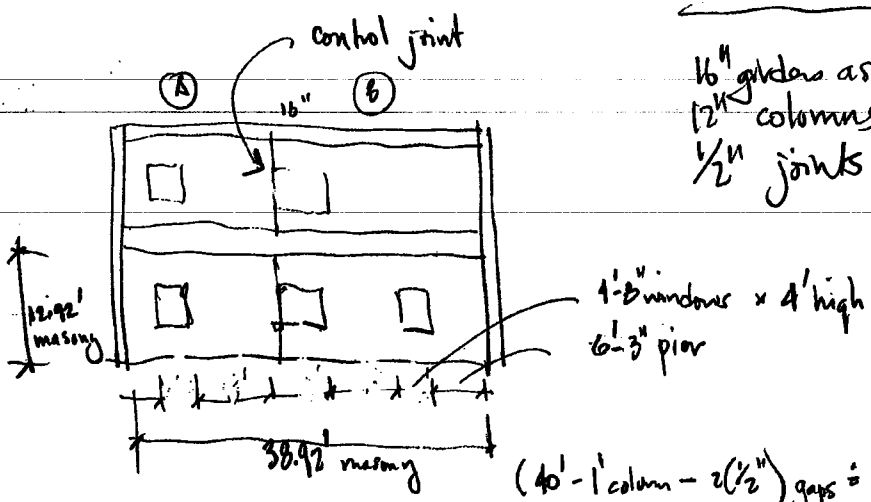
SUBJECT: TYPE I

WALL PANELS



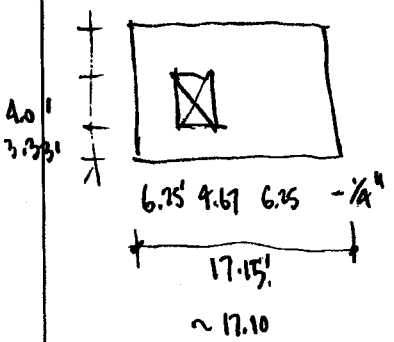
Determine Masonry Dimensions

16" girders assumed
12" columns
1/2" joints

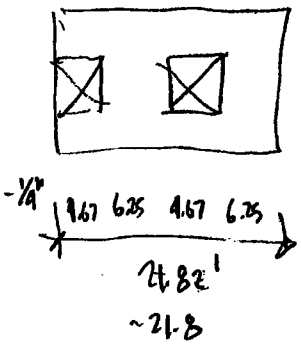


(40' - 1' column - 2(1/2") gaps = 38.92')

TYPE A



TYPE B



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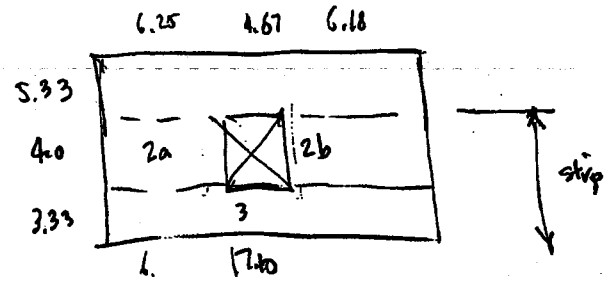
PROJECT NUMBER: 9128 DATE: 11/1/07 PAGE: S2

PROJECT: EXAMPLE HYBRID BY: DVB

SUBJECT: TYPE I

(A) ONE WINDOW PANEL



$$\Delta_{wall} = \Delta_{solid(c)} - \Delta_{strip(c)} + \Delta_{2a,2b,3(f)}$$

$$\Delta_{2a,2b,3(f)} = \Delta_{strip(f)} - \Delta_{2(f)} + \Delta_{2a,2b(f)}$$

$$\Delta_{solid(c)} = 4\left(\frac{12.66}{17.10}\right)^3 + 3\left(\frac{12.66}{17.10}\right) \doteq 1.623 + 2.22 = 3.844$$

$$\Delta_{solid\,strip(c)} = 4\left(\frac{7.33}{17.10}\right)^3 + 3\left(\frac{7.33}{17.10}\right) \doteq 0.315 + 1.286 \doteq 1.601$$

$$\Delta_{strip(f)} = \left(\frac{7.33}{17.10}\right)^3 + 3\left(\frac{7.33}{17.10}\right) \doteq 0.079 + 1.286 \doteq 1.365$$

$$\Delta_{2a(f)} = \left(\frac{4}{6.25}\right)^3 + 3\left(\frac{4}{6.25}\right) \doteq 0.262 + 1.920 \doteq 2.182$$

$$\Delta_{2b(f)} \doteq \left(\frac{4}{6.18}\right)^3 + 3\left(\frac{4}{6.18}\right) \doteq .271 + 1.942 \doteq 2.213$$

$$\frac{1}{\Delta_{2a,2b(f)}} = \frac{1}{\Delta_{2af}} + \frac{1}{\Delta_{2bf}}$$

$$\frac{1}{\Delta} = \frac{1}{2.182} + \frac{1}{2.213}$$

$$\frac{1}{\Delta} = .458 + .452$$

$$\frac{1}{\Delta} = 0.91$$

$$\Delta = 1.1$$

$$\Delta_{2(f)} = \left(\frac{4}{17.10}\right)^3 + 3\left(\frac{4}{17.10}\right) = 0.013 + .702 \doteq .715$$

$$\Delta_{wall(f)} = (3.844) - (1.601) + [1.365 - .715 + 1.1] = 3.993$$

$$K_{WALL} = 0.250$$

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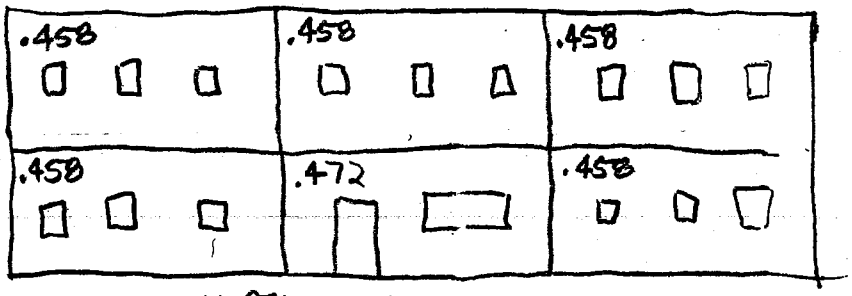
PROJECT NUMBER: 9128 DATE: 11/1/07 PAGE: L7

PROJECT: EXAMPLE HYBRID BY: DTB

SUBJECT: TYPE I

WALLS WITH PANEL STIFFNESSES

STIFFNESS/FLOOR

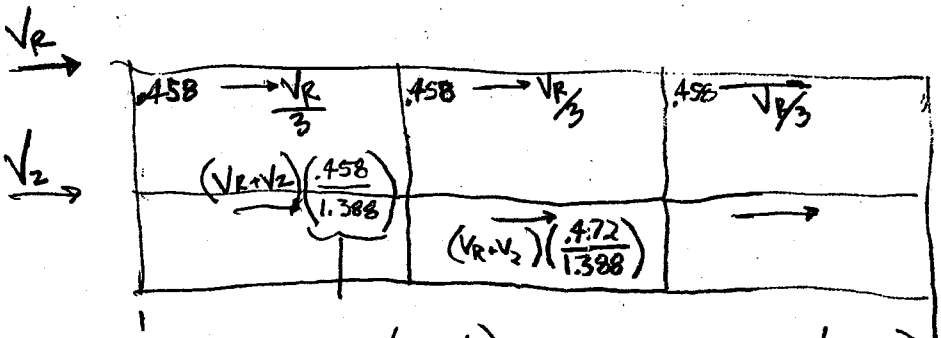


$\Sigma k = 1.374$

$\Sigma k = 1.388$

NORTH WALL

DISTRIBUTION OF LOAD



33% (V_R+V_2) 34% (V_R+V_2) 33% (V_R+V_2)

NORTH WALL

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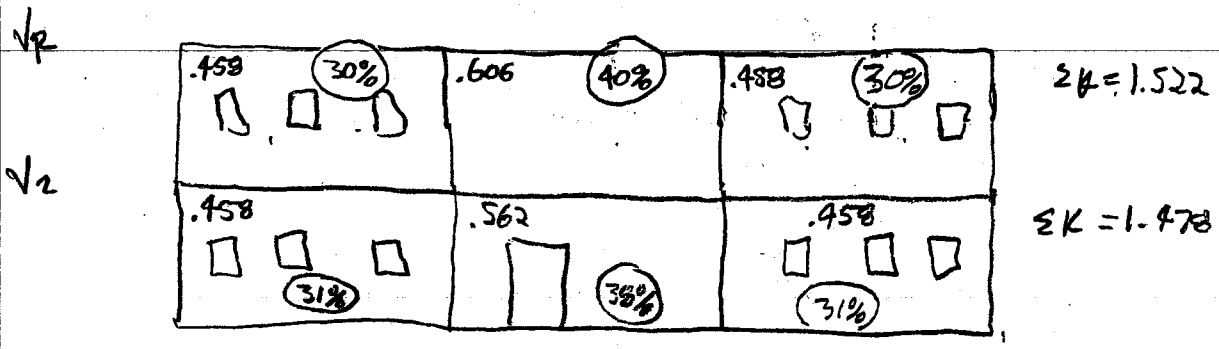
PROJECT NUMBER: 9128 DATE: 11/1/07 PAGE: 18

PROJECT: EXAMPLE HYBRID BY: DTB

SUBJECT: TYPE I



SOUTH WALL

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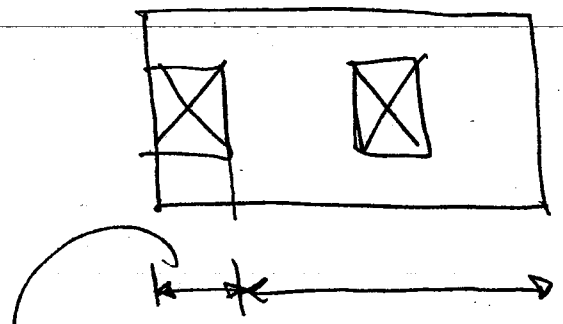
PROJECT NUMBER: 9128 DATE: 11/1/07 PAGE: S3

PROJECT: EXAMPLE HYBRID BY: DIB

SUBJECT: TYPE I

(B) CJ TYPE

TWO WINDOW PANEL



IDENTICAL TO (A)

$K_{wall} = 0.250$

∴ 50% distribution between (A) + (B)

SINCE WINDOW ALIGNS w/ CJ, IGNORE STIFFNESS CONTRIBUTION OVER EDGE WINDOW

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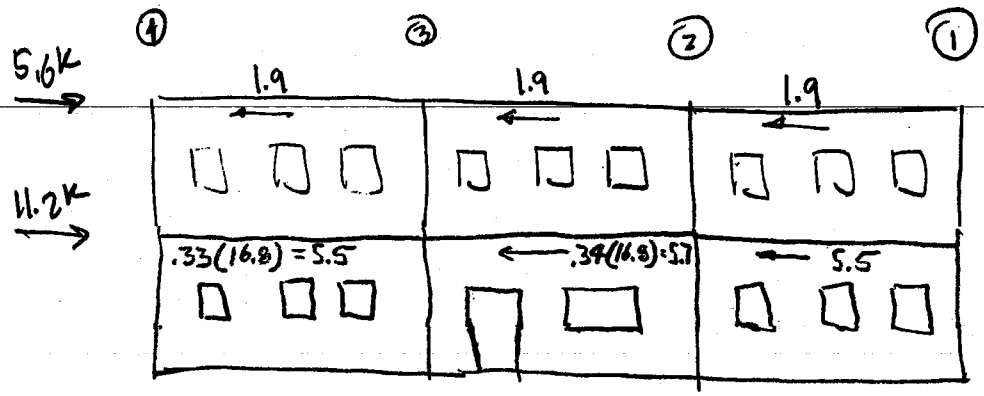
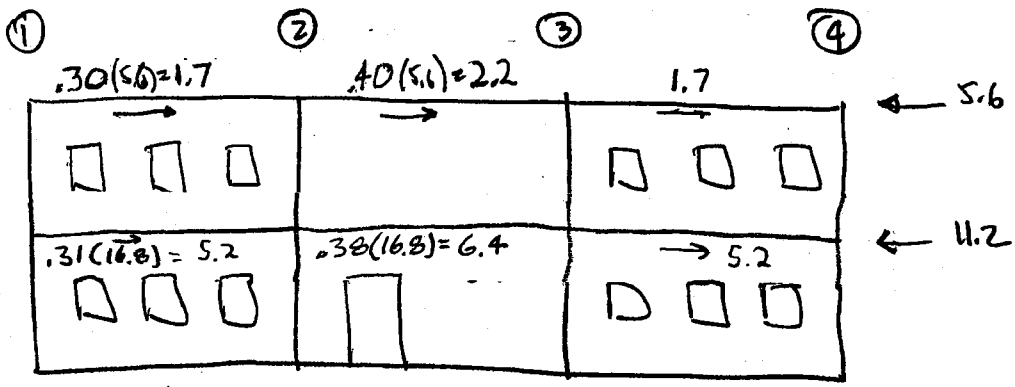
PROJECT NUMBER: 9128 DATE: 11/1/07 PAGE: 19

PROJECT: EXAMPLE HYBRID BY: DTB

SUBJECT: TYPE I



NORTH WALL



SOUTH WALL

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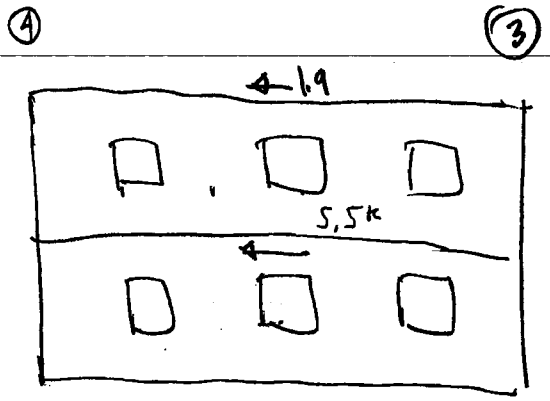
PROJECT NUMBER: 9128 DATE: 11/1/07 PAGE: L10

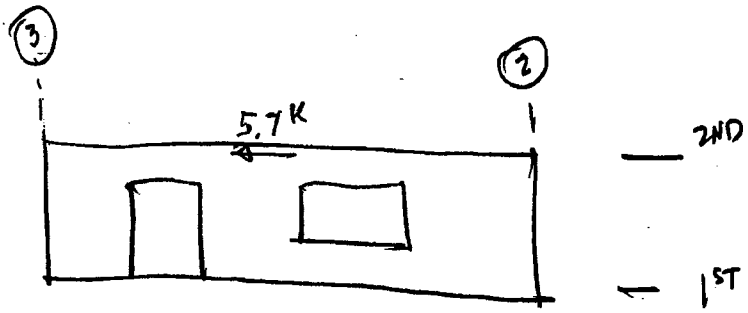
PROJECT: EXAMPLE HYBRID BY: DTB

SUBJECT: TYPE I



WORST CASES

(DESIGN LOADS)



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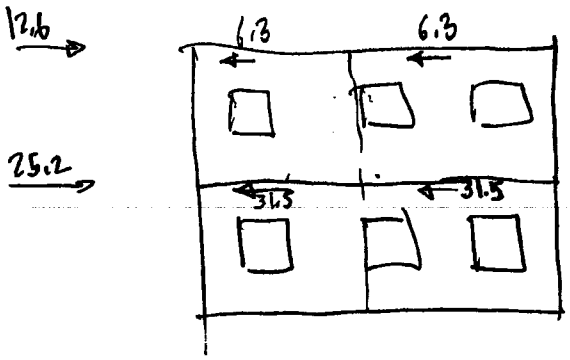
PROJECT NUMBER: 9128 DATE: 11/1/07 PAGE: S4

PROJECT: EXAMPLE HYBRID BY: DJB

SUBJECT: TYPE I

WALL DESIGN LOADS



WEST WALL
(EAST WALL SIMILAR)

Steel Design Criteria - Hybrid -Type I Example Office Building

- Steel is designed for gravity loads only; the masonry shear walls will resist lateral loads.
- Floor design is composite.
- ASD method is used for the steel design to be consistent with the preferred masonry design method (ASD). Other methods are acceptable.
- See page SS-2 for loads including pre- and post-composite dead load breakdown.
- A construction live load of 20 psf is included in all steel designs.

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OFFICE BLD - 2 STORY (SEE P1)

FLOOR:
LL = 50 PSF
DL =

		LOAD ON STEEL	LOAD ON MASONRY
PRE-COMPOSITE	• MASONRY WALL ABOVE	541 PLF	—
	• 5½" SLAB ON DECK	54 PSF	—
	• DECK (22 GA)	1.5 PSF	—
	• STEEL	8 PSF	—
POST-COMPOSITE	• CEILING	2 PSF	—
	• MECH/ELEC	5.5 PSF	—
	• MISC DL	15 PSF	—
	TOTAL	86 PSF	Ø PSF

ROOF:
SL = 40 PSF
DL =

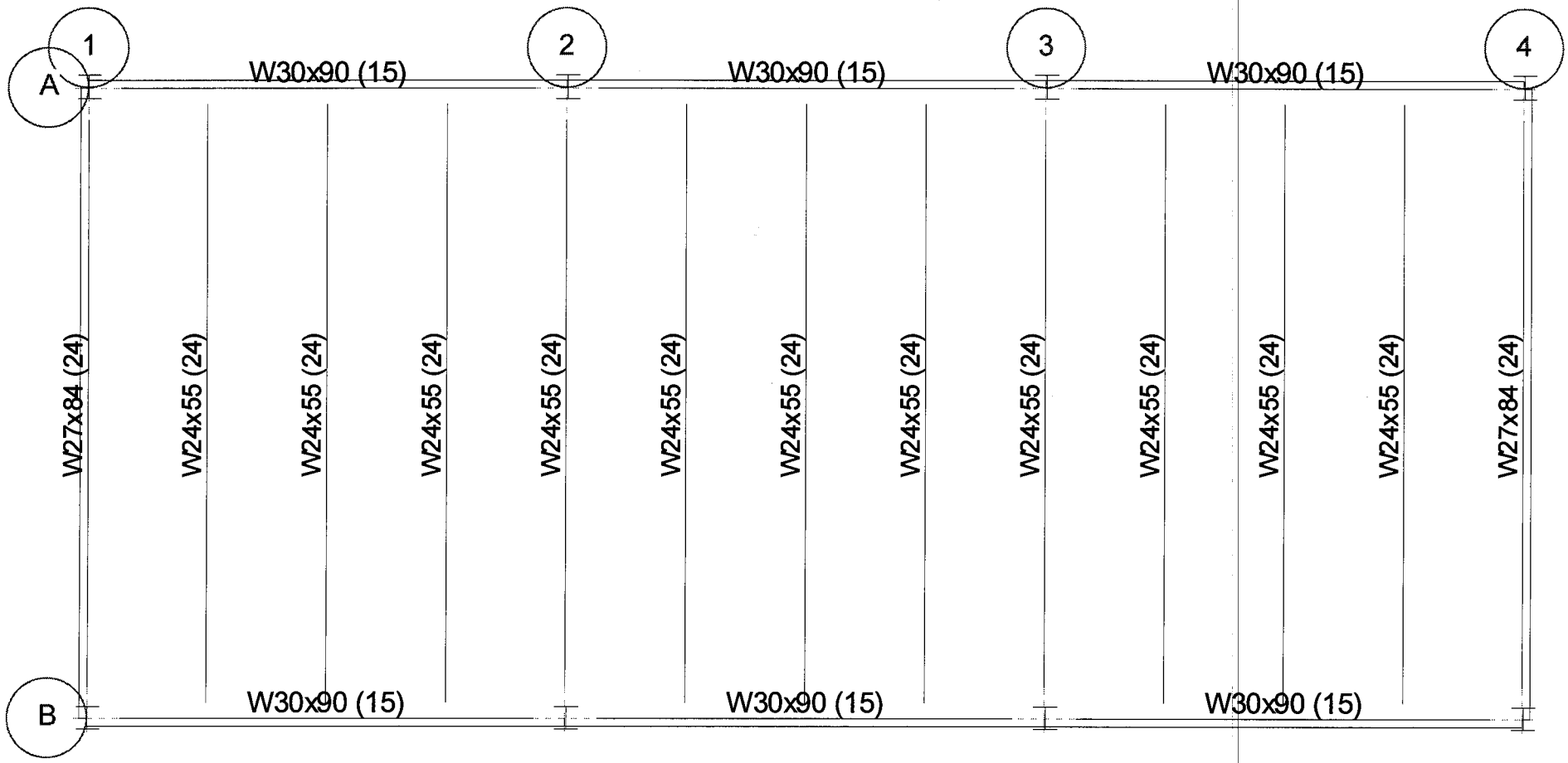
PRE-COMPOSITE	• 4½" SLAB ON DECK	42 PSF	—
	• DECK (22 GA)	1.5 PSF	—
	• STEEL	7 PSF	—
POST-COMPOSITE	• CEILING	2 PSF	—
	• MECH ELEC	5.5 PSF	—
	• MISC DL	10 PSF	—
	TOTAL	68 PSF	Ø PSF

Floor Map

RAM Steel v11.0
DataBase: Office Bld Typ1
Building Code: IBC

01/02/08 14:15:01
Steel Code: ASD 9th Ed.

Floor Type: Second



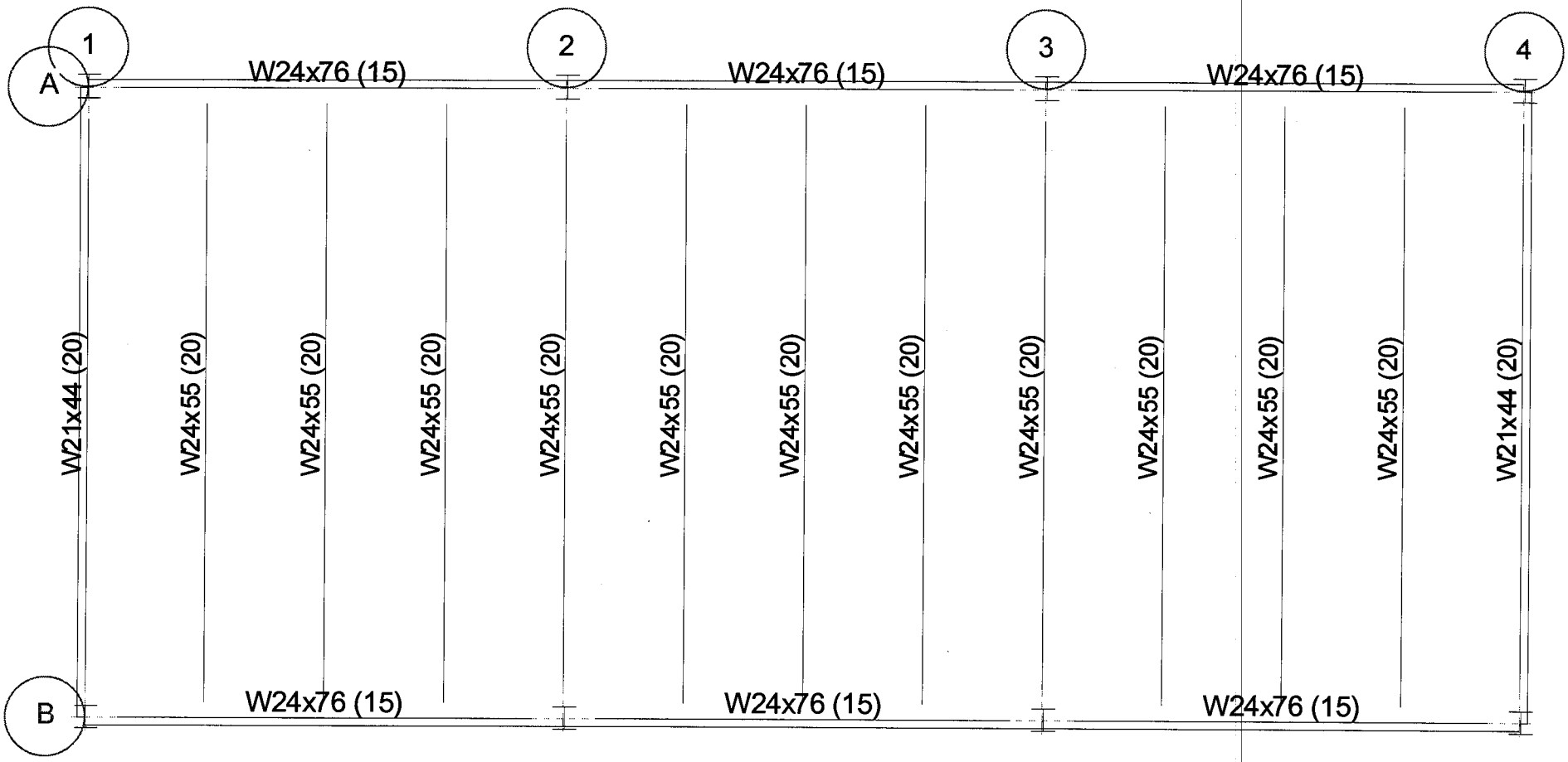
PAGE: 55-3

Floor Map

RAM Steel v11.0
DataBase: Office Bld Typ1
Building Code: IBC

01/02/08 13:58:34
Steel Code: ASD 9th Ed.

Floor Type: Roof



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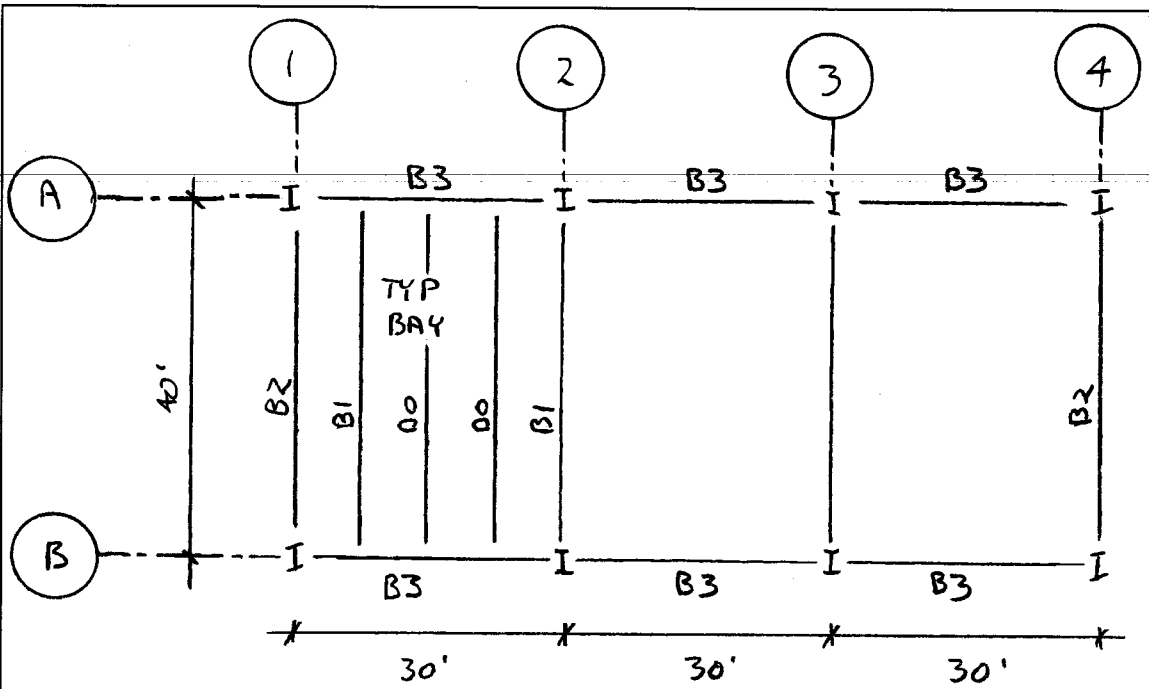
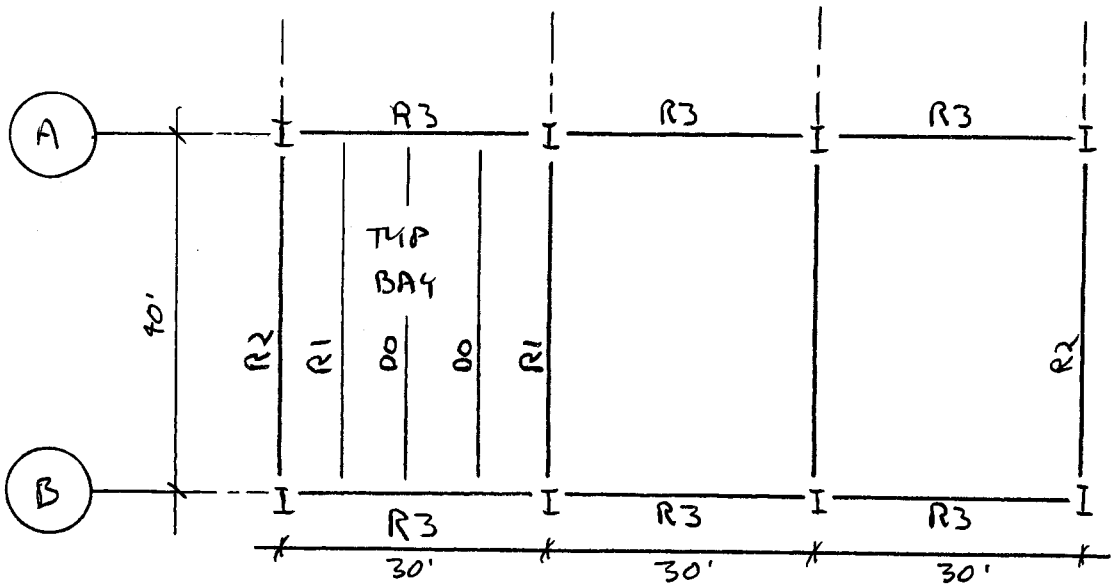
PROJECT NUMBER: 9128 DATE: 11-29-07 PAGE: SS-5

PROJECT: EXAMPLE HYBRID TYPE I/II BY: MEH

SUBJECT: BEAM PLAN



2ND FLOOR BEAM PLAN



ROOF BEAM PLAN

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BEAM B1 DESIGN

- TYPICAL INTERIOR BAY, 2nd FLOOR
- LOAD COMBO ANALYZED = D + SDL + L

SPAN = 40'
TRIB = 7.5'
F_y = 50 ksi
L_b = 0'-0"
ASD METHOD

D = 64 PSF x 7.5' = .480 KLF
SDL = 23 PSF x 7.5' = .173 KLF
L = 50 PSF x 7.5' = .375 KLF

USE: RAMSBEAM TO DESIGN

D = D + SDL = (.480 + .173) = .653 KLF
CONSTR. D = .480 KLF
L = .375 KLF
CONSTR. L = 20 PSF x 7.5' = .150 KLF

DEFLECTION: LIMIT PRE-COMPOSITE CROSS BAY Δ TO SMALLER OF $\ell/360$ OR 1" PER RBA TECH NOTE => $\ell/360 = \frac{50 \times 12}{360} = 1.67"$ ∴ USE $\boxed{1"}$

DEFAULTS:	L/d	Δ
Initial	0.00	0.75"
Post LL	360	0.00
Post Super.	360	0.00
NET TOT.	240	0.00

OPTIMIZED BEAM IS: $\boxed{W24 \times 55\ [24]}$ Δ = 1.1" (TOTAL)
Δ = 0.7" (INITIAL)

INITIAL Δ ALLOWABLE FOR GIRDER B3 = 1.0" − 0.7" = 0.3"

Gravity Beam Design

RAM SBeam v3.0
Licensed to: Ryan-Biggs Associates

BM B1 - HYBRID TYPE I 11/30/07 14:21:04

STEEL CODE: ASD 9th Ed.

SPAN INFORMATION (ft): I-End (0.00,0.00) J-End (40.00,0.00)

Beam Size (Optimum) = W24X55 Fy = 50.0 ksi
Total Beam Length (ft) = 40.00
Distance to Adjacent Beam on Left (ft) = 7.5
Distance to Adjacent Beam on Right (ft) = 7.5

COMPOSITE PROPERTIES (Not Shored):

				Left			**Right**
Concrete thickness (in)				3.50			3.50
Unit weight concrete (pcf)				150.00			150.00
f'c (ksi)				3.00			3.00
Decking Orientation				perpendicular			perpendicular
Decking type				USD 2" Lok-Floor			USD 2" Lok-Floor
beff (in)	=	90.00		Y bar(in)	=		22.51
Seff (in3)	=	149.02		Str (in3)	=		182.02
Ieff (in4)	=	2756.74		Itr (in4)	=		4111.41
Stud length (in)	=	4.00		Stud diam (in)	=		0.75

Stud Capacity (kips) q = 8.6
# of studs: Full = 72 Partial = 24 Actual = 24
Number of Stud Rows = 1 Percent of Full Composite Action = 25.77
Top flange braced by decking.

LINE LOADS (k/ft):

Load	Dist (ft)	DL	CDL	LL	CLL
1	0.000	0.653	0.480	0.375	0.150
	40.000	0.653	0.480	0.375	0.150

SHEAR: Max V (DL+LL) = 20.56 kips fv = 2.30 ksi Fv = 18.78 ksi

MOMENTS:

Span	Cond	Moment	@	Lb	Cb	Tension Flange		Compr Flange	
		kip-ft	ft	ft		fb	Fb	fb	Fb
Center	PreCmp+	126.0	20.0	0.0	1.00	13.15	33.00	13.15	33.00
	Max +	205.6	20.0	---	---				
	Mmax/Seff					16.56	33.00	---	---
	Mconst/Sx+Mpost/Seff					18.84	45.00	---	---
Controlling		205.6	20.0	---	---	16.56	33.00	---	---

fc (ksi) = 0.24 Fc = 1.35

REACTIONS (kips):

	Left	**Right**
Initial reaction	12.60	12.60
DL reaction	13.06	13.06
Max +LL reaction	7.50	7.50
Max +total reaction	20.56	20.56

DEFLECTIONS:

Initial load (in) at 20.00 ft = -0.701 L/D = 685

Gravity Beam Design

Live load (in)	at	20.00 ft	=	-0.270	L/D	=	1777
Post Comp load (in)	at	20.00 ft	=	-0.395	L/D	=	1216
Net Total load (in)	at	20.00 ft	=	-1.096	L/D	=	438

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BEAM B2 DESIGN

- TYPICAL EDGE BEAMS ALONG COL LINES 1 & 4, 2nd FLOOR
- LOAD COMBO ANALYZED = D + SDL + L

SPAN = 39'
TRIB = 3.75'
F_y = 50 ksi
L_b = 0'-0"
ASD METHOD

D* = 56 PSF × 3.75' = .210 KLF
SDL = 23 PSF × 3.75' = .086 KLF
L = 50 PSF × 3.75' = .188 KLF
WALL WEIGHT = 541 PLF; ADD TO D
* ESTIMATE FOR STEEL WT (8 PSF) SUBTRACTED; LET RAM INCLUDE IN CALC., SINCE IT WILL LIKELY BE MORE THAN 8 PSF

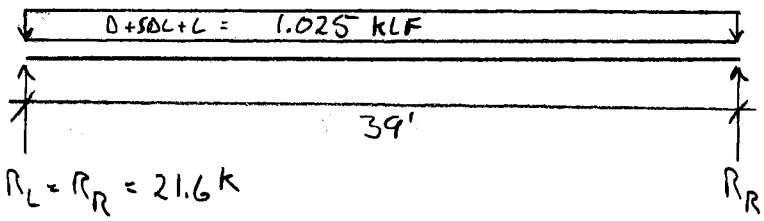
USE: RAM SBEAM TO DESIGN

D = D+SDL+WALL = (.210 + .086 + .541) = .837 KLF
CONSTR. D = .210 KLF
L = .188 KLF
CONSTR. L = 20 PSF × 3.75' = .075 KLF

DEFLECTION: BEAM WILL SUPPORT A MASONRY WALL => l/600 OR .3"
$l/600 = \frac{40 \times 12}{600} = 0.8"$ ∴ USE 0.3" FOR POST-COMPOSITE SUPERIMPOSED DEFLEC.

* SEE B1 FOR ADDITIONAL Δ REQUIREMENTS.

BEAM DIAGRAM:



M = 210.9 K-FT

OPTIMIZED BEAM: W27x84 [20] SEE RAM OUTPUT

Gravity Beam Design

PAGE: SS-10

RAM SBeam v3.0
Licensed to: Ryan-Biggs Associates B2 - HYBRID TYPE I 12/04/07 08:47:23

STEEL CODE: ASD 9th Ed. LC: D+SDL+L

SPAN INFORMATION (ft): I-End (0.00,0.00) J-End (39.00,0.00)

Beam Size (Optimum) = W27X84 Fy = 50.0 ksi
Total Beam Length (ft) = 39.00
Distance to Adjacent Beam on Left (ft) = 7.5
Distance to Adjacent Edge on Right (ft) = 0.4

COMPOSITE PROPERTIES (Not Shored):

	Left	Right
Concrete thickness (in)	3.50	3.50
Unit weight concrete (pcf)	150.00	150.00
f'c (ksi)	3.00	3.00
Decking Orientation	perpendicular	perpendicular
Decking type	USD 2" Lok-Floor	USD 2" Lok-Floor

beff (in)	=	50.04	Y bar(in)	=	21.00
Seff (in3)	=	260.68	Str (in3)	=	289.73
Ieff (in4)	=	4877.72	Itr (in4)	=	6112.69
Stud length (in)	=	4.00	Stud diam (in)	=	0.75

Stud Capacity (kips) q = 8.6
# of studs: Full = 48 Partial = 20 Actual = 20
Number of Stud Rows = 1 Percent of Full Composite Action = 38.62
Top flange braced by decking.

LINE LOADS (k/ft):

Load	Dist (ft)	DL	CDL	LL	CLL
1	0.000	0.084	0.084	0.000	0.000
	39.000	0.084	0.084	0.000	0.000
2	0.000	0.837	0.210	0.188	0.075
	39.000	0.837	0.210	0.188	0.075

SHEAR: Max V (DL+LL) = 21.63 kips fv = 1.85 ksi Fv = 19.44 ksi

MOMENTS:

Span	Cond	Moment	@	Lb	Cb	Tension Flange		Compr Flange	
		kip-ft	ft	ft		fb	Fb	fb	Fb
Center	PreCmp+	70.2	19.5	0.0	1.00	3.96	33.00	3.96	33.00
	Max +	210.9	19.5	---	---				
	Mmax/Seff					9.71	33.00	---	---
	Mconst/Sx+Mpost/Seff					10.29	45.00	---	---
Controlling		210.9	19.5	---	---	9.71	33.00	---	---

fc (ksi) = 0.38 Fc = 1.35

REACTIONS (kips):

	Left	Right
Initial reaction	7.20	7.20
DL reaction	17.97	17.97
Max +LL reaction	3.67	3.67
Max +total reaction	21.63	21.63

Gravity Beam Design

RAM SBeam v3.0
Licensed to: Ryan-Biggs Associates

B2- HYBRID TYPE I
LC: D+SDL+L

DEFLECTIONS:

Initial load (in)	at	19.50 ft	=	-0.185	L/D	=	2524
Live load (in)	at	19.50 ft	=	-0.069	L/D	=	6765
Post Comp load (in)	at	19.50 ft	=	-0.300	L/D	=	1560
Net Total load (in)	at	19.50 ft	=	-0.485	L/D	=	964

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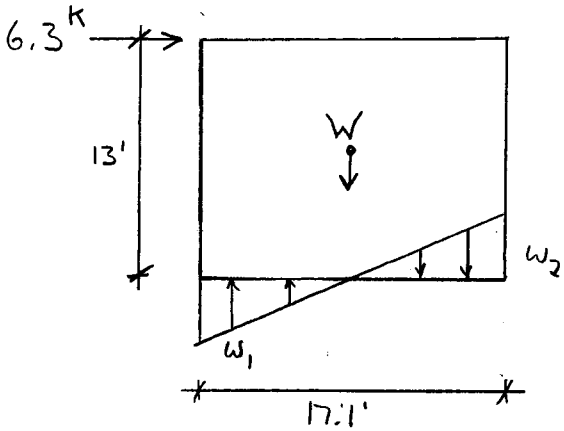
BEAM B2 DESIGN con't w/ DIFFERENT LC

- TYPICAL EDGE BEAMS ALONG COL LINES 1 & 4, 2nd FLOOR
- LOAD COMBO ANALYZED = D+SDL+L+W

BEAM SUPPORTS 2 SHEAR WALLS OF EQUAL DIMENSION



$V = 6.3^K$

$w = 541$ PLF

$S = bd^2/6 = 8(17.1 \times 12)^2/6$

$S = 56143 \text{ IN}^3$

$M = 6.3^K \times 13' = 81.9$ K-FT

$f_b = \frac{M}{S} = \frac{81.9(12)}{56{,}143} = .0175$ ksi

$.0175 \text{ ksi} \times 8 \text{ IN} \times 12 \text{ IN} = \underline{\underline{1.68 \text{ K/FT}}}$

$w_1 = .541 - 1.68 = -1.14$ K/FT

$w_2 = .541 + 1.68 = 2.22$ K/FT

SPAN = 39'
TRIB = 3.75'
F_y = 50 ksi
L_b = 0'-0"
ASD METHOD

D* = 56 PSF × 3.75' = .210 KLF
SDL = 23 PSF × 3.75' = .086 KLF
L = 50 PSF × 3.75' = .188 KLF
WALL WT. = 541 PLF
*ESTIMATE FOR STEEL SUBTRACTED

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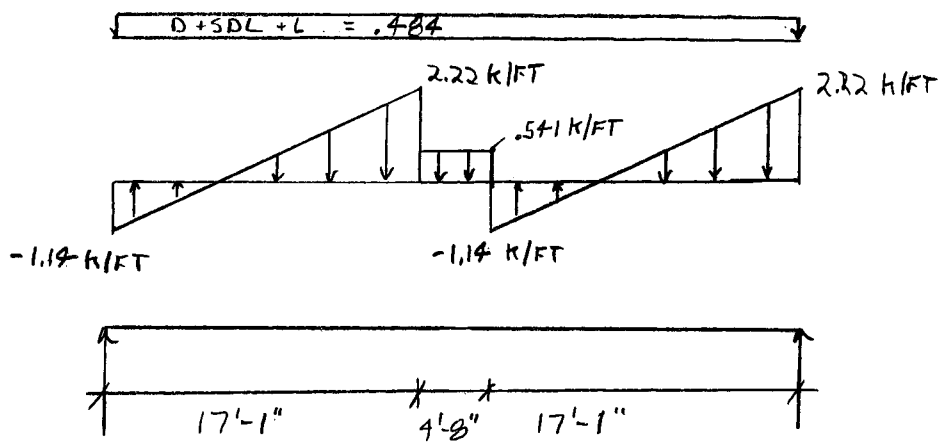
BEAM B2 DESIGN con't w/ DIFFERENT LC

USE: RAM SBEAM TO DESIGN

D = D + SDL = (.210 + .086) = .296 KLF
CONSTR. D = .210 KLF
L = .188 KLF
CONSTR. L = .075 KLF
ADD PARTIAL WALL TO D = .541 KLF

BEAM DIAGRAM



$R_L = 17.2$ K $R_R = 26.6$ K

M = 215.3 K-FT

OPTIMIZED BEAM : W27 x 84 [24] SEE RAM OUTPUT

Gravity Beam Design

RAM SBeam v3.0
Licensed to: Ryan-Biggs Associates

BM B2 - HYBRID TYPE I
LC: D+SDL+L+W

12/04/07 10:31:15

STEEL CODE: ASD 9th Ed.

SPAN INFORMATION (ft): I-End (0.00,0.00) J-End (39.00,0.00)

Beam Size (Optimum) = W27X84 Fy = 50.0 ksi
Total Beam Length (ft) = 39.00
Distance to Adjacent Beam on Left (ft) = 7.5
Distance to Adjacent Edge on Right (ft) = 0.4

COMPOSITE PROPERTIES (Not Shored):

						Left	Right
Concrete thickness (in)						3.50	3.50
Unit weight concrete (pcf)						150.00	150.00
f'c (ksi)						3.00	3.00
Decking Orientation						perpendicular	perpendicular
Decking type						USD 2" Lok-Floor	USD 2" Lok-Floor
beff (in)	=	50.04	Y bar(in)	=	21.00		
Seff (in3)	=	260.68	Str (in3)	=	289.73		
Ieff (in4)	=	4877.72	Itr (in4)	=	6112.69		
Stud length (in)	=	4.00	Stud diam (in)	=	0.75		

Stud Capacity (kips) q = 8.6
# of studs: Full = 53 Partial = 24 Actual = 24
Number of Stud Rows = 1 Percent of Full Composite Action = 36.81
Top flange braced by decking.

LINE LOADS (k/ft):

Load	Dist (ft)	DL	CDL	LL	CLL
1	0.000	0.084	0.084	0.000	0.000
	39.000	0.084	0.084	0.000	0.000
2	0.000	0.296	0.210	0.188	0.075
	39.000	0.296	0.210	0.188	0.075
3	17.160	0.541	0.000	0.000	0.000
	21.160	0.541	0.000	0.000	0.000
4	0.000	-1.140	0.000	0.000	0.000
	17.160	2.220	0.000	0.000	0.000
5	21.160	-1.140	0.000	0.000	0.000
	39.000	2.220	0.000	0.000	0.000

SHEAR: Max V (DL+LL) = 26.01 kips fv = 2.22 ksi Fv = 19.44 ksi

MOMENTS:

Span	Cond	Moment	@	Lb	Cb	Tension Flange		Compr Flange	
		kip-ft	ft	ft		fb	Fb	fb	Fb
Center	PreCmp+	70.2	19.5	0.0	1.00	3.96	33.00	3.96	33.00
	Max +	215.3	16.5	---	---				
	Mmax/Seff					9.91	33.00	---	---
	Mconst/Sx+Mpost/Seff					10.49	45.00	---	---
Controlling		215.3	16.5	---	---	9.91	33.00	---	---

fc (ksi) = 0.39 Fc = 1.35

Gravity Beam Design

BM B2 - HYBRID TYPE I
LC: D+SDL+L+W

REACTIONS (kips):

	Left	Right
Initial reaction	7.20	7.20
DL reaction	13.55	22.35
Max +LL reaction	3.67	3.67
Max +total reaction	17.22	26.01

DEFLECTIONS:

Initial load (in)	at	19.11 ft	=	-0.185	L/D	=	2525
Live load (in)	at	19.11 ft	=	-0.069	L/D	=	6768
Post Comp load (in)	at	19.11 ft	=	-0.296	L/D	=	1582
Net Total load (in)	at	19.11 ft	=	-0.481	L/D	=	973

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BEAM B3 DESIGN

- TYPICAL ALONG COL. LINES A & B, 2nd FLOOR
- LOAD COMBO ANALYZED = D + SDL + L

SPAN = 30'
F_y = 50 ksi
L_b = 7.5'
ASD METHOD

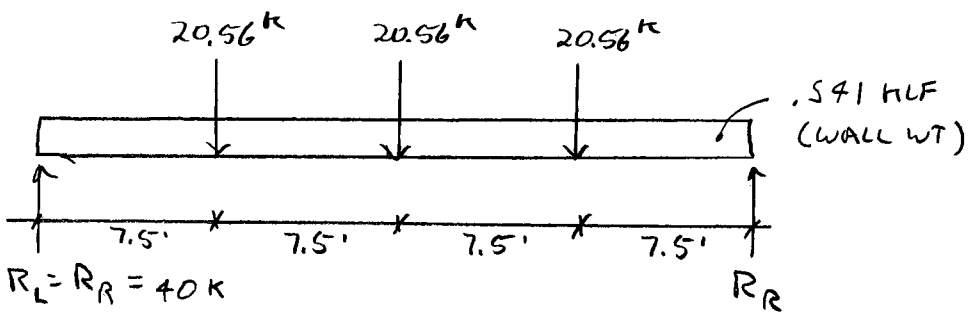
(3) POINT LOADS ; 1@ 7.5', 1@ 15', 1@ 22.5'

D = 13.06 K (FROM RAM OUTPUT FOR B1)
L = 7.5 K (FROM RAM OUTPUT FOR B1)
CONSTR. DL = 64 PSF x 7.5' x $\frac{40'}{2}$ = 9.6 K
CONSTR. LL = 20 PSF x 7.5' x $\frac{40'}{2}$ = 3.0 K

UNIFORM LOAD FROM SUPPORTED WALL = 541 PLF
Δ req'd = $\ell/600$ OR .3", $\ell/600$ = .6" ∴ USE .3"

* SEE B1 FOR ADDITIONAL Δ REQUIREMENTS

BEAM DIAGRAM:



M = 377.0 K-FT

OPTIMIZED BEAM IS: W30x90 [15'] SEE RAM OUTPUT

Gravity Beam Design

PAGE: SS-17

RAM SBeam v3.0 BM B3 - HYBRID TYPE I

Licensed to: Ryan-Biggs Associates LC: D+SDL+L 01/02/08 13:13:56

STEEL CODE: ASD 9th Ed.

SPAN INFORMATION (ft): I-End (0.00,0.00) J-End (30.00,0.00)

Beam Size (Optimum) = W30X90 Fy = 50.0 ksi

Total Beam Length (ft) = 30.00

Distance to Adjacent Beam on Left (ft) = 40.0

Distance to Adjacent Edge on Right (ft) = 0.4

COMPOSITE PROPERTIES (Not Shored):

						Left	**Right**
Concrete thickness (in)						3.50	3.50
Unit weight concrete (pcf)						150.00	150.00
f'c (ksi)						3.00	3.00
Decking Orientation						parallel	parallel
Decking type						USD 2" Lok-Floor	USD 2" Lok-Floor
beff (in)	=	50.04	Y bar(in)	=	23.59		
Seff (in3)	=	290.50	Str (in3)	=	330.92		
Ieff (in4)	=	5851.38	Itr (in4)	=	7842.89		
Stud length (in)	=	4.00	Stud diam (in)	=	0.75		

Stud Capacity (kips) q = 11.5

# of studs: Full = 50 Partial = 15 Actual = 15

Number of Stud Rows = 1 Percent of Full Composite Action = 28.04

Top flange braced by decking.

POINT LOADS (kips):

Dist (ft)	DL	CDL	LL	CLL	Flange Bracing Top	Flange Bracing Bottom
7.500	13.06	9.60	7.50	3.00	Yes	No
15.000	13.06	9.60	7.50	3.00	Yes	No
22.500	13.06	9.60	7.50	3.00	Yes	No

LINE LOADS (k/ft):

Load	Dist (ft)	DL	CDL	LL	CLL
1	0.000	0.090	0.090	0.000	0.000
	30.000	0.090	0.090	0.000	0.000
2	0.000	0.541	0.000	0.000	0.000
	30.000	0.541	0.000	0.000	0.000

SHEAR: Max V (DL+LL) = 40.30 kips fv = 3.03 ksi Fv = 17.85 ksi

MOMENTS:

Span	Cond	Moment kip-ft	@ ft	Lb ft	Cb	Tension Flange fb	Tension Flange Fb	Compr Flange fb	Compr Flange Fb
Center	PreCmp+	199.1	15.0	7.5	1.13	9.75	30.00	9.75	30.00
	Max +	379.4	15.0	---	---				
	Mmax/Seff					15.67	33.00	---	---
	Mconst/Sx+Mpost/Seff					16.85	45.00	---	---
Controlling		379.4	15.0	---	---	15.67	33.00	---	---

fc (ksi) = 0.44 Fc = 1.35

Gravity Beam Design

REACTIONS (kips):

	Left	Right
Initial reaction	20.25	20.25
DL reaction	29.05	29.05
Max +LL reaction	11.25	11.25
Max +total reaction	40.30	40.30

DEFLECTIONS:

Initial load (in)	at	15.00 ft	=	-0.227	L/D	=	1584
Live load (in)	at	15.00 ft	=	-0.102	L/D	=	3528
Post Comp load (in)	at	15.00 ft	=	-0.207	L/D	=	1737
Net Total load (in)	at	15.00 ft	=	-0.435	L/D	=	828

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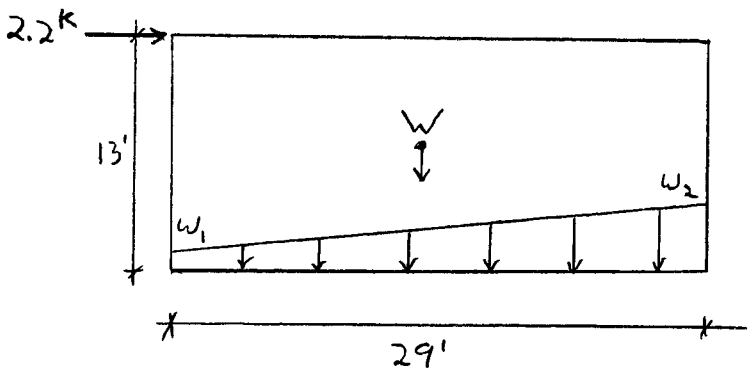
BEAM B3 DESIGN con't w/ DIFFERENT LC

- TYPICAL EDGE BEAM ALONG COL LINES 1 & 4, 2nd FLOOR
- LOAD COMBO ANALYZED = D+SDL+L+W

BEAM SUPPORTS SHEAR WALL



$V = 2.2K$

$w = 541$ PLF

$S = bd^2/6 = 8(29 \times 12)^2/6$

$S = 161472 \text{ IN}^3$

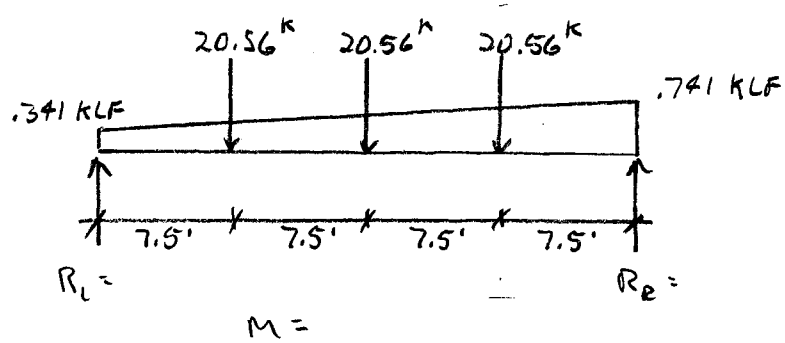
$M = 2.2 \times 13' = 28.6$ K-FT

$F_b = \frac{M}{S} = \frac{28.6(12)}{161472} = .0021$ KSI

$.0021 \text{ ksi} \times 8" \times 12" = .20$ K/FT

$w_1 = .541 - .20$ K/FT $= .341$

$w_2 = .541 + .20$ K/FT $= .741$



OPTIMIZED BEAM IS: W30x90 [15] SEE RAM OUTPUT

Gravity Beam Design

PAGE: SS-20

RAM SBeam v3.0 BM B3-HYBRID TYPE I

Licensed to: Ryan-Biggs Associates LC: D+SDL+L+W 01/02/08 13:19:51

STEEL CODE: ASD 9th Ed.

SPAN INFORMATION (ft): I-End (0.00,0.00) J-End (30.00,0.00)

Beam Size (Optimum) = W30X90 Fy = 50.0 ksi

Total Beam Length (ft) = 30.00

Distance to Adjacent Beam on Left (ft) = 40.0

Distance to Adjacent Edge on Right (ft) = 0.4

COMPOSITE PROPERTIES (Not Shored):

						Left	Right
Concrete thickness (in)						3.50	3.50
Unit weight concrete (pcf)						150.00	150.00
f'c (ksi)						3.00	3.00
Decking Orientation						parallel	parallel
Decking type						USD 2" Lok-Floor	USD 2" Lok-Floor
beff (in)	=	50.04	Y bar(in)	=	23.59		
Seff (in3)	=	290.50	Str (in3)	=	330.92		
Ieff (in4)	=	5851.38	Itr (in4)	=	7842.89		
Stud length (in)	=	4.00	Stud diam (in)	=	0.75		

Stud Capacity (kips) q = 11.5

# of studs: Full = 50 Partial = 15 Actual = 15

Number of Stud Rows = 1 Percent of Full Composite Action = 28.04

Top flange braced by decking.

POINT LOADS (kips):

					Flange Bracing	
Dist (ft)	DL	CDL	LL	CLL	Top	Bottom
7.500	13.06	9.60	7.50	3.00	Yes	No
15.000	13.06	9.60	7.50	3.00	Yes	No
22.500	13.06	9.60	7.50	3.00	Yes	No

LINE LOADS (k/ft):

Load	Dist (ft)	DL	CDL	LL	CLL
1	0.000	0.090	0.090	0.000	0.000
	30.000	0.090	0.090	0.000	0.000
2	0.000	0.341	0.000	0.000	0.000
	30.000	0.741	0.000	0.000	0.000

SHEAR: Max V (DL+LL) = 41.30 kips fv = 3.11 ksi Fv = 17.85 ksi

MOMENTS:

Span	Cond	Moment	@	Lb	Cb	Tension Flange		Compr Flange	
		kip-ft	ft	ft		fb	Fb	fb	Fb
Center	PreCmp+	199.1	15.0	7.5	1.13	9.75	30.00	9.75	30.00
	Max +	379.4	15.0	---	---				
	Mmax/Seff					15.67	33.00	---	---
	Mconst/Sx+Mpost/Seff					16.85	45.00	---	---
Controlling		379.4	15.0	---	---	15.67	33.00	---	---

fc (ksi) = 0.44 Fc = 1.35

Gravity Beam Design

REACTIONS (kips):

	Left	Right
Initial reaction	20.25	20.25
DL reaction	28.05	30.05
Max +LL reaction	11.25	11.25
Max +total reaction	39.30	41.30

DEFLECTIONS:

Initial load (in)	at	15.00 ft	=	-0.227	L/D	=	1584
Live load (in)	at	15.00 ft	=	-0.102	L/D	=	3528
Post Comp load (in)	at	15.00 ft	=	-0.207	L/D	=	1737
Net Total load (in)	at	15.00 ft	=	-0.435	L/D	=	828

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BEAM R1 DESIGN

- TYPICAL INTERIOR BAY
- LOAD COMBO ANALYZED = D + SDL + SL

SPAN = 40'
TRIB = 7.5'
F_y = 50 ksi
L_b = 0'-0"
ASD METHOD

D* = 44 PSF
SDL = 18 PSF
SL = 40 PSF
* SUBTRACTED 7 PSF ESTIMATE OF STL WT - LET RAM CALCULATE.

USE : RAM STRUCTURAL SYSTEM

D = D + SDL = 62 PSF
CONSTR. D = 44 PSF
SL = 40 PSF
CONSTR. L = 20 PSF

DEFLECTION : LIMIT PRE-COMPOSITE CROSS BAY Δ TO SMALLER OF ℓ/360 OR 1" PER RBA TECH NOTE = $\ell/360 = \frac{50' \times 12}{360} = 1.67"$ ∴ USE 1.0"

DEFAULTS:	L/d	Δ
Initial	0.00	0.75"
Post LL	360	0.00
Post Super.	360	0.00
NET TOTAL	240	0.00

OPTIMIZED BEAM = W24x55 [20] , Δ = .9" (TOTAL)
Δ = .56" (INITIAL)

INITIAL Δ ALLOWABLE FOR GIRDER R3 = 1.0 - .56" = .44"

Gravity Beam Design

PAGE: SS-23

RAM Steel v11.0
Hybrid Type 1
DataBase: Office Bld Typ1
Building Code: IBC

BM R1 - HYBRID TYPE I
LC: D + SDL + SL

12/04/07 18:42:02
Steel Code: ASD 9th Ed.

Floor Type: Roof **Beam Number = ~~1~~**

SPAN INFORMATION (ft): I-End (15.00,0.00) J-End (15.00,40.00)

Beam Size (Optimum) = W24X55 Fy = 50.0 ksi
Total Beam Length (ft) = 40.00

COMPOSITE PROPERTIES (Not Shored):

		Left			Right
Concrete thickness (in)		2.50			2.50
Unit weight concrete (pcf)		150.00			150.00
f'c (ksi)		3.00			3.00
Decking Orientation		perpendicular			perpendicular
Decking type		USD 2" Lok-Floor			USD 2" Lok-Floor
beff (in)	= 90.00		Y bar(in)	=	21.02
Seff (in3)	= 146.40		Str (in3)	=	172.27
Ieff (in4)	= 2607.27		Itr (in4)	=	3634.71
Stud length (in)	= 3.50		Stud diam (in)	=	0.75

Stud Capacity (kips) q = 8.6
# of studs: Full = 56 Partial = 20 Actual = 20
Number of Stud Rows = 1 Percent of Full Composite Action = 30.06

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	CLL
1	0.000	0.465	0.330	0.300	---	NonR	0.150
	40.000	0.465	0.330	0.300			0.150
2	0.000	0.055	0.055	0.000	---	NonR	0.000
	40.000	0.055	0.055	0.000			0.000

SHEAR: Max V (DL+LL) = 16.41 kips fv = 1.84 ksi Fv = 18.78 ksi

MOMENTS:

Span	Cond	Moment	@	Lb	Cb	Tension Flange		Compr Flange	
		kip-ft	ft	ft		fb	Fb	fb	Fb
Center	PreCmp+	107.1	20.0	0.0	1.00	11.17	33.00	11.17	33.00
	Max +	164.1	20.0	---	---				
	Mmax/Seff					13.45	33.00	---	---
	Mconst/Sx+Mpost/Seff					15.18	45.00	---	---
Controlling		164.1	20.0	---	---	13.45	33.00	---	---

fc (ksi) = 0.23 Fc = 1.35

REACTIONS (kips):

	Left	Right
Initial reaction	10.71	10.71
DL reaction	10.41	10.41
Max +LL reaction	6.00	6.00
Max +total reaction	16.41	16.41

DEFLECTIONS:

Initial load (in)	at	20.00 ft	=	-0.563	L/D =	853
Live load (in)	at	20.00 ft	=	-0.229	L/D =	2100

Post Comp load (in)	at	20.00 ft	=	-0.331	L/D	=	1448
Net Total load (in)	at	20.00 ft	=	-0.894	L/D	=	537

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BEAM R2 DESIGN

- TYPICAL EDGE BEAMS ALONG COL LINES 1 & 4, ROOF
- LOAD COMBO ANALYZED = D + SDL + SL

SAME DESCRIPTION AS R1 EXCEPT TRIB WILL BE ABOUT ½ (7.5')

USE: RAM STRUCTURAL SYSTEM
SAME INPUTS AS R1

DEFLECTION:
SAME AS DEFAULTS FOR R1

OPTIMIZED BEAM = W21 x 44 [20] SEE RAM OUTPUT

Gravity Beam Design

PAGE: SS-26

RAM Steel v11.0
Hybrid Type 1
DataBase: Office Bld Typ1
Building Code: IBC

BM R2 - HYBRID TYPE I
LC: D+SDL+SL

12/04/07 18:42:02
Steel Code: ASD 9th Ed.

Floor Type: Roof **Beam Number =X**

SPAN INFORMATION (ft): I-End (0.00,0.00) J-End (0.00,40.00)

Beam Size (Optimum) = W21X44 Fy = 50.0 ksi
Total Beam Length (ft) = 40.00

COMPOSITE PROPERTIES (Not Shored):

			Left		Right
Concrete thickness (in)			2.50		2.50
Unit weight concrete (pcf)			150.00		150.00
f'c (ksi)			3.00		3.00
Decking Orientation			perpendicular		perpendicular
Decking type			USD 2" Lok-Floor		USD 2" Lok-Floor
beff (in)	=	51.00	Y bar(in)	=	17.54
Seff (in3)	=	109.87	Str (in3)	=	120.41
Ieff (in4)	=	1774.99	Itr (in4)	=	2122.50
Stud length (in)	=	3.50	Stud diam (in)	=	0.75

Stud Capacity (kips) q = 8.6
# of studs: Full = 38 Partial = 20 Actual = 20
Number of Stud Rows = 1 Percent of Full Composite Action = 53.05

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	CLL
1	0.000	0.031	0.022	0.020	---	NonR	0.010
	40.000	0.031	0.022	0.020			0.010
2	0.000	0.232	0.165	0.150	---	NonR	0.075
	40.000	0.232	0.165	0.150			0.075
3	0.000	0.044	0.044	0.000	---	NonR	0.000
	40.000	0.044	0.044	0.000			0.000

SHEAR: Max V (DL+LL) = 9.55 kips fv = 1.38 ksi Fv = 18.99 ksi

MOMENTS:

Span	Cond	Moment	@	Lb	Cb	Tension Flange		Compr Flange	
		kip-ft	ft	ft		fb	Fb	fb	Fb
Center	PreCmp+	63.2	20.0	0.0	1.00	9.30	33.00	9.30	33.00
	Max +	95.5	20.0	---	---				
	Mmax/Seff					10.44	33.00	---	---
	Mconst/Sx+Mpost/Seff					12.19	45.00	---	---
Controlling		95.5	20.0	---	---	10.44	33.00	---	---

fc (ksi) = 0.24 Fc = 1.35

REACTIONS (kips):

	Left	Right
Initial reaction	6.32	6.32
DL reaction	6.15	6.15
Max +LL reaction	3.40	3.40
Max +total reaction	9.55	9.55

Gravity Beam Design

RAM Steel v11.0
Hybrid Type 1
DataBase: Office Bld Typ1
Building Code: IBC

BM R2 - HYBRID TYPE I
LC: D+SDL+SL

12/04/07 18:42:02
Steel Code: ASD 9th Ed.

DEFLECTIONS:

Initial load (in)	at	20.00 ft	=	-0.545	L/D	=	881
Live load (in)	at	20.00 ft	=	-0.190	L/D	=	2523
Post Comp load (in)	at	20.00 ft	=	-0.276	L/D	=	1740
Net Total load (in)	at	20.00 ft	=	-0.821	L/D	=	585

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BEAM B3 DESIGN

- TYPICAL ALONG COL LINES A & B, ROOF
- LOAD COMBO ANALYZED = D + SDL + SL

SPAN = 30'
F_y = 50 ksi
L_b = 7.5'
ASD METHOD

SAME LOADS AS B1
USE RAM STRUCTURAL SYSTEM

DEFLECTION:
INITIAL ALLOWABLE Δ = .44" (SEE B1 FOR EXPLANATION)

OPTIMIZED BEAM = W24x76 [15], SEE RAM OUTPUT

Gravity Beam Design

PAGE: 55-29

RAM Steel v11.0
DataBase: Office Bld Typ1
Building Code: IBC

BM R3 - HYBRID TYPE I
LC: D+SDL+SL

01/02/08 14:03:52
Steel Code: ASD 9th Ed.

Floor Type: Roof **Beam Number = 8**

SPAN INFORMATION (ft): I-End (0.00,40.00) J-End (30.00,40.00)

Beam Size (User Selected) = W24X76 Fy = 50.0 ksi
Total Beam Length (ft) = 30.00

COMPOSITE PROPERTIES (Not Shored):

			Left		Right
Concrete thickness (in)			2.50		2.50
Unit weight concrete (pcf)			150.00		150.00
f'c (ksi)			3.00		3.00
Decking Orientation			parallel		parallel
Decking type			USD 2" Lok-Floor		USD 2" Lok-Floor
beff (in)	=	51.00	Y bar(in)	=	18.90
Seff (in3)	=	209.09	Str (in3)	=	231.63
Ieff (in4)	=	3466.34	Itr (in4)	=	4397.39
Stud length (in)	=	3.50	Stud diam (in)	=	0.75

Stud Capacity (kips) q = 11.5
# of studs: Full = 40 Partial = 15 Actual = 15
Number of Stud Rows = 1 Percent of Full Composite Action = 35.37

POINT LOADS (kips):

Dist	DL	CDL	RedLL	Red%	NonRLL	StorLL	Red%	RoofLL	Red%	CLL
7.500	10.41	7.71	0.00	0.0	6.00	0.00	0.0	0.00	Snow	3.00
15.000	10.41	7.71	0.00	0.0	6.00	0.00	0.0	0.00	Snow	3.00
22.500	10.41	7.71	0.00	0.0	6.00	0.00	0.0	0.00	Snow	3.00

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	CLL
1	0.000	0.031	0.022	0.020	---	NonR	0.010
	30.000	0.031	0.022	0.020			0.010
2	0.000	0.076	0.076	0.000	---	NonR	0.000
	30.000	0.076	0.076	0.000			0.000

SHEAR: Max V (DL+LL) = 26.52 kips fv = 2.52 ksi Fv = 20.00 ksi

MOMENTS:

Span	Cond	Moment	@	Lb	Cb	Tension Flange		Compr Flange	
		kip-ft	ft	ft		fb	Fb	fb	Fb
Center	PreCmp+	172.8	15.0	7.5	1.13	11.78	33.00	11.78	33.00
	Max +	260.4	15.0	---	---				
	Mmax/Seff					14.95	33.00	---	---
	Mconst/Sx+Mpost/Seff					16.31	45.00	---	---
Controlling		260.4	15.0	---	---	14.95	33.00	---	---

fc (ksi) = 0.39 Fc = 1.35

REACTIONS (kips):

	Left	Right
Initial reaction	17.69	17.69
DL reaction	17.22	17.22

Gravity Beam Design

RAM Steel v11.0
DataBase: Office Bld Typ1
Building Code: IBC

BM R3-HYBRID TYPE I
LC: D +SDL +SL

01/02/08 14:03:52
Steel Code: ASD 9th Ed.

	Left	Right
Max +LL reaction	9.30	9.30
Max +total reaction	26.52	26.52

DEFLECTIONS:

Initial load (in)	at	15.00 ft	=	-0.322	L/D	=	1119
Live load (in)	at	15.00 ft	=	-0.141	L/D	=	2546
Post Comp load (in)	at	15.00 ft	=	-0.205	L/D	=	1756
Net Total load (in)	at	15.00 ft	=	-0.527	L/D	=	684

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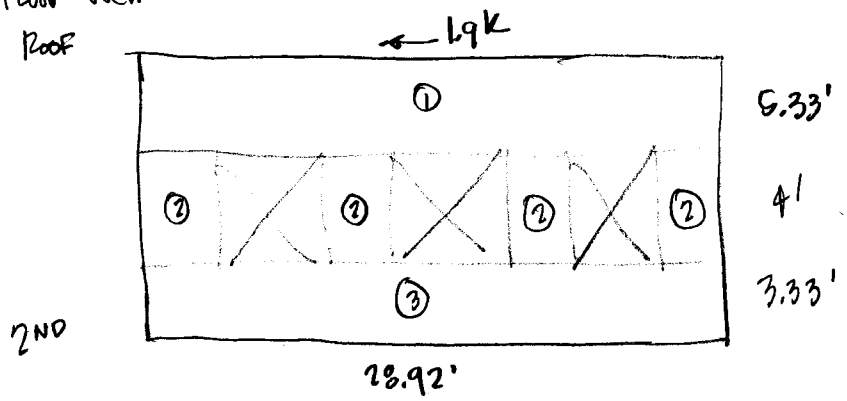
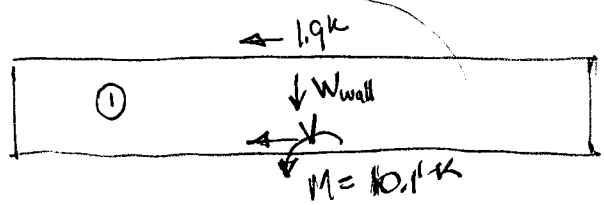
FROM p. L9 - NORTH WALL

TRY $f'_m = 1500$ psi

2ND FLOOR WALL



①



$f = \frac{P}{A} \pm \frac{M}{S} \Rightarrow$ $P = W_{wall} = 44\text{psf} \times 28.92(5.33) \doteq 6.8^k$

$A = 28.92' \times (40.7 in^2/ft) \doteq 1177 in^2$

ASSUME GROUTED @ 48" ⇒ NCMA TEK 14-13

b = net width $\frac{40.7 in^2/ft}{12} \doteq 3.39$ in

$S = \frac{bl^2}{6} = \frac{3.39 \times (28.92 \times 12)^2}{6} \doteq 68047 in^3$

(D+W) $\therefore f = \frac{6.8^k}{1177} \pm \frac{10.1'^k \times 12}{68047} \doteq 5.78 \pm 1.78 \Rightarrow$ 7.6 psi (max), 4.0 psi (min)

total compression .. ∴ no steel req'd

(.6D+W) $f = 3.5 \pm 1.8 \Rightarrow$ still no tension

$f_v = \frac{1900^\#}{1177 in^2} \doteq 1.6$ psi ∴ no shear reinforcement needed

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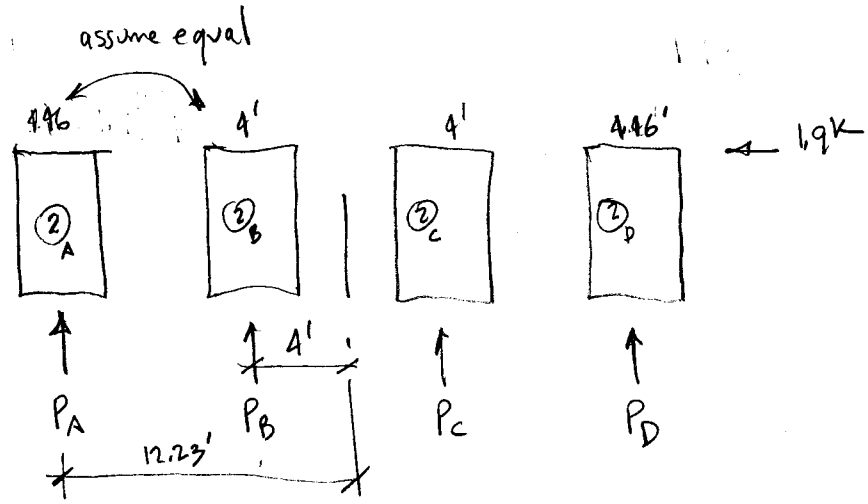
PROJECT NUMBER: 9128 DATE: 12/26/07 PAGE: L12

PROJECT: HYBRID - TYPE I BY: DTB

SUBJECT: SHEAR WALLS

(2)



$P_A(12.23) + P_B(4') - P_C(4) - P_D(12.33) = M = 1.9^k(5.33+4) = 17.7'^{-k}$

$\Sigma P = 6.8^k + \underbrace{(4.46' + 4')2(44psf) \times 4' high}_{\sim 3.0^k} = 9.8^k$

$12.23(P_A - P_D) + 4'(P_B - P_C) = M$

$P = \frac{\Sigma P}{n} \pm \frac{My_n}{I_n} = \frac{9.8^k}{4} \pm \frac{17.7\,y_n}{331} = 2.45 \pm \overbrace{\frac{17.7(12.23')}{331}}^{.65}$ $\quad 3.1^k = P_A$, $1.8^k = P_D$

$\Sigma Ad^2 = 2(12.23)^2 + 2(4)^2 = 331$

$2.45 \pm \underbrace{\frac{17.7(4)}{331}}_{.21}$ $\quad 2.66 = P_B$, $2.24 = P_C$

$\Sigma P = 9.8$ ✓

check $M = \underbrace{(3.1 - 1.8)12.23}_{15.9} + \underbrace{(2.66 - 2.24)}_{1.7} = 17.6'^{-k}$ ✓

$M = \frac{Vh}{2} = .5^k\frac{(4')}{2} = 1'^{-k}$

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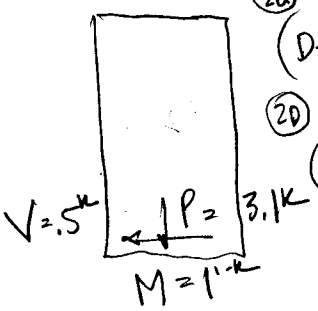
PROJECT NUMBER: 9128 DATE: 12/26/07 PAGE: L13

PROJECT: HYBRID - TYPE I BY: DMD

SUBJECT: SHEAR WALLS



(2a) (D+W) $f_b = \frac{P}{A} + \frac{M}{S}$: $f_b = \frac{3.1^k}{4.46'(40.7 in^2)} + \frac{1'^k \times 12}{3.39\left(\frac{4.46 \times 12}{6}\right)^2} \doteq 17.0 \pm 7.4$ ∴ no tension

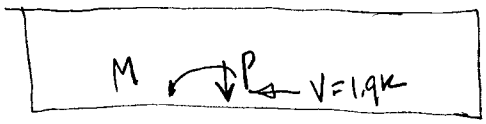
(2b) (.6D+W) $f_b = \frac{.6(1.8)}{(\ldots)} \pm 7.4 = 5.9 - 7.4 \Rightarrow$ tension

$A_s \cong \frac{(M)}{f_s j d} \doteq .028'' \Rightarrow$ USE #4 as minimum..

NO NEED FOR EXACT ANALYSIS

$f_v = \frac{0.5^k}{4.46(40.7)} \doteq 2.8$ psi (MINIMAL)

(3)



$M = 1.9(12.67') = 24.1'^k$

$P = [\underbrace{28.92(12.67)' - 4(4' \times 4')}_{302.4\ sf}]\ 44psf \doteq 13.3^k$

(D+W) $f_b = \frac{P}{A} \pm \frac{M}{S} = \frac{13.3}{117} \pm \frac{24.1 \times 12}{68047} = 11.3 \pm 4.3 \Rightarrow$ 15.6 psi (max) ← see (1) 7.0 psi (min)

(.6D+W) $f_b = 6.8 \pm 4.3$ .. still no tension ∴ no reinforcement needed

$f_v = \frac{1900}{28.92(40.7)} \doteq 1.6$ psi minimal

∴ ONLY PRESCRIPTIVE REINF. NEEDED – 2ND FLOOR

#4 @ JAMBS

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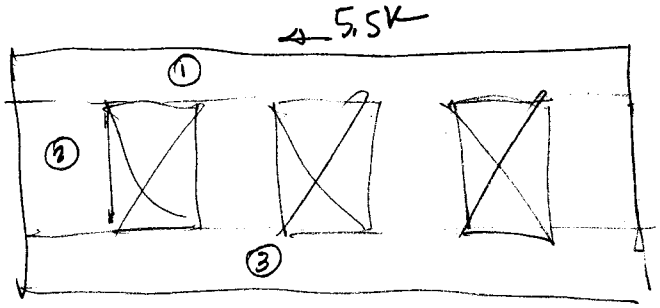
PROJECT NUMBER: 9128 DATE: 12/26/07 PAGE: L14

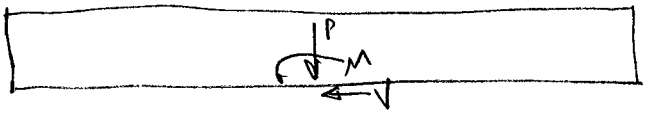
PROJECT: HYBRID - TYPE I BY: DTB

SUBJECT: SHEAR WALLS

1ST FLOOR



1



(D+W) $f_b = \frac{P}{1177} \pm \frac{M}{68047} = \frac{6.8^k}{1177} \pm \frac{6.8(5.33)}{68047} \times 12 \doteq 5.8 \pm 6.4$ psi

SEE 2ND FLOOR

∴ TENSION ∴ CRACKED SECTION ⇓ check bars from below

$f_v = \frac{5500}{28.92(40.7)} \doteq 4.7$ psi minimal

(.6D+W) $f_b \Rightarrow$ increases tension.

From 2 #4 bars ⇒ $M = A_s f_s j d$

$.20\frac{(24)}{12}(.9)(\quad) \doteq 122'^k \gg$ req'd OK

~ 340^k estimated

so far over capacity, no need for exact check

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(2) $P = \frac{\Sigma P}{n} \pm \frac{M y_n}{I_n} = \frac{9.8}{4} \pm \frac{\overbrace{5.5(9.33')}^{51.31^k}}{331} y_n \doteq 2.45 \pm 1.9^k \Rightarrow P_A = 4.4^k$, $P_D = .5^k$

SEE 2ND PAGE

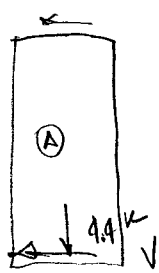
y_n: 12.23' → 4.0

$\Rightarrow 2.45 \pm .62 \rightarrow P_B = 3.1^k$, $P_C = 1.8^k$

Check: $\Sigma P = 9.8^k$ ✓

$\Sigma M = (4.4 - .5)\,12.23 + (3.1 - 1.8)\,4 \doteq 47.7 + 5.2 = 52.9^{1\text{-}k}$ ✓

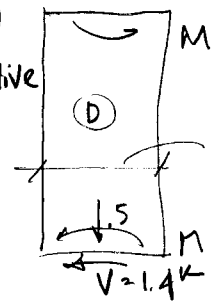
(D+L+W)



$M = \frac{Vh}{2} = 1.4(4')/2 \doteq 2.8^{1\text{-}k}$

$V = 5.5^k/4 = 1.4^k$

(.6D+W) 0 .. conservative



$4.46' \times 12 = 53.5''$

$d \sim 48''$

$f_b = \frac{4.4}{4.46(40.7)} \pm \frac{2.8 \times 12}{3.39\frac{(4.46 \times 12)^2}{6}} \doteq 24.2 \pm 20.8$ psi

∴ no tension

$F_b = \sim \frac{1}{3} f_m = 500$ psi OK

$f_v = \frac{1400^\#}{3.39(48)} \doteq 8.6$ psi minimal

IGNORE LOW AXIAL:

use $A_s = \frac{M}{f_s j d} = \frac{2.8 \times 12}{24(.9)\,48}$

$A_s = .03\,\square''$

.9 est.

minimal use #4
(all jamb steel)

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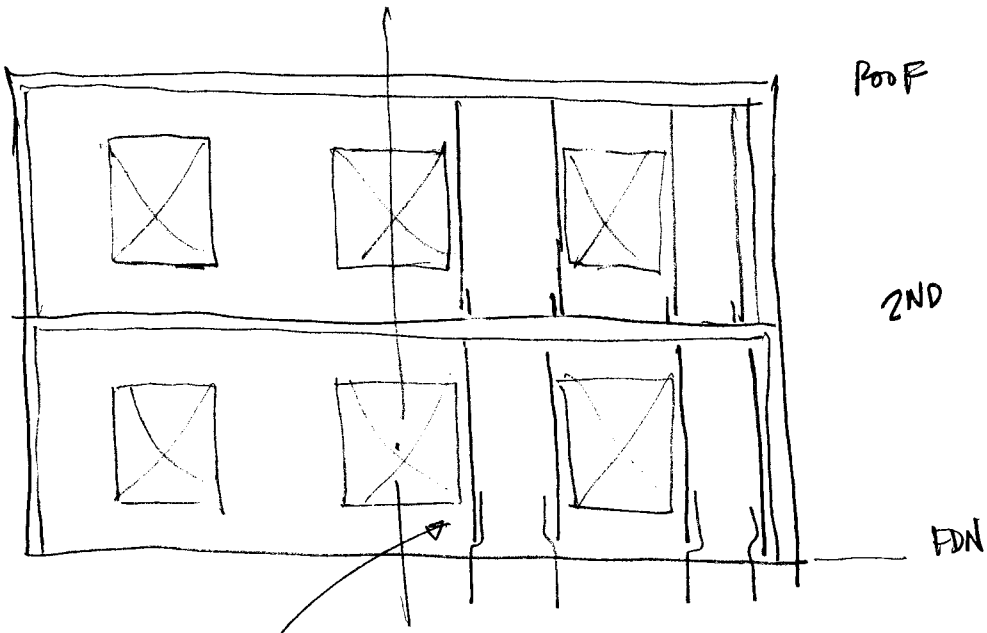
(3)

$M = 5.5(12.67) \doteq 69.7^{1\text{-}k}$

$f_b = \frac{13.3}{1177} \pm \frac{69.7 \times 12}{68097} = 11.3 \pm 12.3$

SEE 2ND floor

minimal on comp. & tension use #4 from (2)



IN-PLANE SHEAR

#4 @ each jamb & edge (lap to foundation at 1ST Floor)

#4 @ each jamb & edge (dowel to slab at 2ND Floor)

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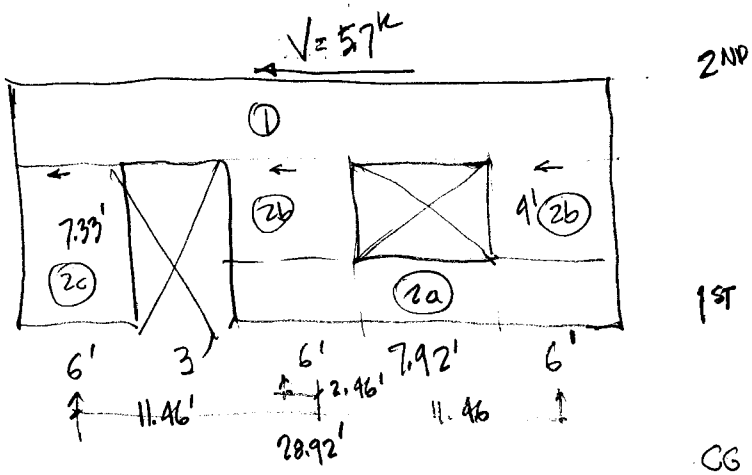
FROM L9 - NORTH WALL (FIRST FLOOR)



CG of vertical from ℄ WALL:

$$\frac{6(11.46)+6(2.46)-6(11.46)}{18} = .82'$$

① .. Similar to ① for 1ST Floor p. L10

② $V_{2c} = \frac{K_{2c}}{\Sigma K}(V) = \frac{.182}{.773}(V) = 1.34^k$

p. L5

$V_{2b} = \frac{V - V_{2c}}{2} \doteq 2.2^k$

$V_{2a} = 4.4^k$

$[28.92(12.67) - (7.33 \times 3) - 4(7.92)]4t \doteq 13.8^k$

$5.7(12.67) \doteq 72.2'^k$

Distribute P:
(use loads @ bottom)

$$P = \frac{\Sigma P}{n} \pm \frac{M y_n}{I_n} = \frac{13.8^k}{3} \pm \frac{72.2}{267} y_n$$

3 $\quad \Sigma A d^2$

$1(11.46-.82)^2 + 1(2.46-.82)^2 + 1(11.46+.82)^2 = 113.2 + 2.7 + 150.8 = 266.7$

$y_n = 10.46'$ for 2c ⇒ 4.6 + 2.8 = 7.4
$y_n = 1.64$ middle 2b ⇒ 4.6 + .4 = 5.0
$y_n = 12.28$ edge 2b 4.6 − 3.3 = 1.3

$\Sigma P = 13.7$ ✓

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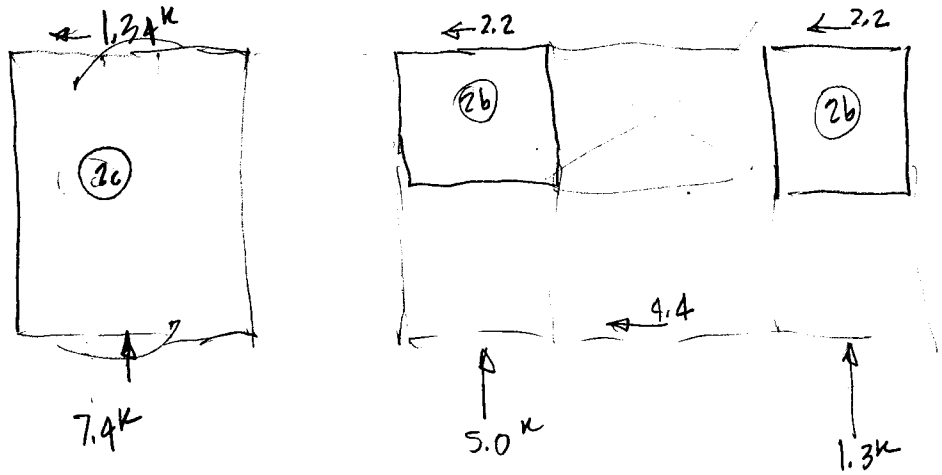
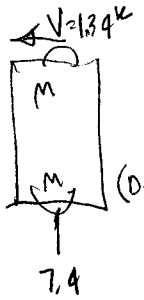


(D+W)

2c



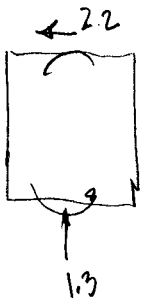
$M = 1.34 \times \frac{7.33}{2} = 4.9^{1\text{-}K}$

$(D+W)\ f_b = \frac{7.4}{6(40.7)} \pm \frac{4.9 \times 12^{\circ}}{3.39\left(\frac{72}{6}\right)^2} \doteq 30.3 \pm 20.1$ compression only

$(.6D+W)\ f_b = 18.2 \pm 20.1$ slight tension .. use minimum jamb steel

$f_v = \frac{1.34}{6(40.7)} \doteq 5.5$ psi minimal

2b



$M = 2.2(4)/2 = 4.4^{1\text{-}K}$

$(.6D+W)\ f_b = \frac{.6(1300)}{6(40.7)} \pm \frac{4.4 \times 12}{3.39\left(\frac{72}{6}\right)^2} \doteq 3.2 \pm 18.0$ .. tension

IGNORE AXIAL: $A_s \cong \frac{M}{f_s jd} = \frac{4.4 \times 12}{24 \times .9(64)} \doteq .040''$ #4 OK

3 $f_v = \frac{4.4^K}{19.92(40.7)} \doteq 5.4$ psi MINIMAL

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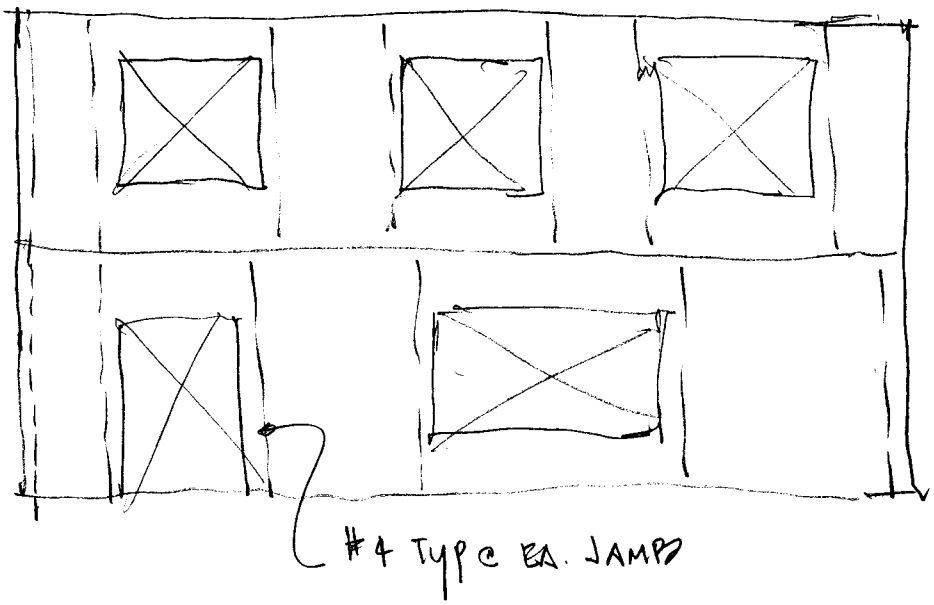
PROJECT NUMBER: 9128 DATE: 12/27/07 PAGE: L19

PROJECT: HYBRID - TYPE I BY: DAB

SUBJECT: SHEAR WALLS

NORTH WALL



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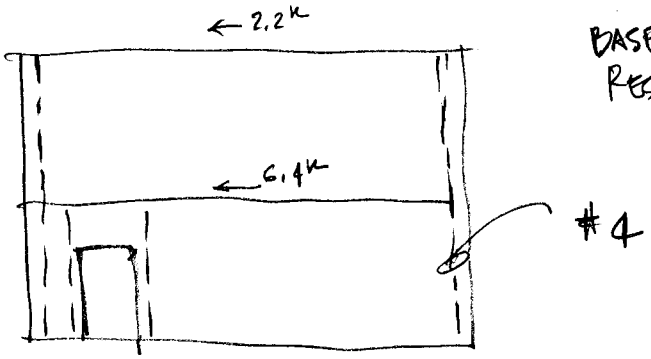
SOUTH WALL



BASED UPON N. WALL RESULTS, MIN. REINF REQ'D

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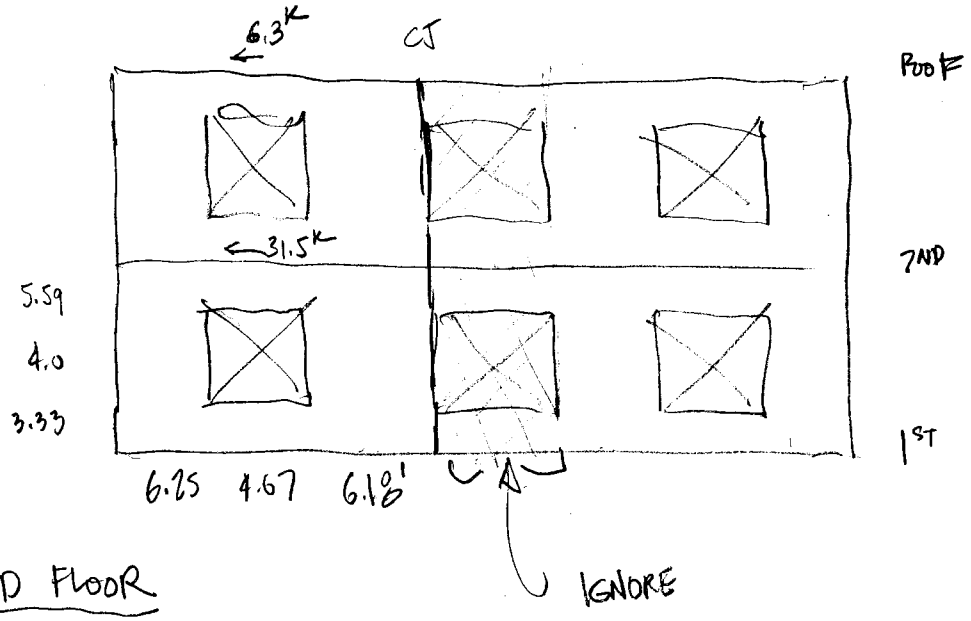
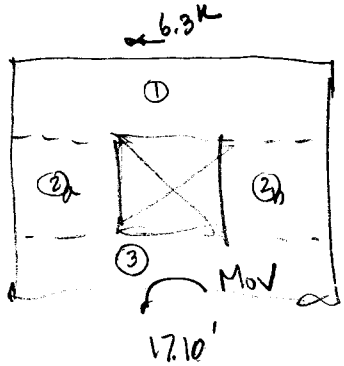
WEST WALL (EAST WALL SIMILAR)



SECOND FLOOR



OVERTURNING

$M_{OV} = 6.3 \times 12.92' \doteq 81.4'\text{-}K$

$P = 44\,psf\left[(17.1 \times 12.92) \div 4(4.67)\right]$

$P \doteq 8.9K$

$\frac{M_{OV}}{P \times \frac{l}{2}} = \frac{81.4}{8.9\left(\frac{17.1}{2}\right)} \doteq 1.07$ ∴ tension expected

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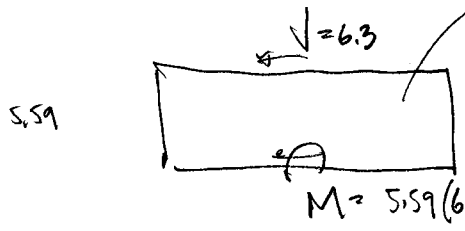


$P = 44(5.59)(17.1) \doteq 4.2^k$

(0.6D+W) $f_b = \frac{P}{A} \pm \frac{M}{S} = \frac{.6(4.2^k)}{17.1(40.7)} \pm \frac{35.2 \times 12}{3.39\left(\frac{17.1 \times 12)^2}{6}\right)}$

$f_b = 2.5 \pm 17.8$ psi

∴ tension .. use cracked analysis

$f_v = \frac{6.3}{17.1 \times 40.7} \doteq 9.1$ psi MINIMAL

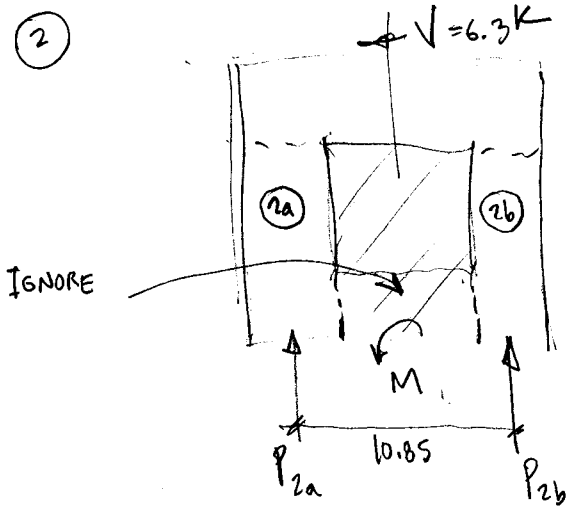
IGNORE AXIAL:

$A_s \sim \frac{M}{f_s j d} \doteq \frac{35.2 \times 12}{24(.9)(197)} \doteq 0.10 \square''$

17.1' - 8"

#4 ok

(2)



$[12.92(17.1) - (4.67 \times 7.33)]44 \doteq 8.2^k$

$6.3 \times 12.92 \doteq 81.4$

$P_x = \frac{\Sigma P}{2} \pm \frac{M}{10.85} \doteq 4.1^k \pm 7.5^k$ ∴ uplift

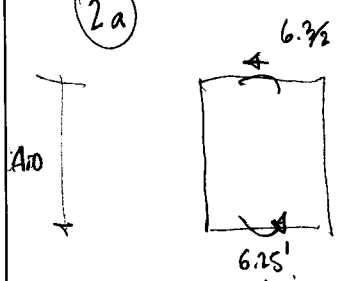
$P_{2a} = 11.6^k$

$P_{2b} \doteq -3.4^k$ (tension)

FOR (.6D+W), ⇒ TENSION INCREASES

$P_{(2b)} = .6(4.1) - 7.5 = -5.0^k$

(2a)



$M = \frac{Vh}{2} = 3.15(4/2) \doteq 6.3^k$

$f_b = \frac{P}{A} \pm \frac{M}{S} = \frac{10.7}{6.25(40.7)} \pm \frac{6.3 \times 12}{3.39\left(\frac{6.25 \times 12)^2}{6}\right)}$

$P = P_{2a} - \frac{3.33(6.25)}{.9}44 = 10.7^k$

(D+W) $f_b = 42 \pm 23.8$ compression only

(.6D+W) ~ $f_b = 25.2 \pm 23.8$

No tension

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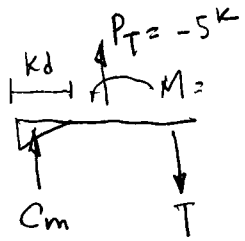
(26)

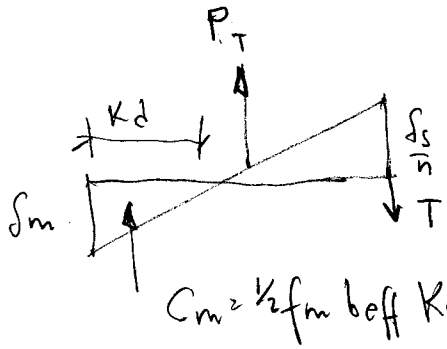


TRY $A_{s_T} = \frac{5^K}{24} \doteq 0.21\,\square^{\prime\prime}$

$V_h/2 = 3.15\left(\frac{4'}{2}\right) = 6.3^{1\text{-}K}$

$A_{s_{FLEXURE}} = \frac{M}{f_s j d} = \frac{6.3 \times 12}{24(.9)(64)} \doteq 0.05\,\square^{\prime\prime}$ (72 - 8")

$\Sigma A_s = .260^{\square} \rightarrow$ try #5 $(0.31\,\square^{\prime\prime})$



$\frac{f_m}{Kd} = \frac{\frac{f_s}{n}}{(d - Kd)} \Rightarrow f_s \; n f_m \left(\frac{d - Kd}{Kd}\right)$

$\therefore \; \frac{1}{2} f_m \underbrace{b_{eff}}_{3.39''} Kd + 5000 = \underbrace{A_s}_{0.31} \left[\overbrace{n}^{21.5} f_m \left(\frac{64 - Kd}{Kd}\right)\right]$

if Kd = 12" $\quad 20.34 f_m + 5000 = 28.88 f_m \Rightarrow$ $f_m = 585$ psi, $f_s = 54537$ psi } NG

if Kd = 8" $\quad 13.56 f_m + 5000 = 46.66 f_m \rightarrow$ $f_m = 151$ psi, $f_s = 22738$ psi

check M: $M = \underbrace{\underbrace{C_m}_{20.34 f_m} \times \underbrace{\left[\frac{72}{2} - \frac{Kd}{3}\right]}_{33.33}}_{3071} + \underbrace{\underbrace{T}_{7049}\underbrace{(64 - 36)}_{28}}_{197372} \doteq 200443^{\square\text{-}\#} \doteq 16.7^{1\text{-}K} > 6.3$ OK

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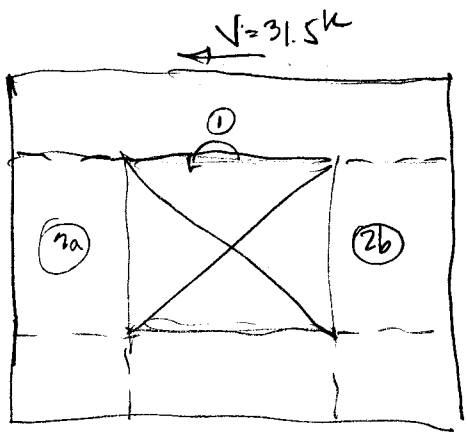
$f_v = 3.15^k / 6.25(40.7) \doteq 12.4$ psi (nominal)

③ check $f_v = \frac{6.3}{17.1 \times 40.7} \doteq 9.0$ psi (nominal)

$f_b = 42 + 23.8$ psi @ 2a $\Rightarrow$ 65.8 psi $\ll \frac{f'_m}{3} = 500$ psi OK

FIRST FLOOR .. EXPECT TENSION



$M_{①} = 31.5(5.59') \doteq 176.1^{1-k}$

try: $A_s = \frac{M}{f_s jd} = \frac{176.1 \times 12}{24(.9)(197)} \doteq .500''$

see 2ND Floor

$f_v = \frac{31.5}{17.1(40.7)} \doteq 45.3$ psi

① $\frac{M}{Vd} = \frac{176.1 \times 12}{31.5(197)} \doteq .34 \Rightarrow F_v = \frac{1}{3}\left[\left(4 - \frac{M}{Vd}\right)\sqrt{f'_m}\right] = 47.3$ psi (D+W)

1500 (MSJC 2.3.5.2.3)

$F_v > f_v$ ∴ no shear reinforcement req'd

Flexure: Try 2-#6 $A_s = 0.88\ in^2 > 0.50$

$np \Rightarrow n = \frac{E_s}{E_m} = \frac{29 \times 10^6}{900 \times 1500} \doteq 21.5$ $\quad np = 21.5\frac{(.88)}{3.39(190)} \doteq 0.029$

beffective; reduced for 2 bars

$k = \sqrt{np^2 + 2np} - np \doteq .214$; $j = 1 - \frac{k}{3} = .929$

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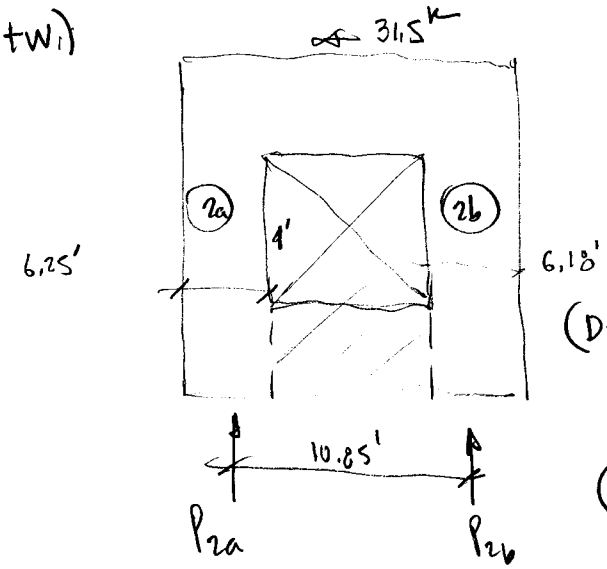
$Kd = .214(190) \doteq 40.7'' < P_{2a} = 6.25'$ OK

$f_m = \frac{2M}{Kjbd^2} \doteq \frac{2 \times 176.1}{.929(.214)(3.39)(190)^2} \times 12000 \doteq 174\,psi \ll \frac{f'_m}{3} = 500\,psi$ OK

∴ use 2-#6 @ ea end

use M @ bottom of wall = 31.5(12.92) = 407.0'-k

(2) (D+W₁)



SEE 1st FLOOR

31.5 × 12.92

$P_x = \frac{\Sigma P}{2} \pm \frac{M}{10.85} \doteq 4.1 \pm \frac{407}{10.85}$

$P_x = 4.1 \pm 37.5^k$

(D+W) $P_{(2a)} = 41.6^k$

$P_{(2b)} \doteq -33.4^k$ TENSION (P_T)

(.6D+W) $P_{(2b)} = 4.1(.6) - 37.5 = -35.0^k$

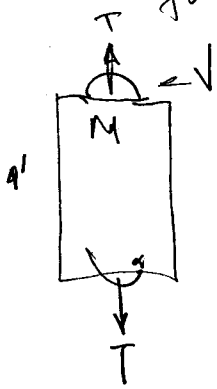
(2b) $A_s = \frac{P_T}{f_s} = \frac{35.0}{24\,ksi} \doteq 1.46\,\square''$; (AT WINDOW) $M'' = \frac{Vh}{2} \doteq \frac{31.5/2 \; 4'}{2} \doteq 31.5'^{-k}$

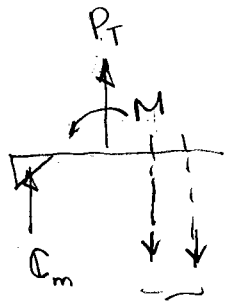
$A_s \sim \frac{31.5 \times 12}{24(.9)(68'')} \doteq .26\,\square''$

$\Sigma A_s \sim 1.71\,\square''$

try 4-#6

$A_s = 1.76\,\square''$





$A_s/2 \Leftarrow$ creates T_s

(1) $C_m + P_T = T_s$

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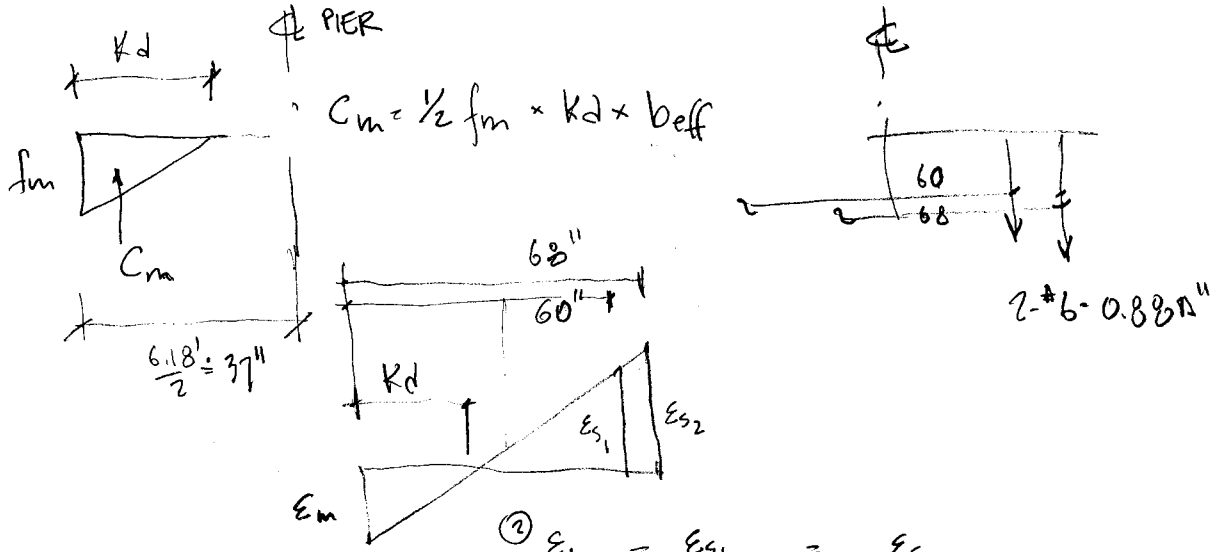

$C_m = \frac{1}{2} f_m \times Kd \times b_{eff}$

② $\frac{\varepsilon_m}{Kd} = \frac{\varepsilon_{s_1}}{60 - Kd} = \frac{\varepsilon_{s_2}}{68 - Kd}$

$f_m = E_m \varepsilon_m \qquad f_{s_1} = E_s \varepsilon_{s_1}$

① $C_m + P_T = T_s$

① $\frac{1}{2} f_m \times Kd \times b_{eff} + 35{,}000^{\#} = 0.44 \text{ in}^2 \left(f_{s_1} + f_{s_2} \right)$

from ② $\frac{\varepsilon_m}{Kd} = \frac{\frac{f_m}{E_m}}{Kd} = \frac{\frac{f_{s_1}}{E_s}}{60 - Kd} = \frac{\frac{f_{s_2}}{E_s}}{68 - Kd}$

$$f_{s_1} = f_{s_2} \left(\frac{60 - Kd}{68 - Kd} \right)$$

$$f_{s_2} = \frac{E_s}{E_m} f_m \left(\frac{68 - Kd}{Kd} \right) = n f_m \left(\frac{68 - Kd}{Kd} \right)$$

① $\frac{1}{2} f_m \times Kd \times b_{eff} + 35000 = 0.44 \left(f_{s_2} \right) \left(\frac{60 - Kd}{68 - Kd} + 1 \right)$

$b_{eff} \rightarrow 3.39"$

③ $M_{℄} = C_m \times \left(37 - \frac{Kd}{3} \right) + T_{s_1} (60 - 37) + T_{s_2} (68 - 37)$

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TRY $kd = 12''$ ⇒ ① $\frac{1}{2}(f_m)(12)(3.39) + 35000 = 0.44\left(f_{s_2}\right)\left(\frac{48}{56} + 1\right)$

$n f_m \left(\frac{56}{12}\right)$

21.5

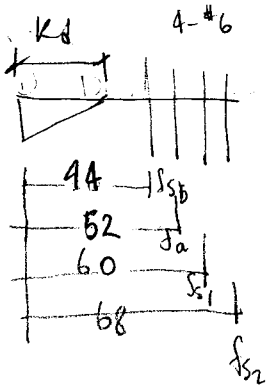
$20.34 f_m + 35000 = 81.9 f_m \Rightarrow f_m = 568$ psi $> F_m$ ng

$61.65 f_m = 35000$

$f_{s_2} = 56961$ psi $> F_s$ ng

TRY ADDED REINFORCEMENT

4ng



$f_{s_2} = n f_m \left(\frac{68-kd}{kd}\right)$

$f_{s_1} = f_{s_2}\left(\frac{60-kd}{68-kd}\right)$

$f_{s_a} = f_{s_2}\left(\frac{52-kd}{68-kd}\right)$

$f_{s_b} = f_{s_2}\left(\frac{44-kd}{68-kd}\right)$

① $\frac{1}{2} f_m (kd)(3.39) + 35000 = 0.44 f_{s_2}\left[1 + \frac{60-kd}{68-kd} + \frac{52-kd}{68-kd} + \frac{44-kd}{68-kd}\right]$

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if $kd = 3''$ ① $5.09 f_m + 35000 = 0.44 \overset{21.5}{(n f_m)} \left(\frac{68 - \overset{3}{kd}}{\underset{3}{kd}}\right) \underbrace{\left[1 + \frac{57}{65} + \frac{49}{65} + \frac{41}{65}\right]}_{3.262}$

$5.09 f_m + 35000 = 668.5 f_m$

$663.41 f_m = 35000 \Rightarrow f_m = 52.8$ psi $< F_m$ OK

$f_{s_2} = 24576 > F_s$ NG

ADD ANOTHER BAR (#6) WITH $kd = 3''$

$f_{s_{2c}} = f_{s_2}\left(\frac{36 - kd}{68 - kd}\right)$

$5.09 f_m + 35000 = 0.44 [21.5 f_m] \frac{65}{3} \underbrace{\left[1 + \frac{57 + 49 + 41 + 33}{65}\right]}_{3.769} = 772.57 f_m$

$f_m = 35000 / 767.48 = 45.6$ psi $\ll F_m$

$f_{s_2} = 21244$ psi $< F_s$

Check

$M = \overset{5.09 f_m = 232}{\underbrace{C_m (36'')}_{8356}} + \underbrace{\underset{9348}{T_{s_2}} (31)}_{289788} + \underbrace{\underset{8197}{T_{s_1}} (23)}_{188531} + \underbrace{\underset{6179}{T_{s_a}} (15)}_{92685} + \underbrace{\underset{5896}{T_{s_b}} (7)}_{41272} + \underbrace{\underset{2994}{T_{s_c}} (-1)}_{-2994} = 617638''\#$

$\doteq 51.5'^{k} >$

$M = 31.5'^{k}$

OK

∴ 5#-6 AT ½ PIER

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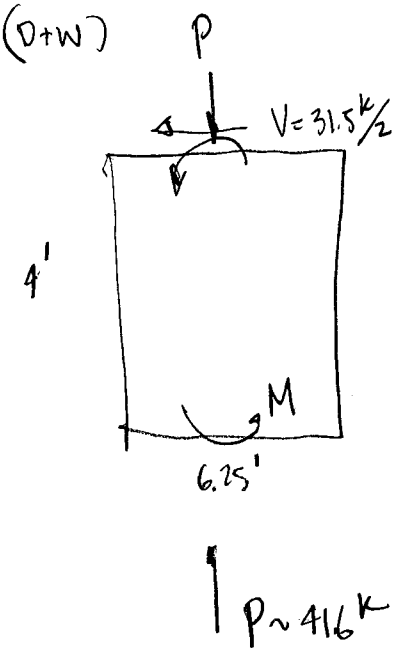
2a (D+W)



$M = \frac{Vh}{2} = 31.5'^{k}$

(D+W)

$f_b = \frac{41.6}{6.25(40.7)} \pm \frac{31.5 \times 12}{\frac{3.39(6.25 \times 12)^2}{6}} = 164 \pm 119$

totally in compression $f_{b_{max}} = 283$ psi

$F_m = \frac{1}{3} f'_m = 500$ psi ok

no reinf. needed

check $f_v = \frac{31.5^k/2}{6.25(40.7)} \doteq 62$ psi

$\frac{M}{Vd} = \frac{31.5'^{k} \times 12}{\frac{31.5}{2}(60)} \doteq 0.4 < 1.0$ ← average

$F_v = \frac{1}{3}\left[\left(4 - \frac{M}{Vd}\right)\sqrt{f'_m}\right] = 46.5$ psi

∴ Requires shear reinforcement

WITH REINFORCEMENT: $F_v = \frac{1}{2}[\quad] = 70.9$ psi $< f_v$ ✓

$F_{v(max)} = 120 - 45\left(\frac{M}{Vd}\right) = 120 - \overset{18}{45(A)} = 102$ psi $< f_v$ ✓

$A_v = \frac{V_s}{F_s d} = \frac{\left(\frac{31.5^k}{2}\right)(16'')}{24\text{ksi}(60)} \doteq 0.18\,\square''$ ← assumed ⇒ #3 ties @ 16"

(2-#3 bars = 0.220"□") ok

spacing ok per 2.3.5.3.2

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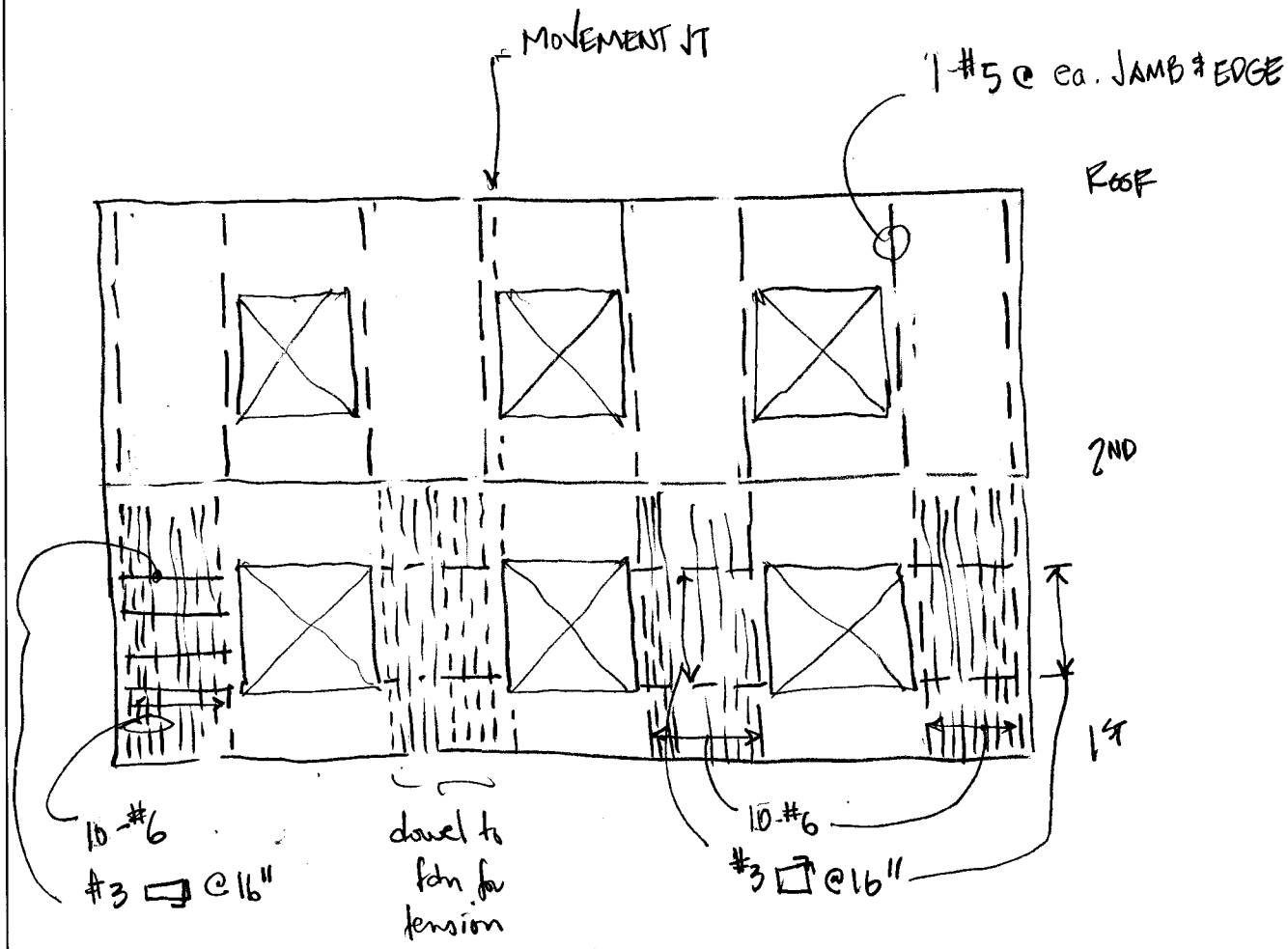
PROJECT NUMBER: 9128 DATE: 12/29/07 PAGE: 5/3

PROJECT: HYBRID - TYPE I BY: DTB

SUBJECT: SHEAR WALLS



IN-PLANE

WEST WALL (EAST WALL - SIM.)

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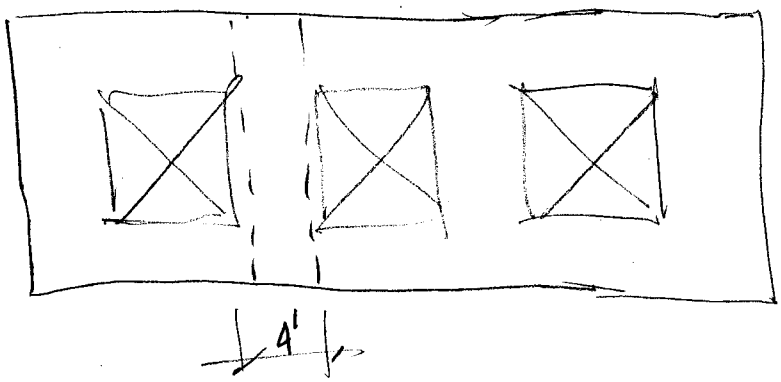
LATERAL LOAD — 20 psf USED

NORTH WALLS — BOTH FLOORS



$W = \frac{8'}{4} \times 20 = 40\text{psf}$ ON THE PIER

$M = \frac{w(12.67')^2}{8} \doteq 6.4'^{k}/4' \doteq 1.6'^{k}/ft$ (TRIB)

$A_s \sim \frac{M \times 12}{F_s j d} = 0.24 \square''/ft \times 4' = 96.\square''$ try 3-#6 ($A_s = 1.32\square''$) (0.33/ft)

(F_s = 24, j = .9, d = 3.8")

$np = \frac{21.5(.33)}{12(3.8)} = 0.156$ $k = .424$ $j = 0.859$ $kd > t_f$ grout solid

$M_s = A_s f_s j d = .33(24)\left(\frac{.859(3.8)}{12}\right) \doteq 2.2'^{k}/ft$ OK

$M_M = \frac{F_m}{2} k j \frac{b d^2}{12} \doteq \frac{500}{2}(.424)(.859)\frac{12(3.8)^2}{12} \doteq 1.3'^{k} < 1.6$ NG

REQ'D $F_m = \frac{1.6}{1.3}(500) = 615$

Req'd $f'_m = 1846$

Type S mortar

Requires CMU strength = 2500 psi

∴ OUT-OF-PLANE CONTROLS

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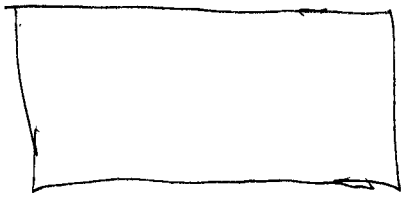
PROJECT NUMBER: 9128 DATE: 1/2/08 PAGE: F2

PROJECT: HYBRID - TYPE I BY: DMB

SUBJECT: OUT-OF-PLANE

NORTH WALL



$M = 20\frac{(12.67)^2}{8} = .4^{1\text{-}k}/ft$

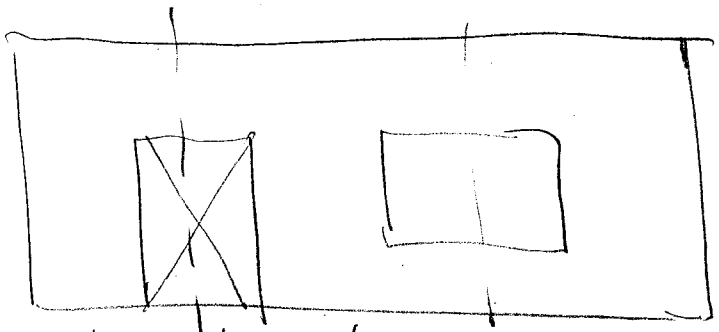
$A_s \cong .06\ \square''/ft$ try # 4 @ 40" ⇒ $A_s = 0.6$

$n_p = \frac{21.5(.06)}{12(3.8)} \doteq .028$ $k = .211$ $j = .930$ $Kd = 0.80'' < t_{face\ shell}$ ok

$M_s = A_s f_s \frac{jd}{12} \doteq .42^{1\text{-}k}/ft$ OK $M_m = \frac{F_m}{2} kjb\frac{d^2}{12} = .7^{1\text{-}k}/ft$ ok

1500 psi ok

4 @ 40



$w_e = \left[\frac{6 + 7.92/2}{6}\right]20 = 33.2\ psf/ft$

$w_e = \frac{9'}{6}(20) = 30\ psf/ft$

$w_e = \left[\frac{1.5 + 6 + 7.92/2}{6}\right]20 = 38.2\ psf/ft$

$M_{max} = 38.2\ psf/ft\frac{(12.67)^2}{8} \doteq .51^{1\text{-}k}/ft$ $A_s \cong 0.07\ \square''/ft$ try # 4 @ 32" ($A_s = .075$)

$n_p = 21.5(.075)/12(3.8) = 0.035$; $k = .233$; $j = .922$ $Kd = 0.89'' < t_f$ ok

$M_s = A_s f_s jd/12 = .53^{1\text{-}k}/ft$ ok $M_m = \frac{F_m}{2} kjb\frac{d^2}{12} \doteq .78$ ok

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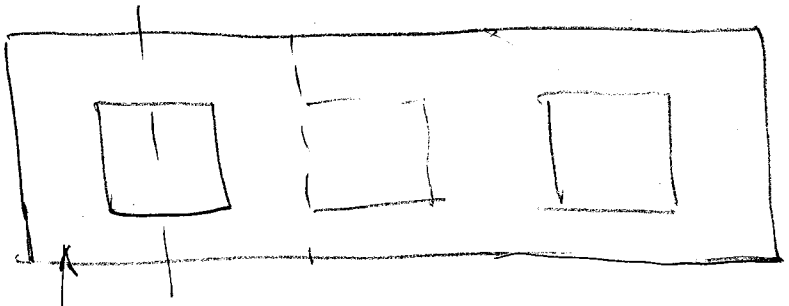
WEST WALL



$$W_e = \left[\frac{\left(6.25 + \frac{4.67}{2}\right)}{6.25}\right] 20 = 27.5 \text{ psf}$$

From IN-PLANE

$M = 27.5(12.67)^2/8 = .56^{1\text{-}k}/\text{ft}$ $\quad A_s \sim 0.08\,\square''/\text{ft}$ $\quad$ #6 @ 24" ($A_s = .22$)

$np = \frac{21.5(.22)}{12(3.8)} = .104$ $\quad k = .363$ $\quad j = .879$ $\quad Kd = 1.38'' > t_{face\ shell}$

$M_s = A_s F_s jd/12 \doteq 1.5^{1\text{-}k}$ OK $\quad M_m = \frac{F_m}{2} kj\frac{bd^2}{12} = 1.2^{1\text{-}k}/\text{ft}$ OK

#6 @ 24"
$f'_m = 1500$ psi

if $Kd = t_{face\ shell}$; $k = .329$, $j = .890$

$M_m = \frac{F_m}{2} kj\frac{bd^2}{12} \doteq 1.1^{1\text{-}k} > .56$ OK

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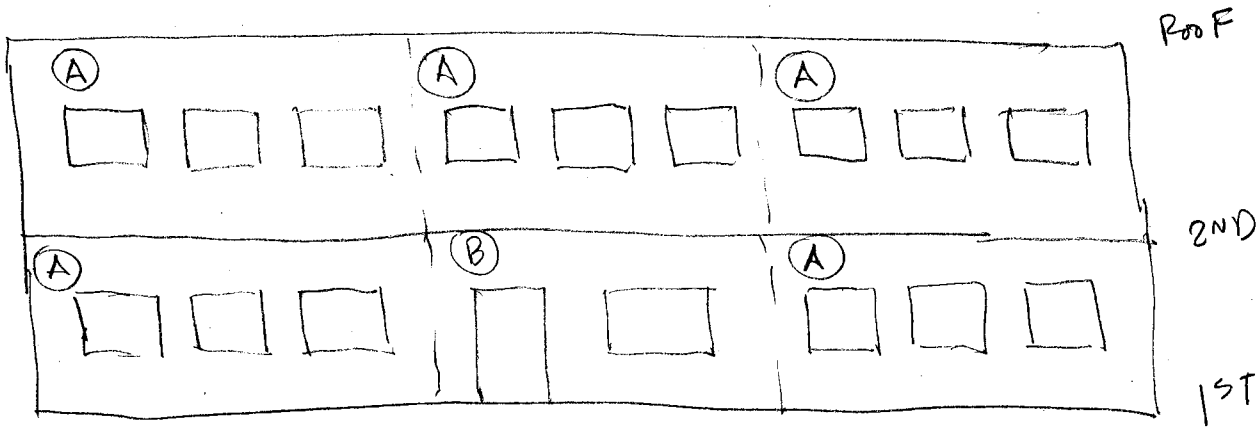
PROJECT NUMBER: 9128 DATE: 1/2/08 PAGE: L4

PROJECT: HYBRID - TYPE I BY: DMB

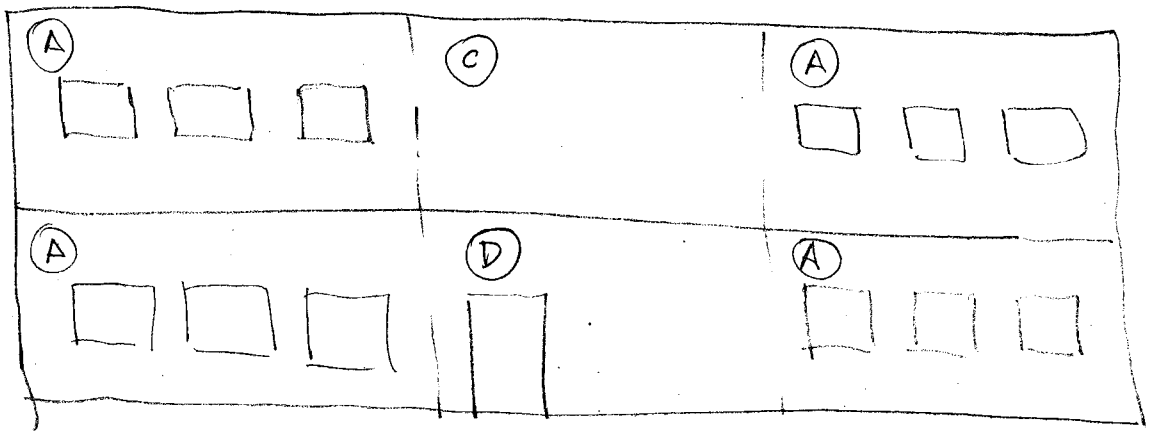
SUBJECT: SUMMARY

SUMMARY INCLUDES RESULTS OF SHEAR WALL & OUT-OF-PLANE DESIGNS

NORTH WALL



SOUTH WALL



SEE NEXT PAGE FOR REINFORCEMENT

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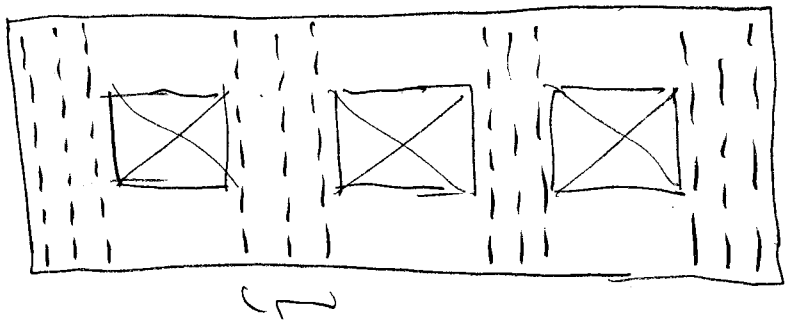
PROJECT NUMBER: 9128 DATE: 1/2/08 PAGE: L22

PROJECT: HYBRID - TYPE I BY: DTB

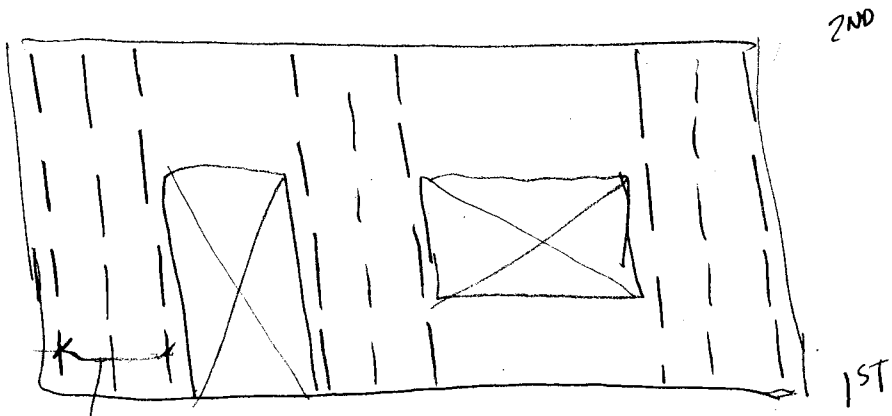
SUBJECT: SUMMARY

(A)



3-#6 PER PIER ; GROUT PIER SOLID
WALL $f'_m = 1850$ psi
Type S mortar ; CMU = 2500 psi by Unit Strength

(B)



3-#4
$f'_m = 1500$ psi
Type S mortar ; CMU = 1900 psi by Unit Strength

(C) SOLID WALL ⇒ #4 @ 40" oc
$f'_m = 1500$ psi

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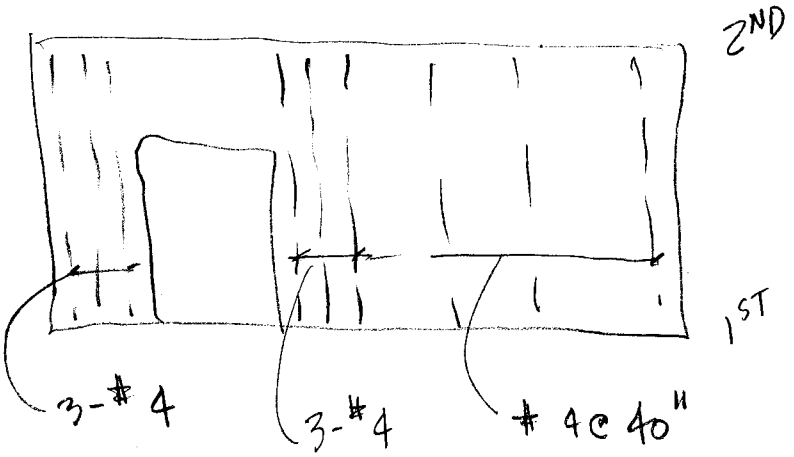
(610) 701-4282 Fax (610) 701-0205

PROJECT NUMBER: 9128 DATE: 1/2/08 PAGE: L23

PROJECT: HYBRID - TYPE I BY: DHB

SUBJECT: SUMMARY

(D)



f'm = 1500 psi

Ryan-Biggs Associates, P.C.
☑ 291 River St., Troy, NY 12180
(518) 272-6266 Fax (518) 272-4467
☐ 7 Fennell St., Skaneateles, NY 13152
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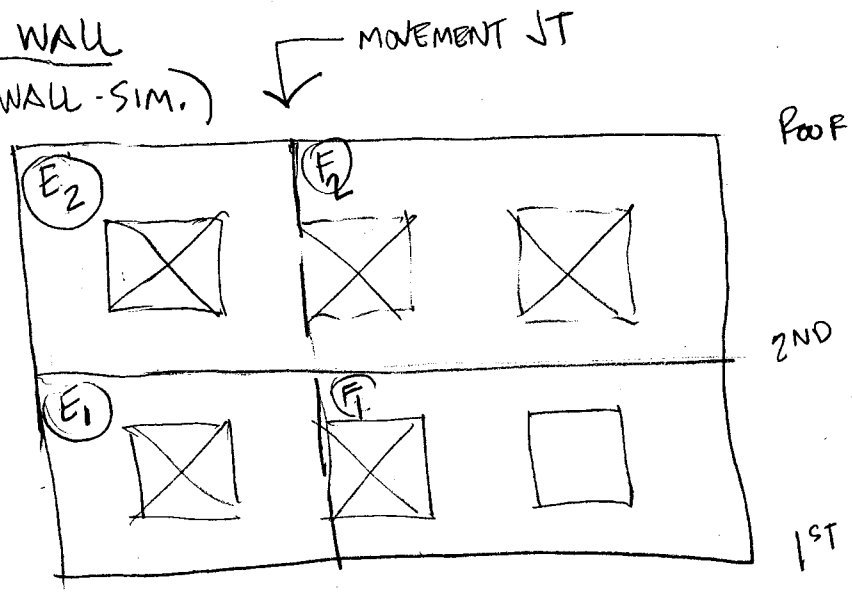
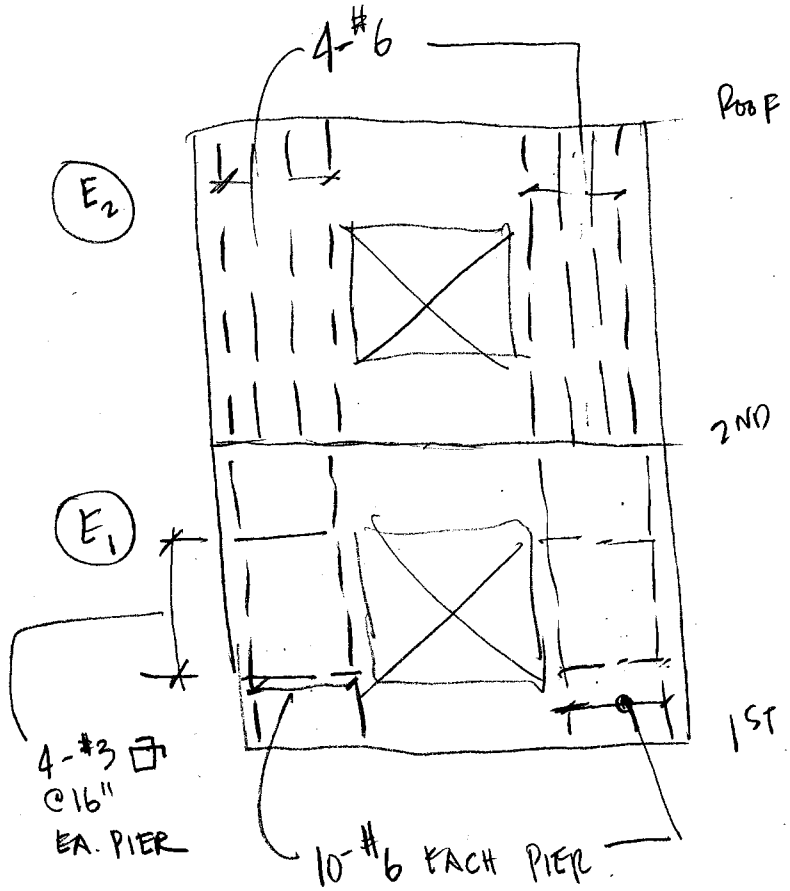
PROJECT NUMBER: 9128 DATE: 1/2/08 PAGE: S14

PROJECT: HYBRID- TYPE I BY: DIB

SUBJECT: SUMMARY

WEST WALL
(EAST WALL - SIM.)





$f'_m = 1500$ psi

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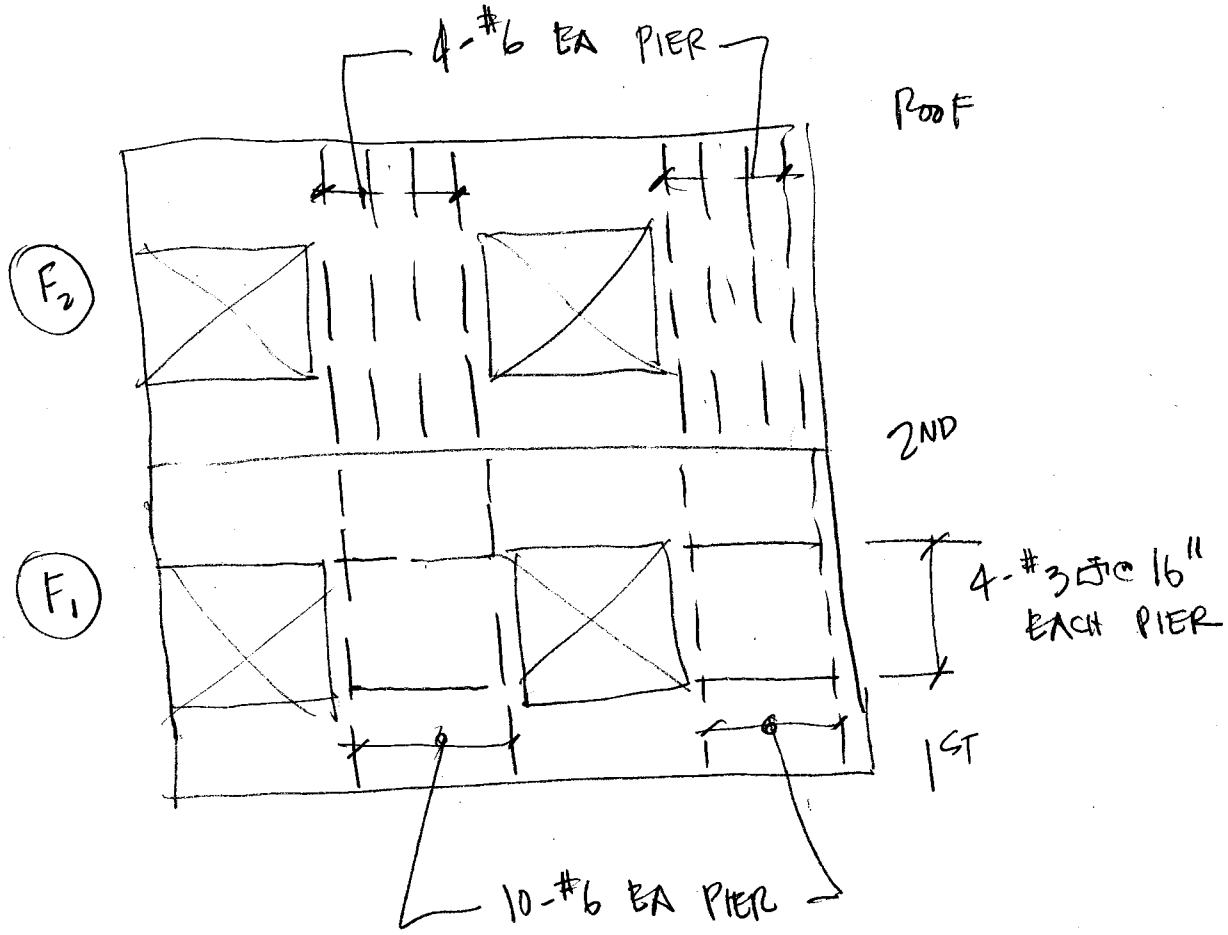
PROJECT NUMBER: 9128 DATE: 1/2/08 PAGE: 515

PROJECT: HYBRID - TYPE I BY: DJB

SUBJECT: SUMMARY



$f'_m = 1500$ psi