DESIMONE

Foundation Permit Submittal

Volume I – Project Overview and Structural Analysis

301 Mission Street
San Francisco, CA

Prepared for:

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DeSimone Project #4069

May 24, 2005

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VOLUME II - FOUNDATION DESIGN

SECTION 4 – TOWER PILE FOUNDATION SYSTEM

4.1 Design Methodology And Assumptions

4.1 Design Methodology and Assumptions

The foundation footprint measures 103'-5"(E-W) x 178'-4"(N-S). The foundation system consists of approximately 950-14"x14" square piles and a 10'-0" thick pile cap, in addition to a 3'-0" thick mat cantilevered from the pile cap. This layout is developed so that the 10'-0" thick portion of the foundation is centered about the tower above, in order to limit differential settlement across the base of the tower.

Loads onto the foundation include gravity loads and seismic loads. For the 10'-0" portion, the effect of the ground water pressure is ignored as it is smaller than the unit weight of the mat. For the 3'-0" portion, however, this is not the case and the ground water pressure is included in the design.

Analysis and design are done with the aide of a three-dimensional computational program, SAFE. Soil sub-grade moduli values are obtained from the project geotechnical engineer, Treadwell & Rollo, dated January 4, 2005. These values are established through close collaboration between the two offices. Estimated settlement values and the corresponding sub-grade modulus values are included in this section.

Since the pile cap is supported by many piles at uniform spacings, per discussion with Treadwell & Rollo, it is designed as a foundation mat with varying sub-grade moduli across the building site.

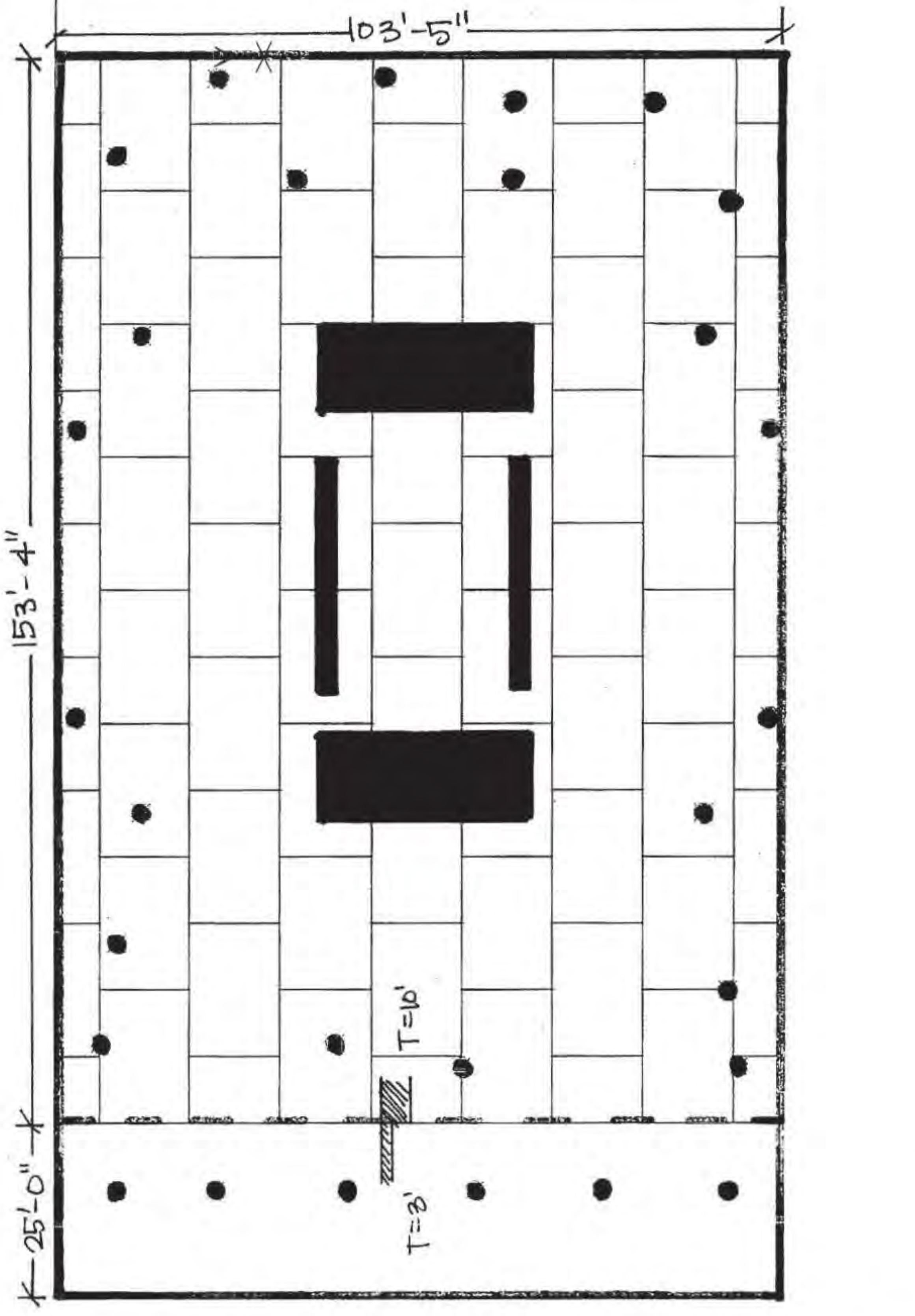
Two SAFE models are considered in the flexural design of the gravity loads (permanent case – model 1) and with seismic loads (transient case – model 2):

Model 1 is developed using the sub-grade moduli from Treadwell & Rollo, which captures the effects of long-term deflection of the sub-grade. The only applied loads are gravity loads.

Model 2 is developed using the relative spacing of the piles under different areas of the pile cap. For instance, the piles are at 42" o.c. under the core and at 56" o.c. elsewhere. So relatively the sub-grade modulus under the core is $56^2/42^2 = 1.78$ times stiffer than the adjacent areas. This is done to reflect the short-term nature of the seismic forces. The only applied loads are the seismic loads.

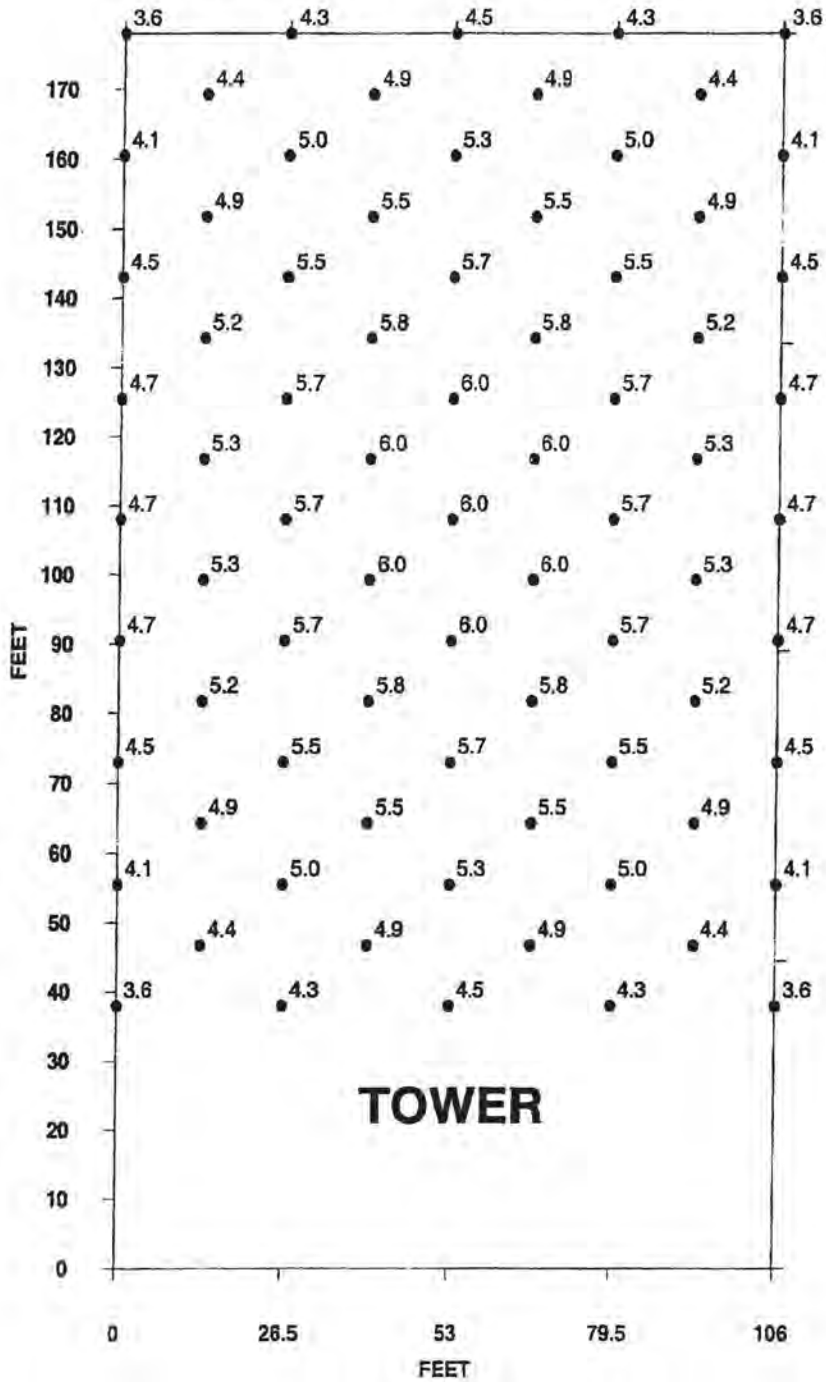
Forces in the two models are then combined for the flexural design.

The shear design of the pile cap is done using the sub-grade moduli from Treadwell & Rollo. This results in a more conservative design than the methodology used in the flexural design.



SAFE v8.0.6 - File: 4069-20050518-Tower-No-Piles-E - May 23,2005 10:30 - Scale: User Scale
Point Loading (DL) - Kip-ft Units

Estimated settlement in inches



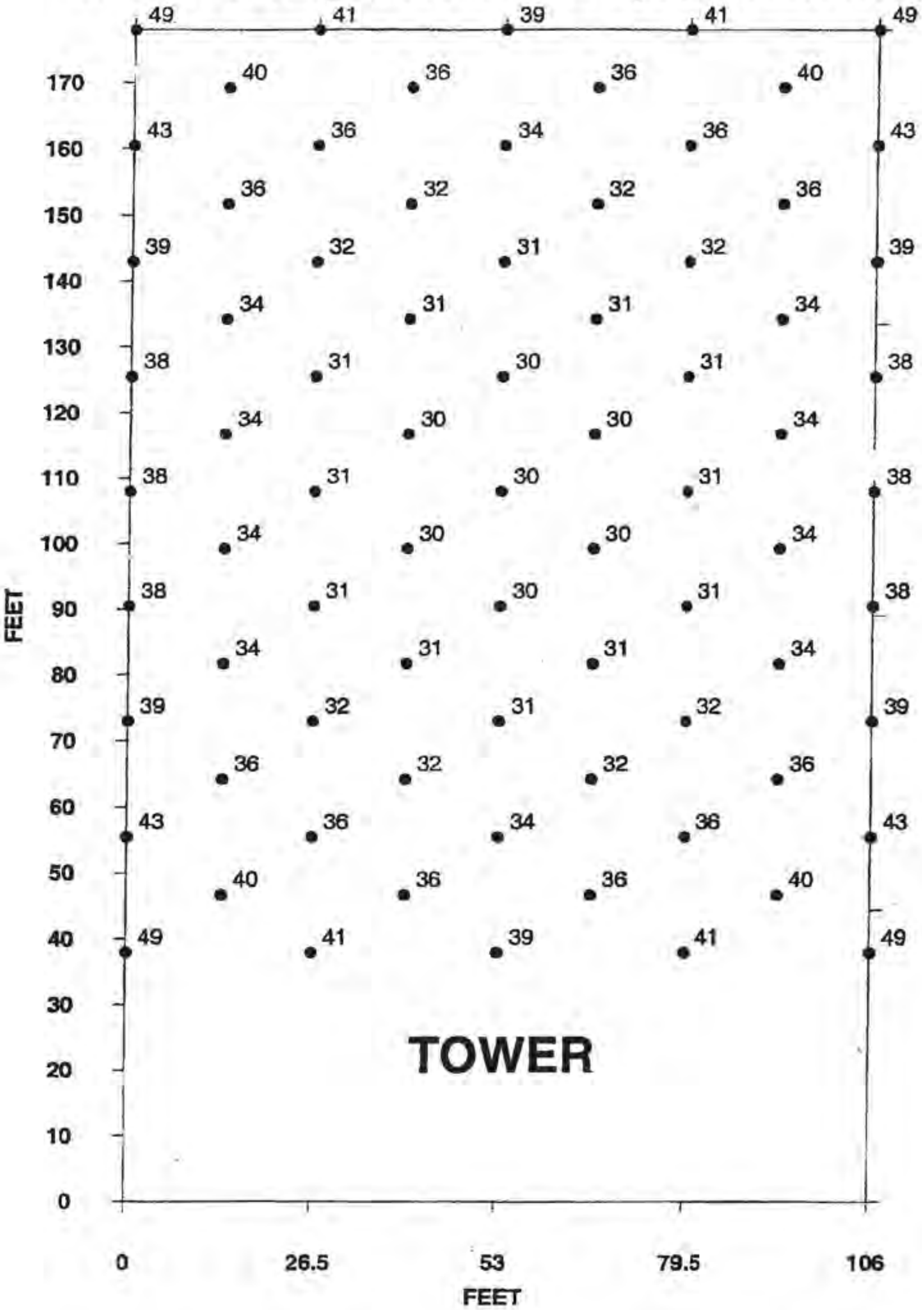
Note: **For a 25 foot excavation - Estimated settlement based on a uniform pressure over the Tower footprint (106'x140') of 14.8 kips per square foot (ksf). Assumes Tower is supported by a pile supported mat foundation.**

301 MISSION STREET
San Francisco, California
Project No. 3157.02
30 DECEMBER 2004

ESTIMATED SETTLEMENT
TREADWELL & ROLLO, INC.

41-3

Modulus of subgrade reaction in kips per cubic feet (kcf)



Note: For a 25 foot excavation - Estimated subgrade modulus calculated by taking a uniform building pressure of 14.8 ksf and dividing by the predicted settlement. Assumes Tower is supported by a pile supported mat foundation (106'x140').

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Project No. 3157.02
30 DECEMBER 2004

MODULI OF SUBGRADE REACTION
TREADWELL & ROLLO, INC.

4.1-4

DODSONNOC00000236

4.2 Design Forces And Load Combinations

4.2 Design Forces and Load Combinations

The following loads are considered in the design of the foundation:

Ground water pressure – This load is ignored in the 10'-0" portion since it is smaller than the unit weight of the mat. It is considered in the design of the 3'-0" portion.

Gravity Loads – Gravity loads used in the design are as shown in this section.

Seismic Loads – Three different levels of seismic forces are considered in the design: Core & Moment Frame force distribution per stiffness (case 2a), Moment Frame resisting 25% of the building base shear (case 2b), and Beyond Code level (case 3).

Load combinations are obtained by considering the different cases as outlines in UBC-97 and include seismic loads in both directions, including orthogonal and torsional effects where appropriate.

Description of the load combinations considered and forces are included in this section.

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1612.3.2 Alt. Load Case ASD

0.) HYDROSTATIC PRESSURE

SFCD		Proj. D.
+2.61'		0.00'
-3.00'	▽	-5.61'
-23.14'	MAT	-25.75'

$Mat = 10^{ft} * 150^{pcf} = 1500^{psf}$

$H = 20.14^{ft} * 62.4^{pcf} = 1257^{psf}$

weight of mat > hydro pressure

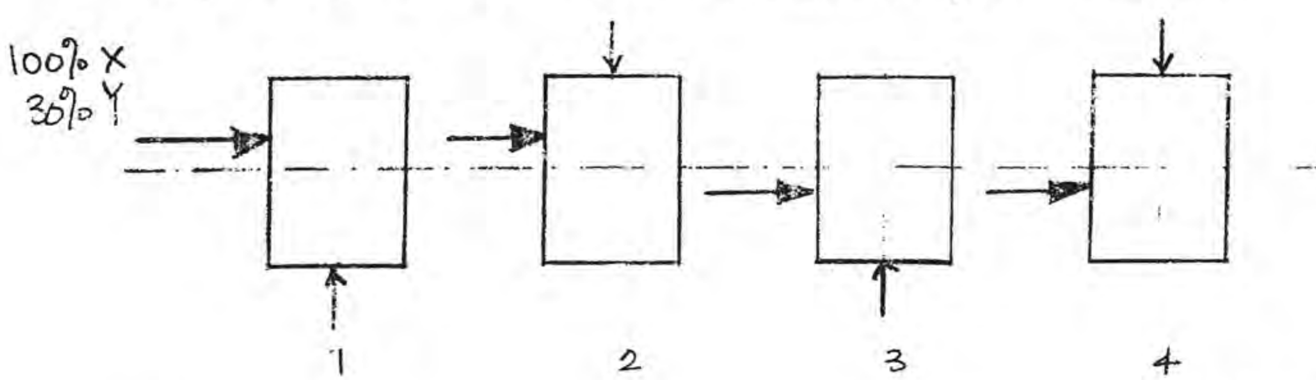
∴ Ignore H in foundation design of T = 10'

1.) GRAVITY LOADS

1. D + mat + L

2.) SEISMIC LOADS

a.) Core & MF force distribution per stiffness



negative : 5 6 7 8

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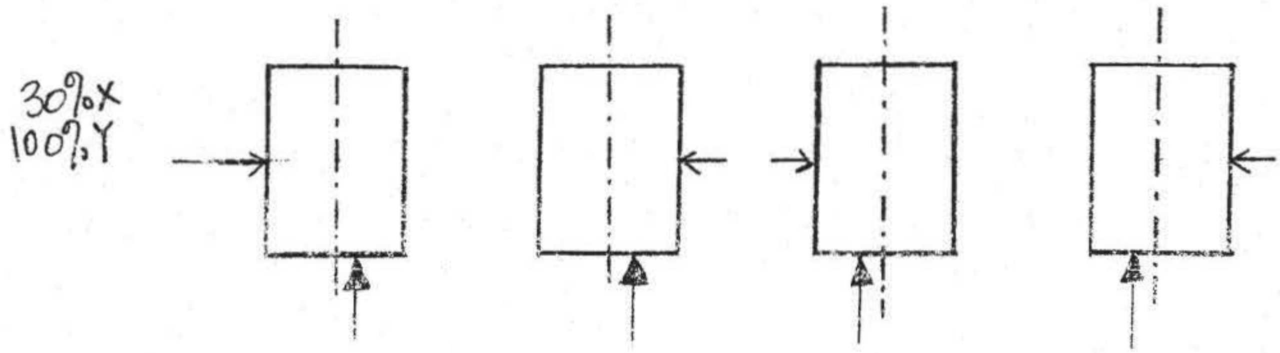


1 2 3 4

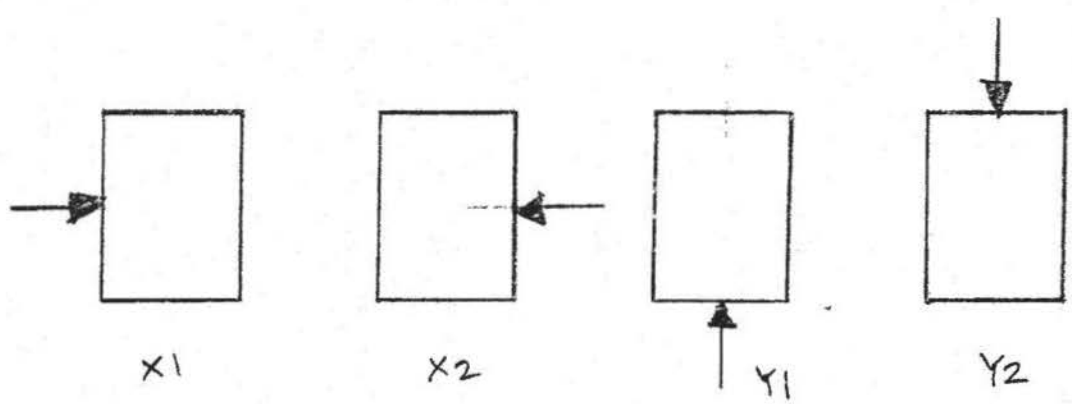
negative : 5 6 7 8

For each seismic case, combine with gravity to give:

i. D + mat + L + E/1.4 ——— 16 cases

ii. 0.9D + 0.9mat ± E/1.4 ——— 16 cases

b.) MF take 25% of total base shear



i D + mat + L + E/1.4 ——— 4 cases

ii 0.9D + 0.9mat ± E/1.4 ——— 4 cases 4.2-3

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16.12.2. Strength Design Load Combo.

1) GRAVITY LOADS.

1. $1.4D + 1.7L$

2) SEISMIC LOADS ($0.5C_aI = 0.5 \times 0.44 \times 1.0 = 0.22$)

a) i) $1.42D + 0.5L + 1.0E$ ——— 16 cases

ii) $0.9D \pm 1.0E$ ——— 16 cases.

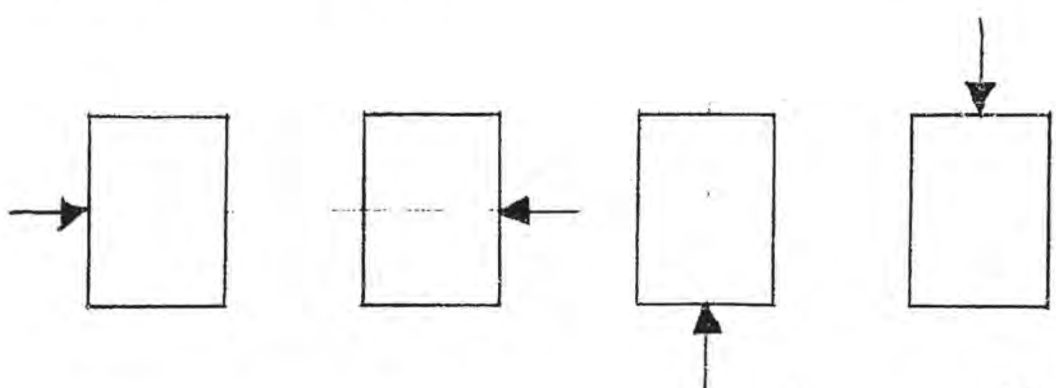
b) i) $1.42D + 0.5L + 1.0E$ ——— 4 cases

ii) $0.9D \pm 1.0E$ ——— 4 cases

BEYOND CODE LEVEL

3. i) $1.2D + 0.5L + 2.8E$ ——— 4 cases

ii) $0.9D \pm 2.8E$ ——— 4 cases



4.2-4

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Item TOWER FDN LOAD COMBO | By ML Ch'kd

For strength design, scale up element forces from ASD combos

If magnify loads input to structure, will result in unrealistic soil pressure distribution.

Equivalent to scaling up element forces from ASD, can scale down element capacity (modify ϕ factors)

Load Case 1 ASD : $D + L$

STRENGTH : $1.4D + 1.7L$

$$\text{SCALE FACTOR} = \frac{1.4D + 1.7L}{D + L} = \frac{1.4 \times 209{,}779 + 1.7 \times 21{,}536}{209{,}779 + 21{,}536}$$

$$= 1.428$$

Load Case 2a ASD : $D + L + E/1.4$

STRENGTH : $1.42D + 0.5L + 1.0E$

GRAVITY COMPONENTS - L is insignificant
↳ SCALE FACTOR ≈ 1.42

SEISMIC COMPONENTS - $1.0E = 1.4E/1.4$
↳ SCALE FACTOR = 1.4

∴ USE SCALE FACTOR = 1.428 (match case 1)

4.2-5

Load Case 2b. ASD : 0.9D + E/1.4

STRENGTH : 0.9D + 1.0E

GRAVITY COMPONENT - NO CHANGE
↳ SCALE FACTOR = 1.0

SEISMIC COMPONENT - 1.0E = 1.4E/1.4
↳ SCALE FACTOR = 1.4

∴ USE SCALE FACTOR = 1.428 (match Case 1)

MODIFY φ FACTOR

SHEAR : $\frac{0.85}{1.428} = 0.60$

FLEXURE : $\frac{0.90}{1.428} = 0.63$

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By ML Ch'kd

BEYOND CODE LEVEL, CASE 3

- Design for 2.8 × E, but need not exceed element capacity
  - ↳ evaluate forces

4.2-7

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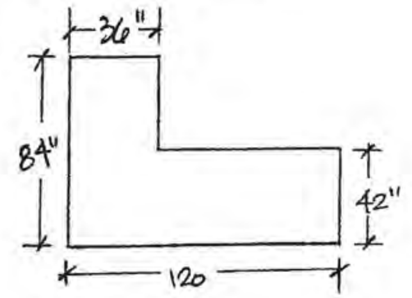
By: ML
Ch'kd ____

Outrigger Column



Vert reinf : 160 - #11 ($f_y = 75$ ksi)

$A_s = 249.6 \text{ in}^2$

$A_{gross} = 36" \times 84" + 42" \times 84" = 6552 \text{ in}^2$

$A_{conc} = 6302.4 \text{ in}^2$

Axial Compression Capacity $f'_c = 10$ ksi

$P_o = 0.85 f'_c A_{conc} + A_s f_y$

$= 0.85 \times 10^{ksi} \times 6302.4^{in^2} + 249.6^{in^2} \times 75^{ksi}$

$= 53,570^k + 18,720^k$

$= \underline{\underline{72,290^k}}$

$2.8E + D + L$
$= 2.8(0.8 \times 10,000) + 8050 + 1108$
$= 31,558^k < P_o$
$\therefore$ design for $\underline{31,558^k}$

Tensile Capacity

$T_n = A_s f_y = 18,720^k$

$2.8E - D = 2.8(0.80 \times 10,000^k) - 7577^k = 14,823^k < A_s f_y$

$\therefore$ design for $\underline{14,820^k}$

4.2-8

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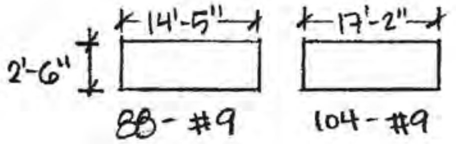
Straight Shearwalls per S3-2.31



$A_s = 192 \times 1.00 = 192\,in^2$

$A_{gross} = 31.583' \times 2.5' \times 144 = 11{,}370\,in^2$

$A_{conc} = 11{,}178\,in^2$

Axial Compression Capacity. $f'_c = 10\,ksi$

$P_o = 0.85 f'_c A_{conc} + A_s f_y$

$= 0.85 \times 10^{ksi} \times 11{,}178^{in^2} + 192^{in^2} \times 75^{ksi}$

$= 95{,}013^k + 14{,}400^k$

$= \underline{109{,}413^k}$

$2.8E + D + L$
$= 2.8(0.8 \times 3500)$
$+ 17{,}814 + 1401$
$= 27{,}055^k < P_o$
∴ design for $\underline{27{,}055^k}$

Tensile Capacity

$T_n = A_s f_y = 14{,}400^k$

$2.8E - D = 2.8(0.80 \times 3500^k) - 17{,}682^k = -9842^k$ in Compression

∴ no net tension in wall
design for reduced compression $= \underline{9842^k}$

4.2-9

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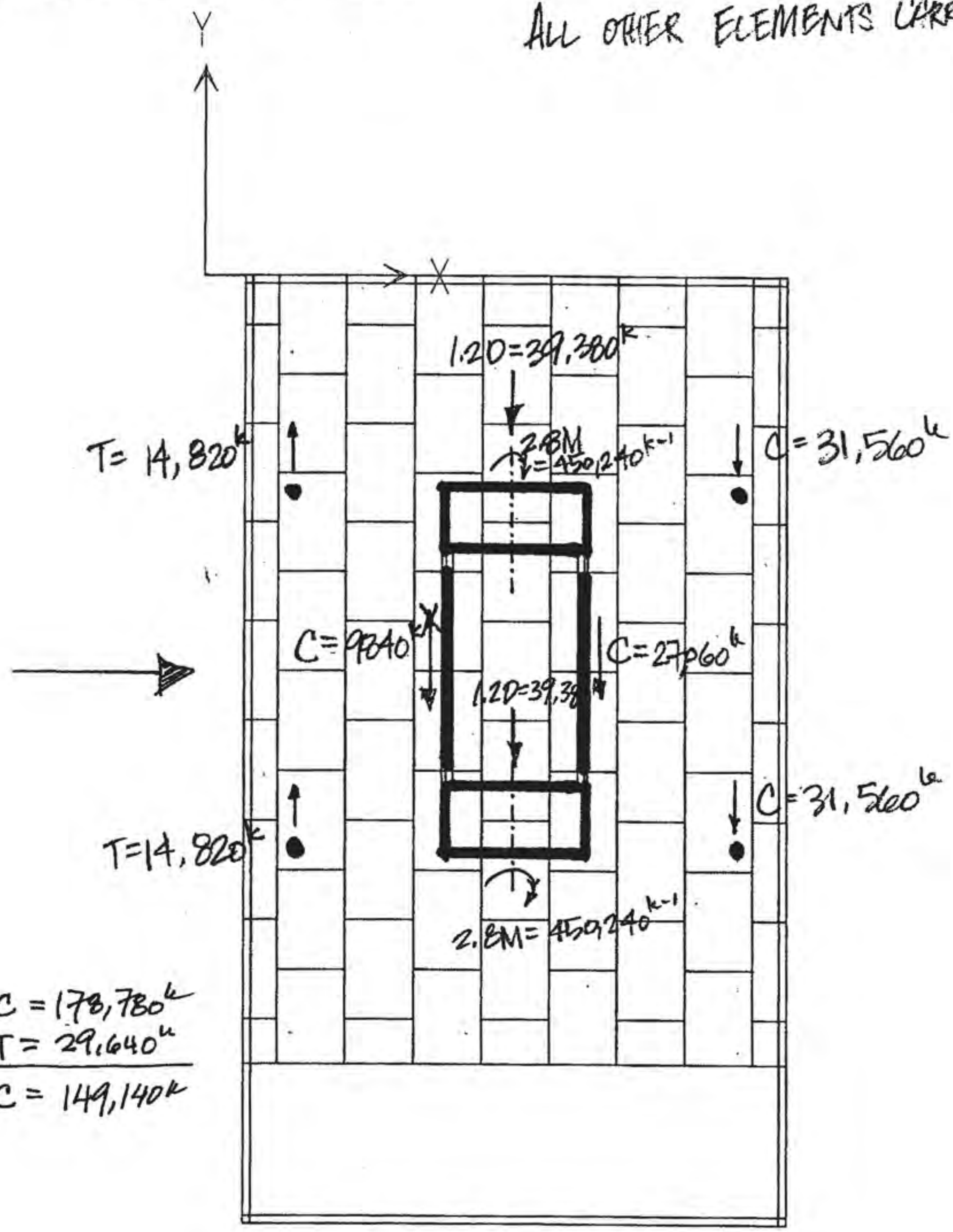
BEYOND CODE LEVEL, CASE 3

ALL OTHER ELEMENTS CARRY 1.2D +0.5L



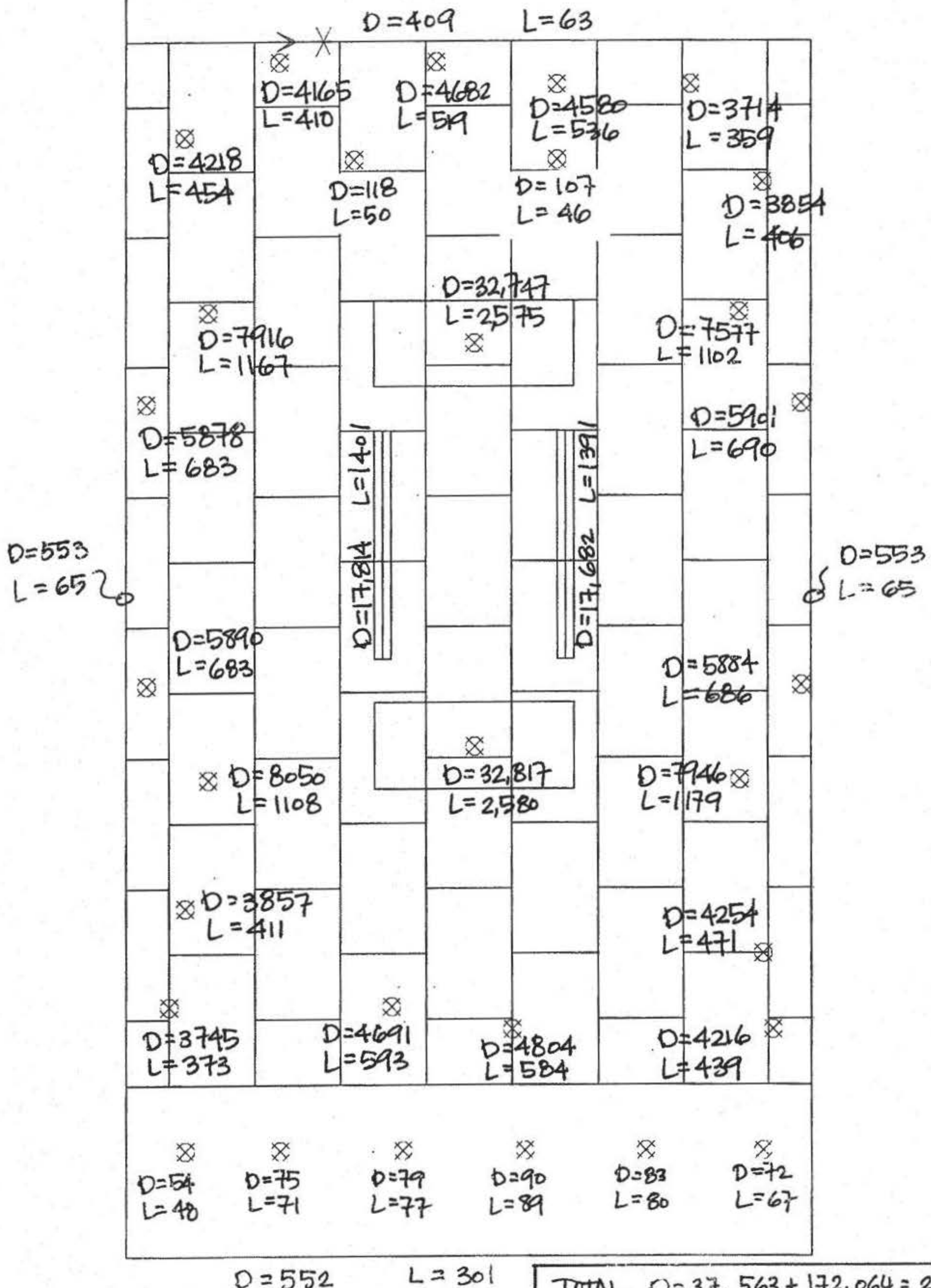
C = 178,780^k
T = 29,640^k

NET C = 149,140^k

$2.8M = 2.8(0.80 \times 201{,}000^{k-'}) = 450{,}240^{k-'}$

4.2-1D

LOADS AT FOUNDATION (k)



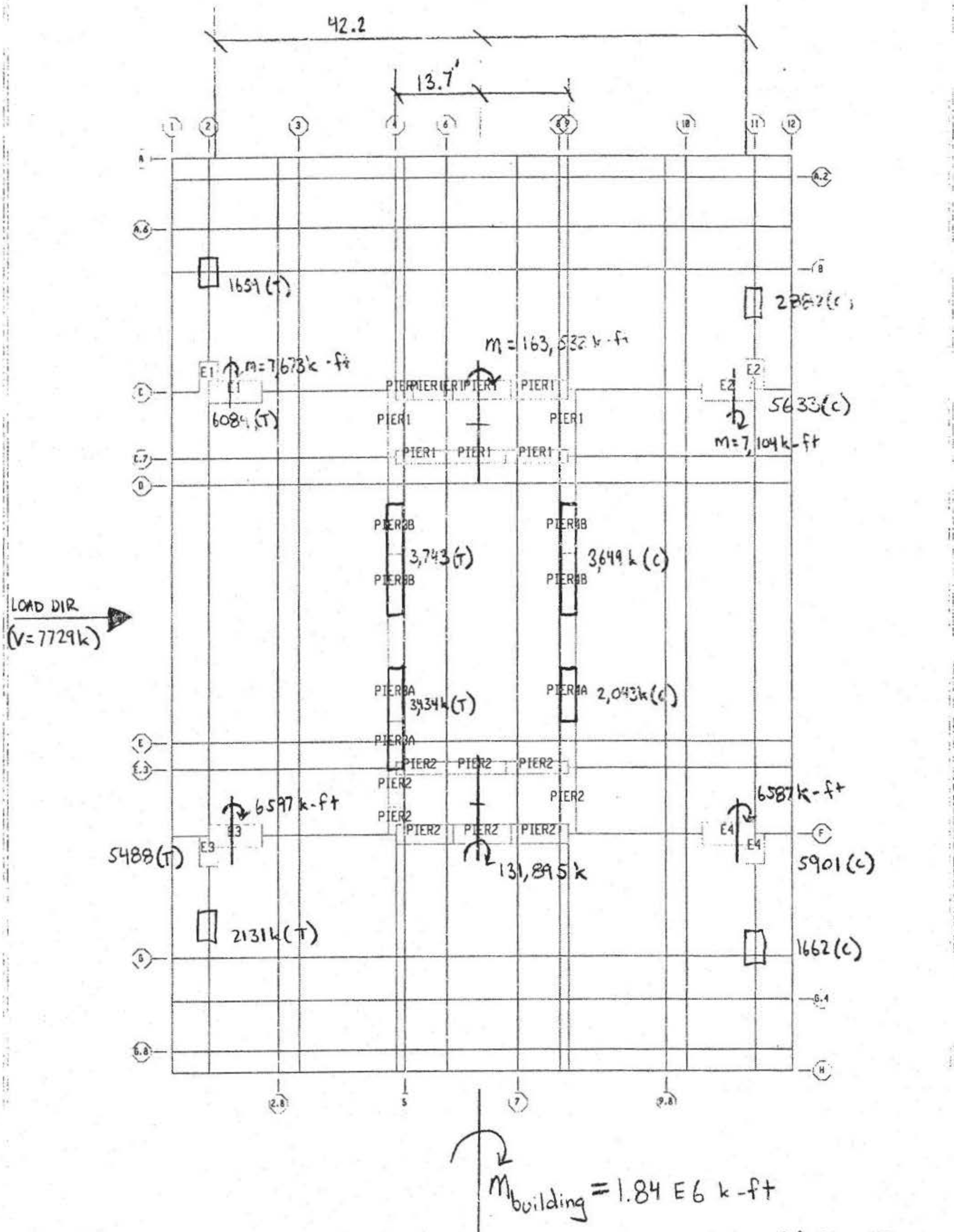
TOTAL D = 37,563 + 172,064 = 209,627
TOTAL L = 3,286 + 18,536 = 21,822

SAFE v8.0.6 - File: 4069-20050518-Tower-No-Piles-E - May 18,2005 17:28 - Scale: User Scale
Point Loading (DL) - Kip-in Units

ETABS

NJR

RSA: FSSX



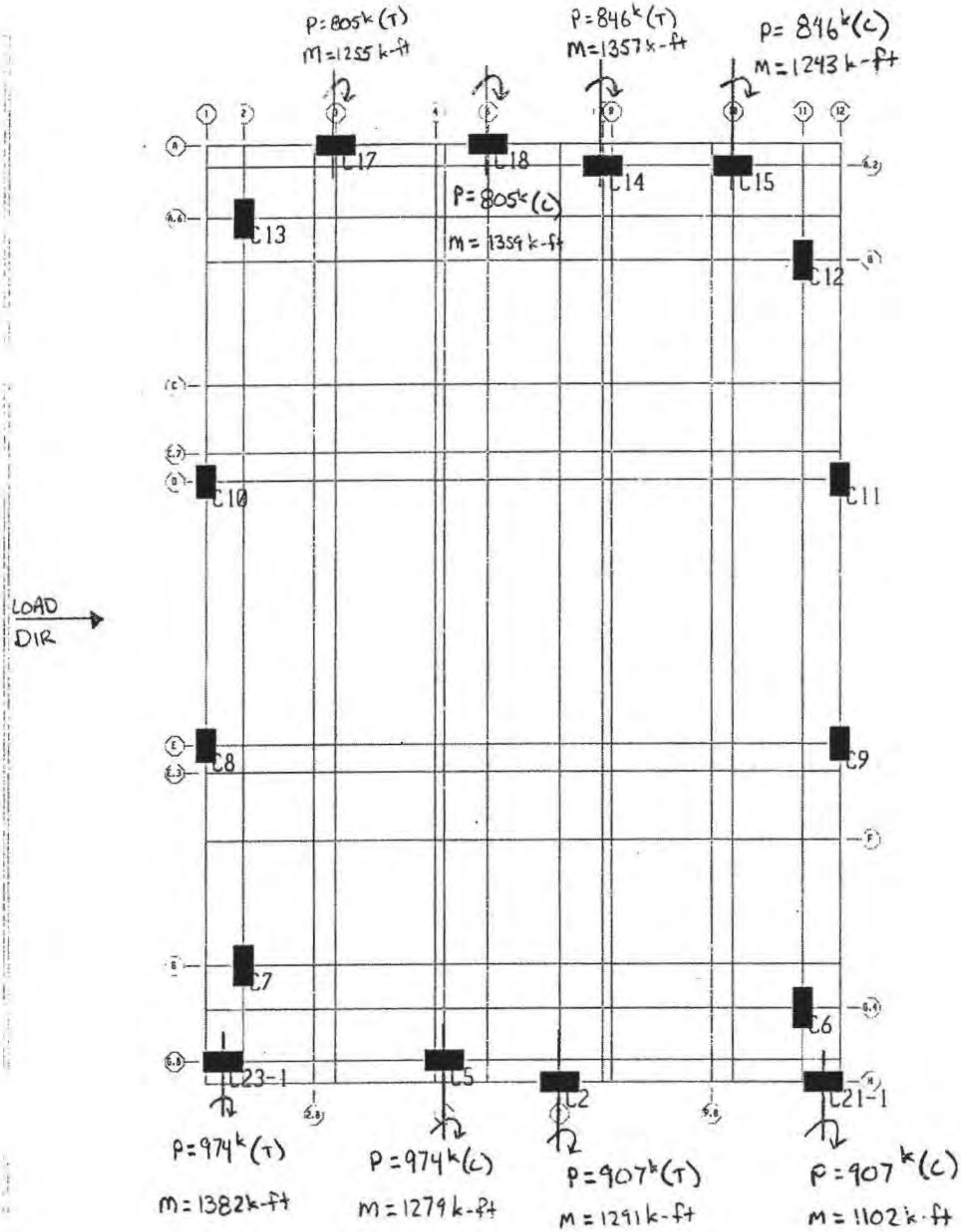
4.2-12

ETABS v8.5.0 - File: 4069-20050509-2.09-CD - May 11,2005 13:42
Plan View - GROUND - Elevation 0 - Kip-in Units

ETABS

NJR

RSA: FSSX



4.2-13

ETABS v8.5.0 - File: 4069-20050509-2.09-CD - May 12,2005 7:33
Plan View - GROUND - Elevation 0 - Kip-ft Units

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Item	FOUNDATION DESIGN LOADS	By NJR	Ch'kd

FSSX.

$$\Sigma M_{\text{CENTER OF BLDG}} = 0$$

M_{CORE}:

Pier 1 ... Pier 2 ... WEB WALL T-C COUPLES

163,532k-ft + 131,895 k-ft + (3,743 + 3,434 + 2,043 + 3,649)(13.7')

= 295,427 k-ft + 176,305 k = 0.47E6 k-ft

$M_{\text{OUTRIGGER T-C COUPLE}}$

E1 ... E2

(7673k + 1659k)(42.2ft) + (5633 + 2882)(42.2ft) +

(5488 + 2131)(42.2ft) + (5901 + 1662)(42.2)

= 1.39 E6 k-ft

$M_{\text{OUTRIGGER}}$

7673 k-ft + 5633 k-ft + 6597 k-ft + 6587 k-ft

= 0.02 E6 k-ft ← NEGLECT

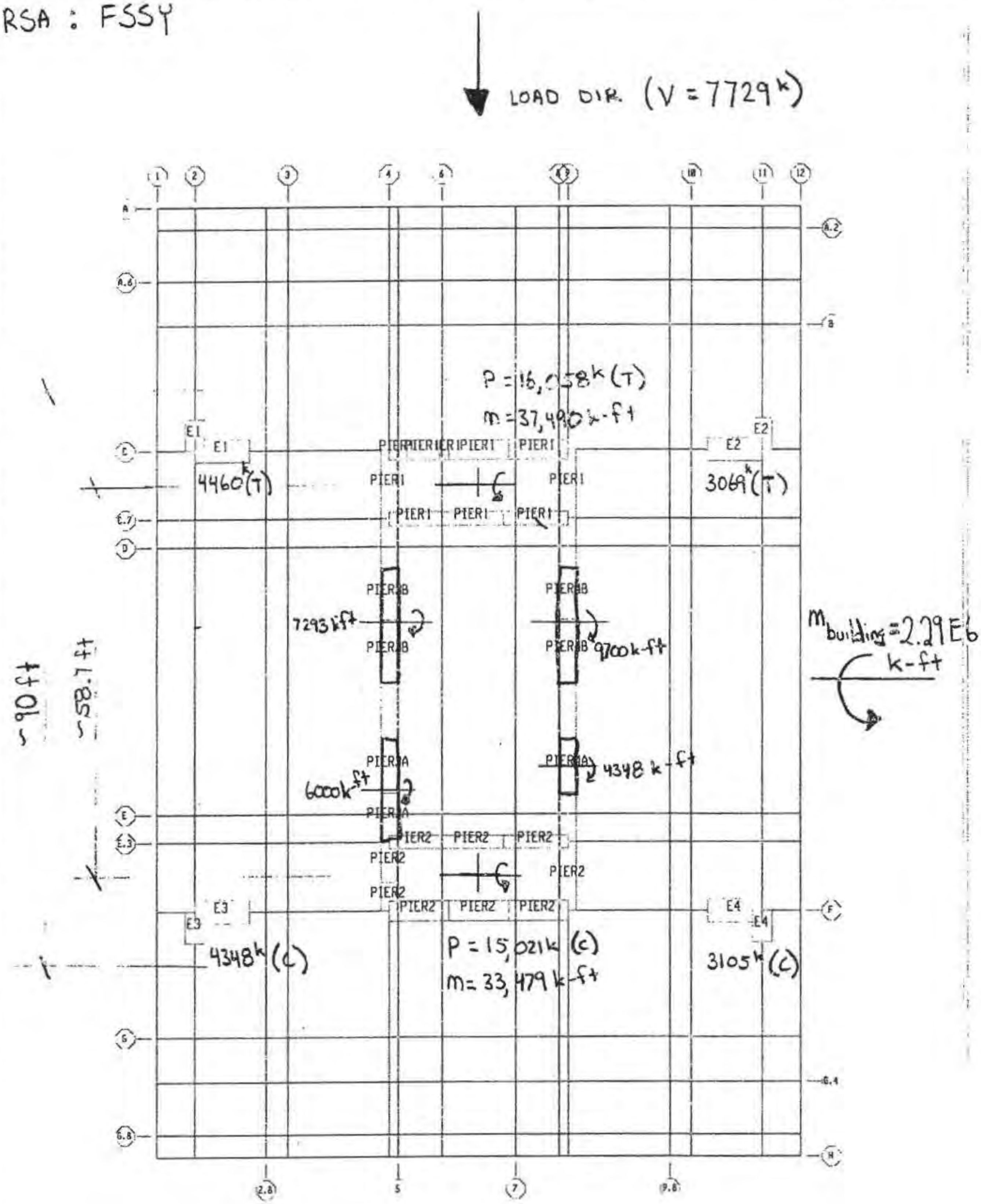
$$\Sigma M = (0.47 + 1.39)E6 - (M_{building} = 1.84\ E6\ k\text{-}ft)$$

$$\Sigma M = 1.86E6 - 1.84E6 \simeq 0$$ OK

4.2-14

ETABS

RSA : FSSY



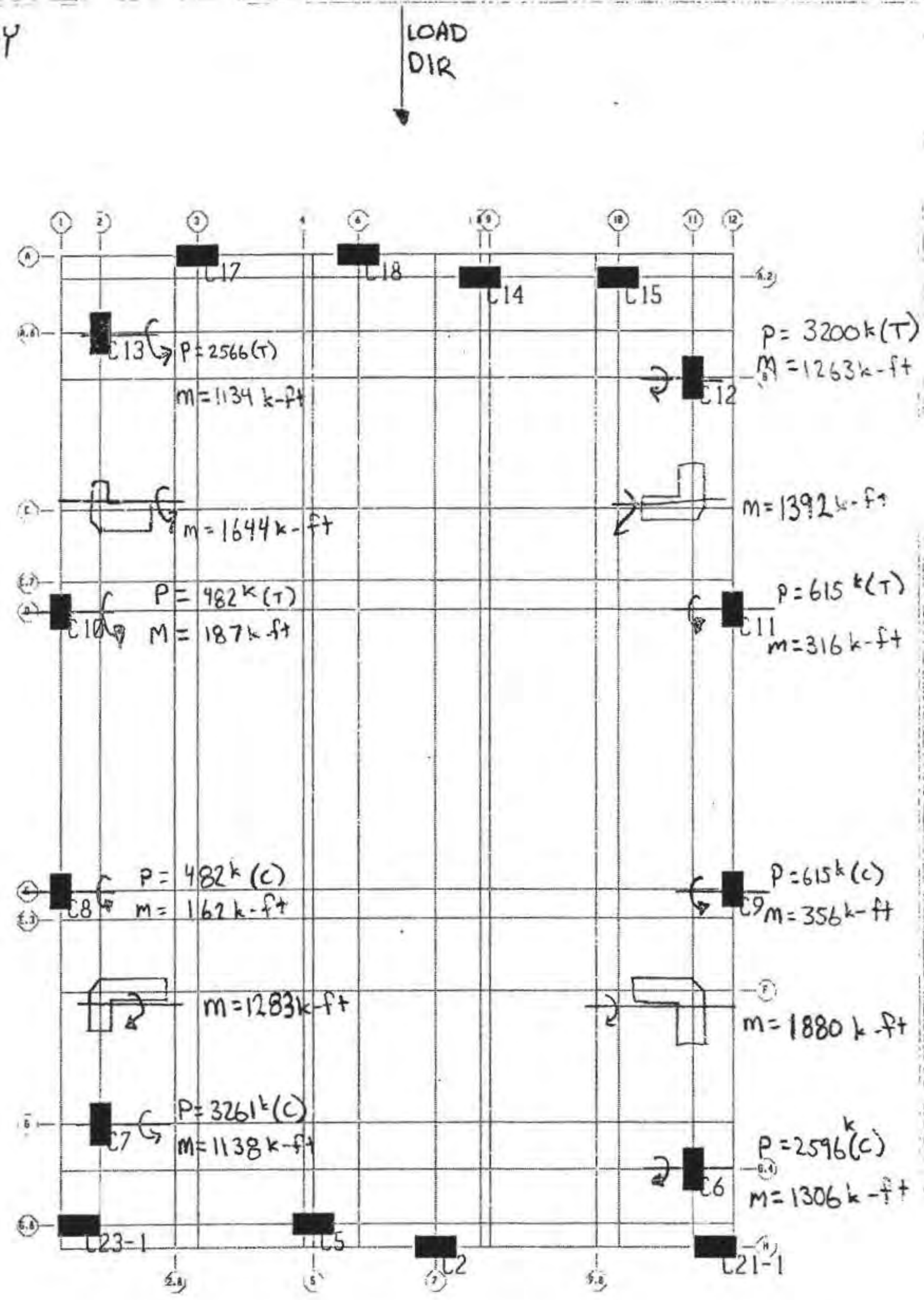
4.2-15

ETABS v8.5.0 - File: 4069-20050509-2.09-CD - May 11,2005 13:42
Plan View - GROUND - Elevation 0 - Kip-in Units

ETABS

NJR

RSA : FSSY



4.2-16

FSSY:

$\Sigma M_{\text{CENTER OF BLDG}} = 0$

(Peir 1) (Peir 2) (Peir 1 T couple) (Peir 2 couple)

SCALE FACTOR [31,490 k-ft + 33,479 k-ft + 16,058 k(29.3ft) + 15,021 k (29.3 ft)

+ 1293 k-ft + 9,700 k-ft + 6000 k-ft + 4348 k-ft

(E1 couple) (E3 couple)

+ (2566 k + 4460 k)(45 ft) + (4348 k + 3261 k)(45 ft)

(E4 couple) (E2 couple)

+ (2600 + 3105 k)(45 ft) + (3069 k + 3200 k)(45 ft)]

− ($M_{building}$ = 2.29 E6 k-ft) = 0

(CORE MOMENTS) (CORE T-C couple) (OUTRIG COL T+C couple)

SCALE factor [(93,300 k-ft + 910,600 k-ft + 1,197,400 k-ft]

= 2.29 E6 k-ft

(REACTIONS) ($M_{building}$)

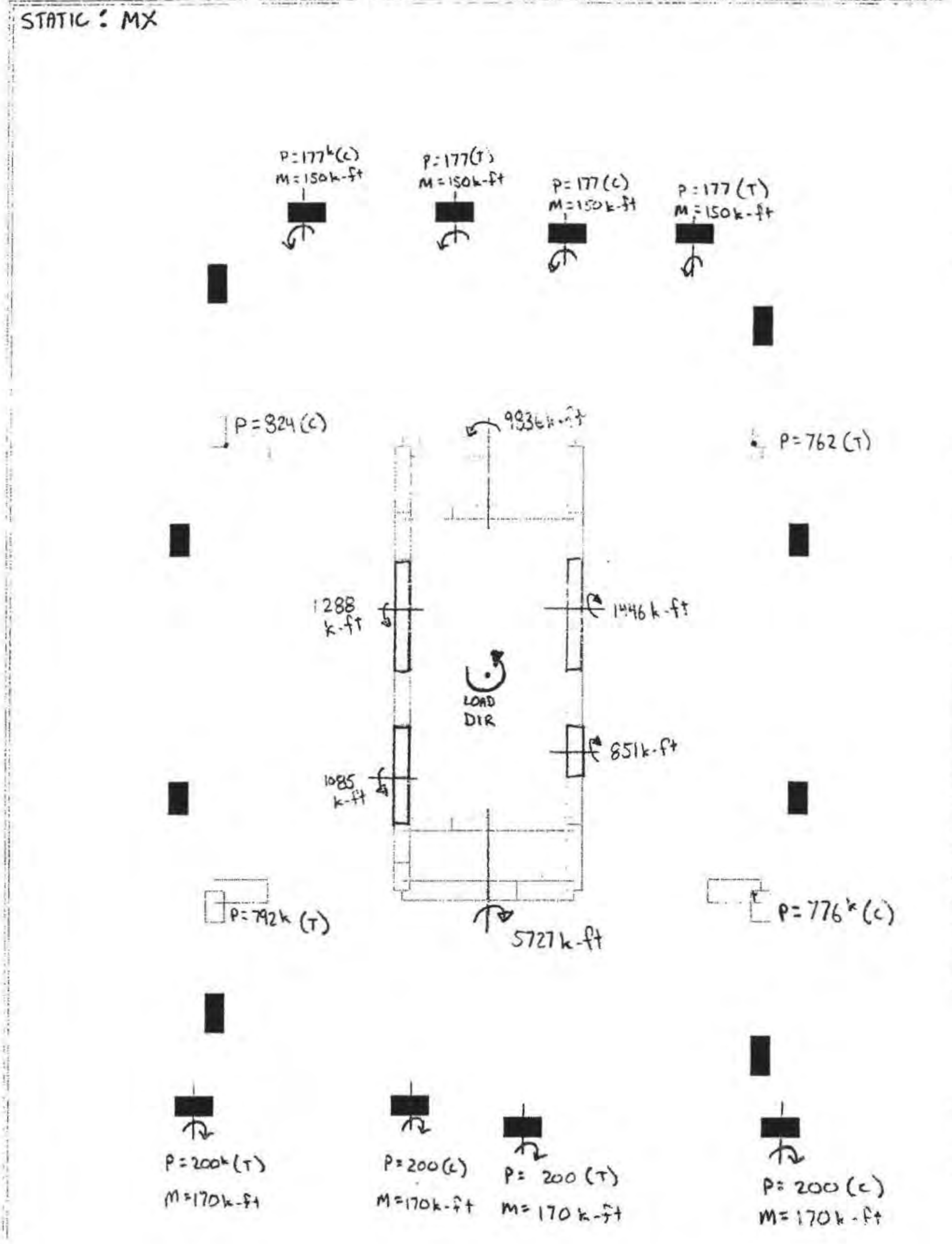
SCALE factor (2.21 E6 k-ft) = 2.29 E6 k-ft

∴ SCALE factor = 1.04

4.2-17

ETABS

NJR



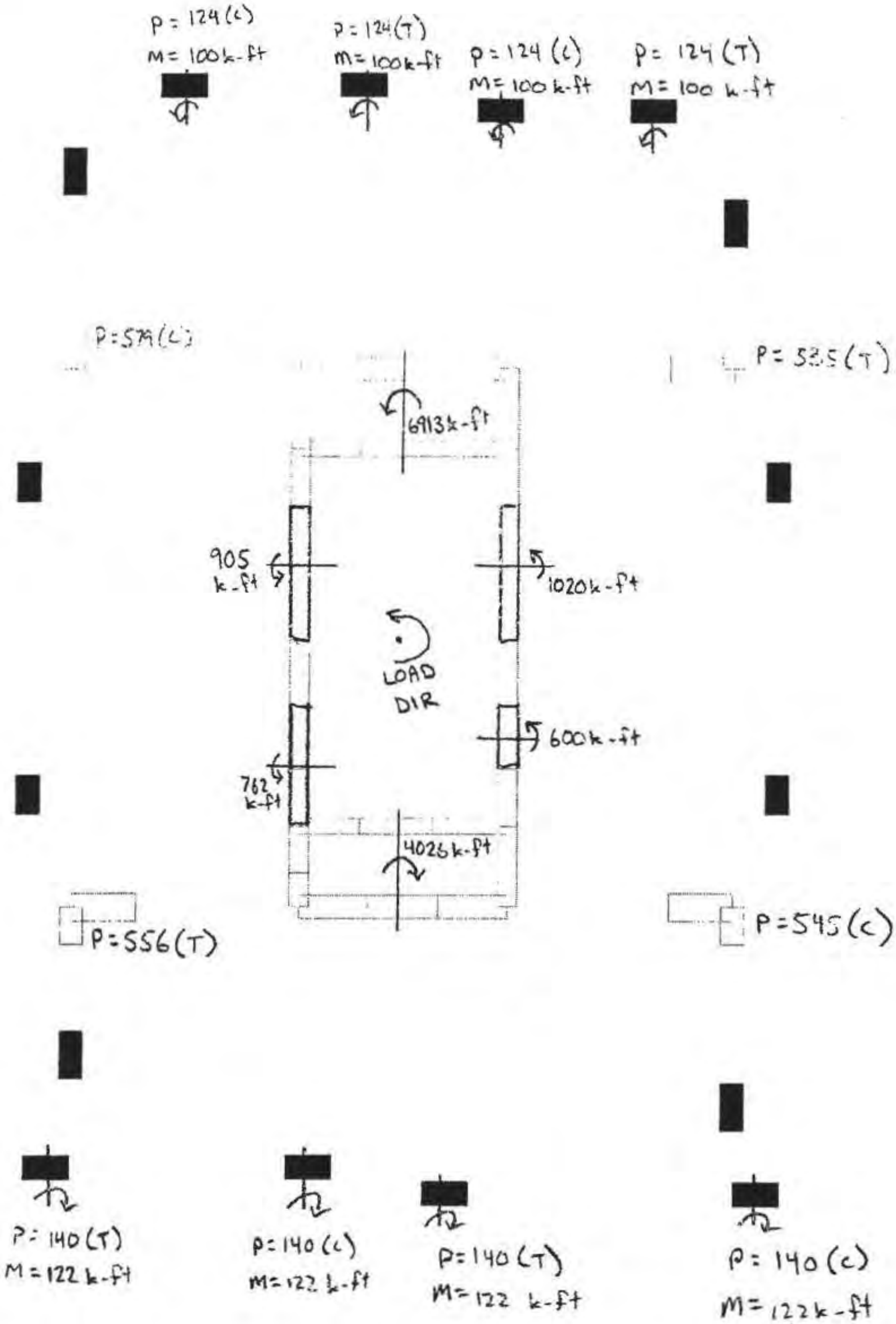
4.2-18

ETABS v8.5.0 - File: 4069-20050509-2.09-CD - May 12,2005 8:42
Plan View - GROUND - Elevation 0 - Kip-ft Units

ETABS

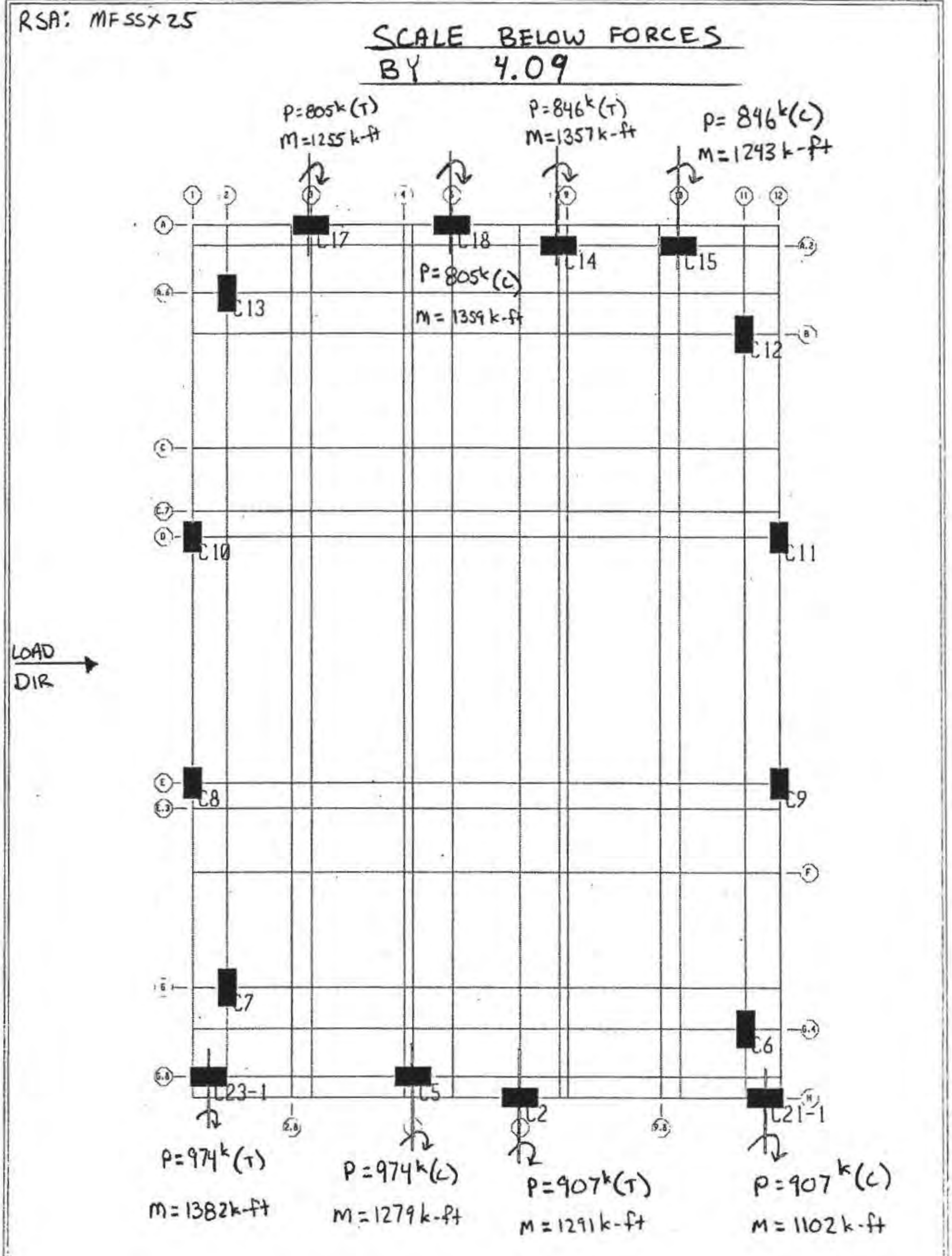
NJR

STATIC: MY



4.2-19

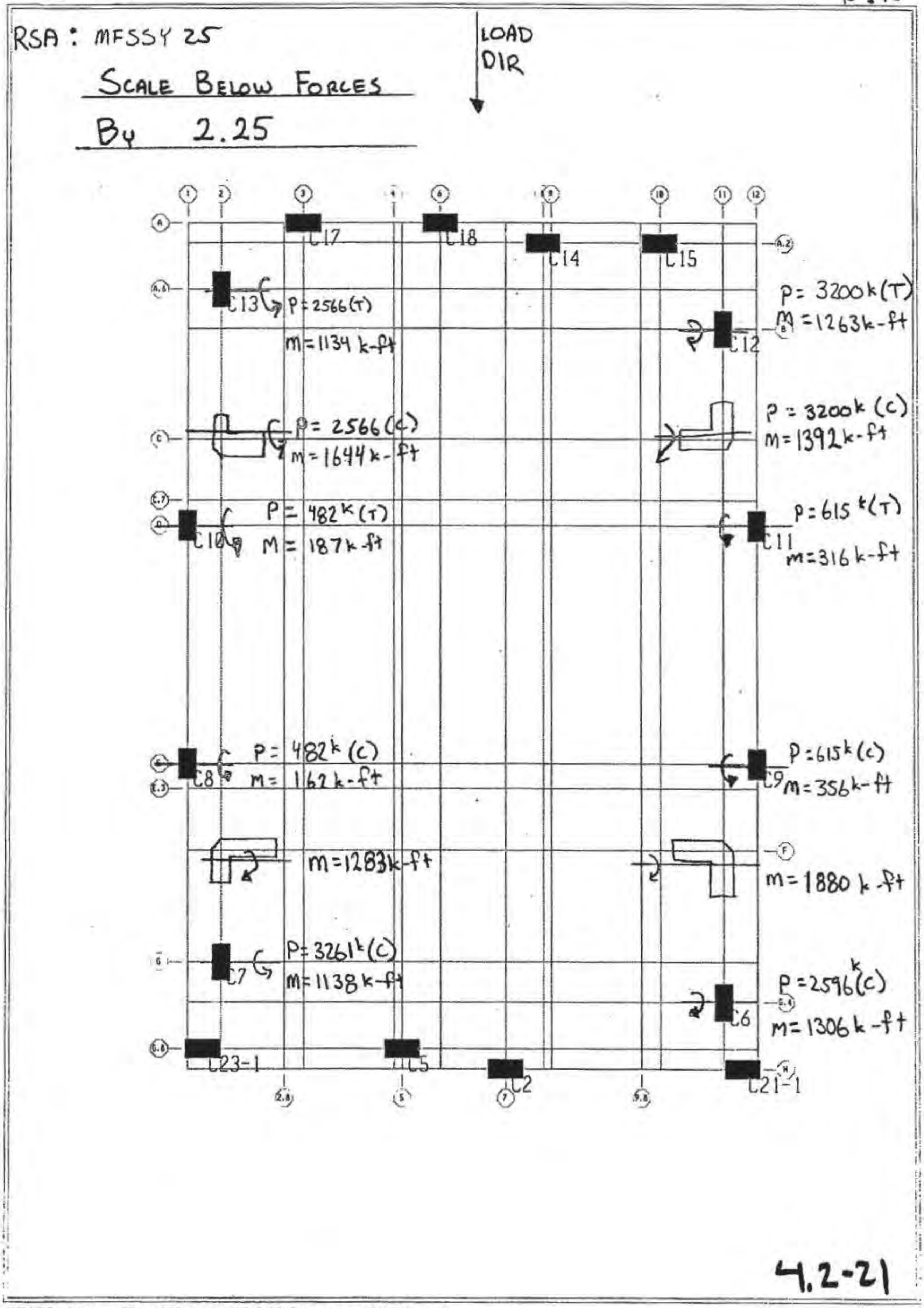
NJR



4.2-20

ETABS v8.5.0 - File: 4069-20050509-2.09-CD - May 12,2005 7:33
Plan View - GROUND - Elevation 0 - Kip-ft Units

ETABS



ETABS v8.5.0 - File: 4069-20050509-2.09-CD - May 12,2005 7:33
Plan View - GROUND - Elevation 0 - Kip-ft Units

4069-20050523-TR-Stiffness-DL-strip.OUT
S A F E (TM)

Version 8.0.0

23 May 2005 13:49:31

Tower Supported by Piles (No Piles Modeled)

G L O B A L   F O R C E   B A L A N C E

TOTAL FORCE AND MOMENT AT THE ORIGIN, IN GLOBAL COORDINATES

LOADDL ------------------

	FX	FY	FZ	MX	MY	MZ
APPLIED	.000000	.000000	-209627.000	1.5670E+07	1.1002E+07	.000000
SPRINGS	.000000	.000000	209612.707	-1.5669E+07	-1.1001E+07	.000000
TOTAL	.000000	.000000	-14.293292	1053.474	1130.149	.000000

LOADLL ------------------

	FX	FY	FZ	MX	MY	MZ
APPLIED	.000000	.000000	-21822.000	1.6945E+06	1.1486E+06	.000000
SPRINGS	.000000	.000000	21820.476	-1.6943E+06	-1.1485E+06	.000000
TOTAL	.000000	.000000	-1.523983	117.570809	121.626615	.000000

4.2-22

DODSONNOC0000259

S A F E (TM)

Version 8.0.0

It is the responsibility of the user to verify all results produced by this program

23 May 2005 13:34:01

U Program SAFE Version 8.0.0 File:4069-20050523-Tower-Pile-Stiffness-E-strip.OUT Page 1

Tower Supported by Piles (No Piles Modeled)

G L O B A L F O R C E B A L A N C E

TOTAL FORCE AND MOMENT AT THE ORIGIN, IN GLOBAL COORDINATES

LOADEX ------------------

	FX	FY	FZ	MX	MY	MZ
APPLIED	.000000	.000000	769.000000	✓-95600.250	1.8642E+06	.000000
SPRINGS	.000000	.000000	-771.346019	95747.301	-1.8639E+06	.000000
TOTAL	.000000	.000000	-2.346019	147.051495	302.213530	.000000

LOADEY ------------------

	FX	FY	FZ	MX	MY	MZ
APPLIED	.000000	.000000	-1063.000	✓-2.2547E+06	108512.250	.000000
SPRINGS	.000000	.000000	1061.452	2.2547E+06	-108412.744	.000000
TOTAL	.000000	.000000	-1.547839	-48.534692	99.506256	.000000

LOADMX ------------------

	FX	FY	FZ	MX	MY	MZ
APPLIED	.000000	.000000	46.000000	✓-817.333333	-4506.000	.000000
SPRINGS	.000000	.000000	-46.001769	815.132718	4506.660	.000000
TOTAL	.000000	.000000	-0.001769	-2.200615	0.659831	.000000

LOADMY ------------------

	FX	FY	FZ	MX	MY	MZ
APPLIED	.000000	.000000	-33.000000	✓611.666667	3197.083	.000000
SPRINGS	.000000	.000000	33.001189	-610.120549	-3197.543	.000000
TOTAL	.000000	.000000	0.001189	1.546118	-0.459745	.000000

LOADMFX ------------------

	FX	FY	FZ	MX	MY	MZ
APPLIED	.000000	.000000	.000000	✓-3.65E-11	469702.750	.000000
SPRINGS	.000000	.000000	-0.591796	37.925987	-469628.441	.000000
TOTAL	.000000	.000000	-0.591796	37.925987	74.309248	.000000

4069-20050523-Tower-Pile-Stiffness-E-strip.OUT

LOADMFY ------------------

	FX	FY	FZ	MX	MY	MZ
APPLIED	.000000	.000000	.000000	✓-703479.333	10227.000	.000000
SPRINGS	.000000	.000000	-0.429844	703461.288	-10198.358	.000000
TOTAL	.000000	.000000	-0.429844	-18.045692	28.642277	.000000

4.2-23

DODSONNOC0000260

4.3 Detailed Design

4.3 Detailed Design

One-Way Shear – 1-way shear in the pile cap is checked by inspecting the shear stress contours of the various load combinations. Typically, the pile cap is reinforced with #14@ 36" o.c. shear reinforcement. Directly under the core and the outrigger columns, the shear reinforcement is tightened to 24" o.c. This added shear capacity is adequate to resist seismic forces considered.

Two-Way Shear – 2-way shear in the pile cap is checked by hand. At failure, the piles within the critical perimeter are considered to take loads up to their capacity (with 1/3-increase for seismic cases) with excessive deflection; hence the force that contributes to the punching of the pile cap is the difference between the force from the vertical element and the capacity of the piles within the critical perimeter. Moments are also considered in the stress calculation. ASD level forces are used in the calculation and a modified phi-factor is used to account for both the strength reduction and the load amplification.

Flexure in T = 10' Region – The 10'-0" region is designed using the two SAFE models outlined in section 4.1, "Design Methodology and Assumptions."

Flexure in T = 3' Region – The 3'-0" region supports only gravity columns, and the design in done with SAFE as an integral part of the pile cap from which it cantilevers. A separate model was created to study the load case in which ground water pressure is present. This is not a controlling case for the design, and is included here only for completeness.

DESIMONE

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Item	TOWER FDN DESIGN – SHEAR	By	ML Ch'kd

SHEAR CAPACITY

CONCRETE ($d = 102''$)

$$V_c = 2\sqrt{5000} \times 12 \times 102 / 1000 = 173^k$$

$$\frac{\phi V_c}{1.428} = \frac{0.85 \times 173}{1.428} = \underline{103^k}$$

#14 @ 36" o.c., E.W.

$$A_v = 2.25 \text{ in}^2 / 3 \text{ ft} = 0.75 \text{ in}^2/\text{ft}$$

$$V_s = 0.75 \times 75 \times 102'' / 36'' = 159^k$$

$$\frac{\phi V_s}{1.428} = \frac{0.85 \times 159}{1.428} = \underline{95^k}$$

#14 @ 24" o.c., E.W

$$A_v = 2.25 \text{ in}^2 / 2 \text{ ft} = 1.125 \text{ in}^2/\text{ft}$$

$$V_s = 1.125 \times 75 \times 102/24 = 359^k$$

$$\frac{\phi V_s}{1.428} = \frac{0.85 \times 359}{1.428} = \underline{213^k}$$

#14 @ 18" o.c. E.W.

$$A_v = 2.25 \text{ in}^2 / 1.5 \text{ ft} = 1.5 \text{ in}^2/\text{ft}$$

$$V_s = 1.5 \times 75 \times 102/18 = 638^k$$

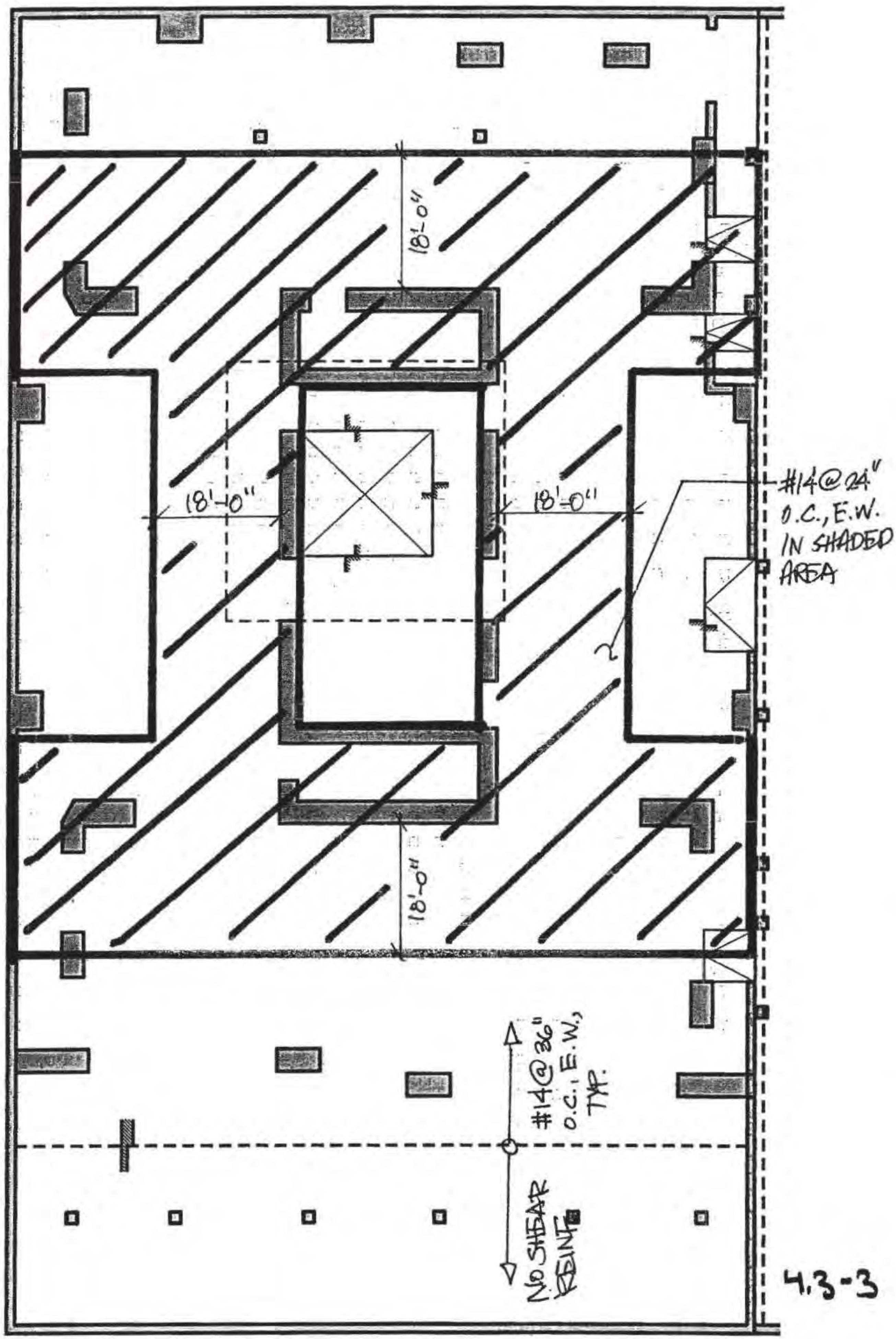
$$\frac{\phi V_s}{1.428} = \frac{0.85 \times 638}{1.428} = \underline{379^k}$$

HENCE CONC w/ #14 @ 36" o.c., E.W. : 198^k per ft

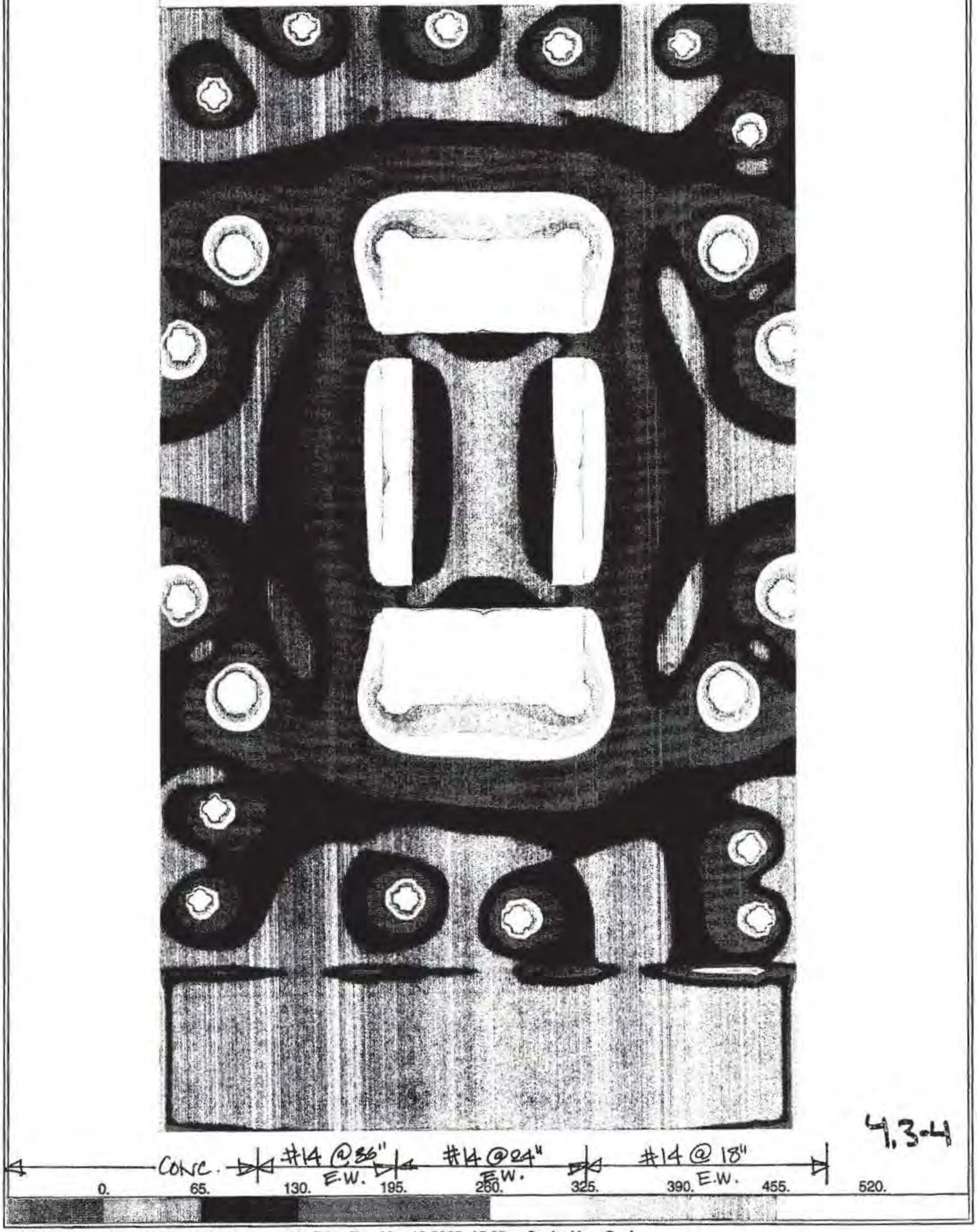
CONC w/ #14 @ 24" o.c., E.W : 316^k per ft

CONC w/ #14 @ 18" o.c., E.W. : 482^k per ft

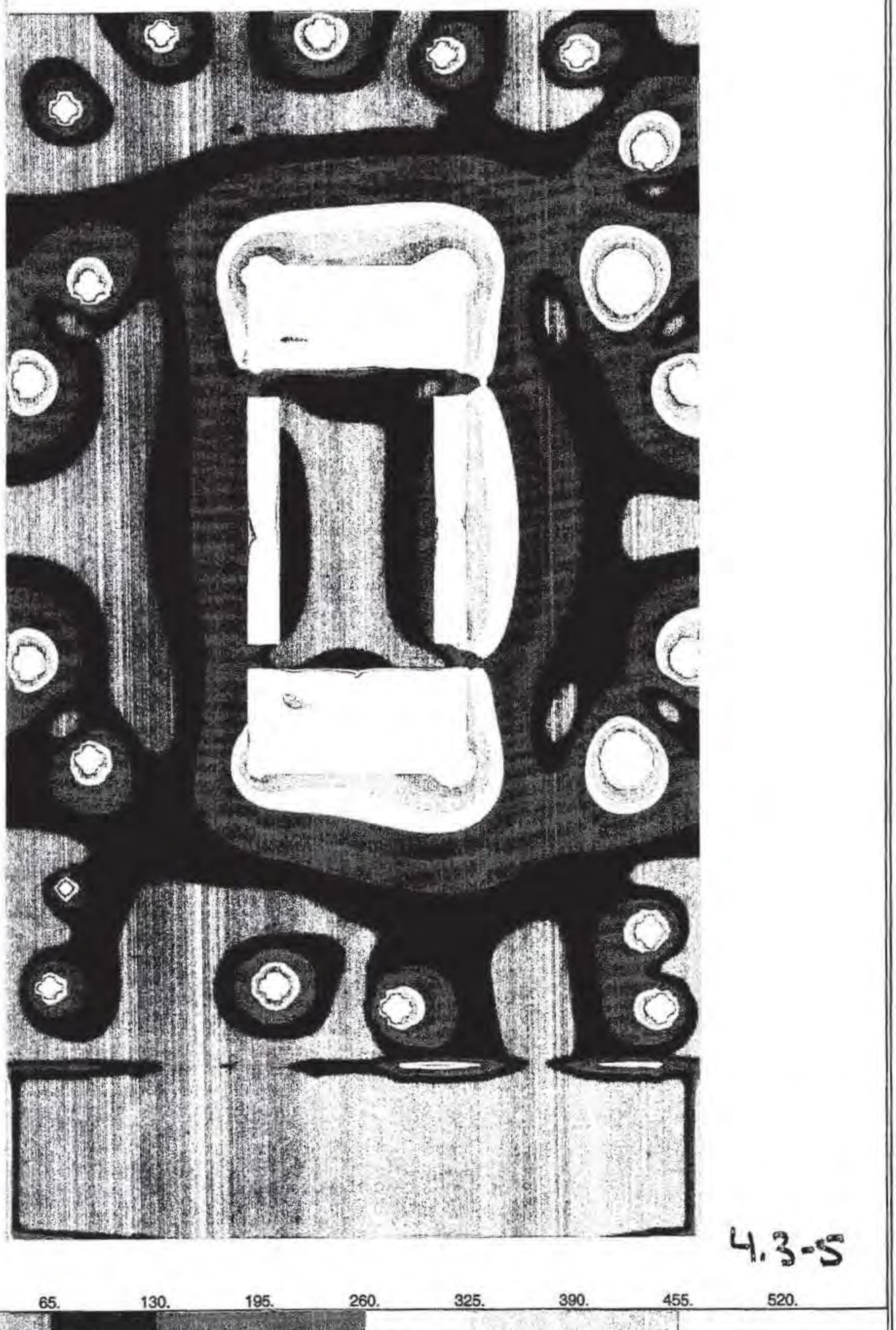
4.3-2



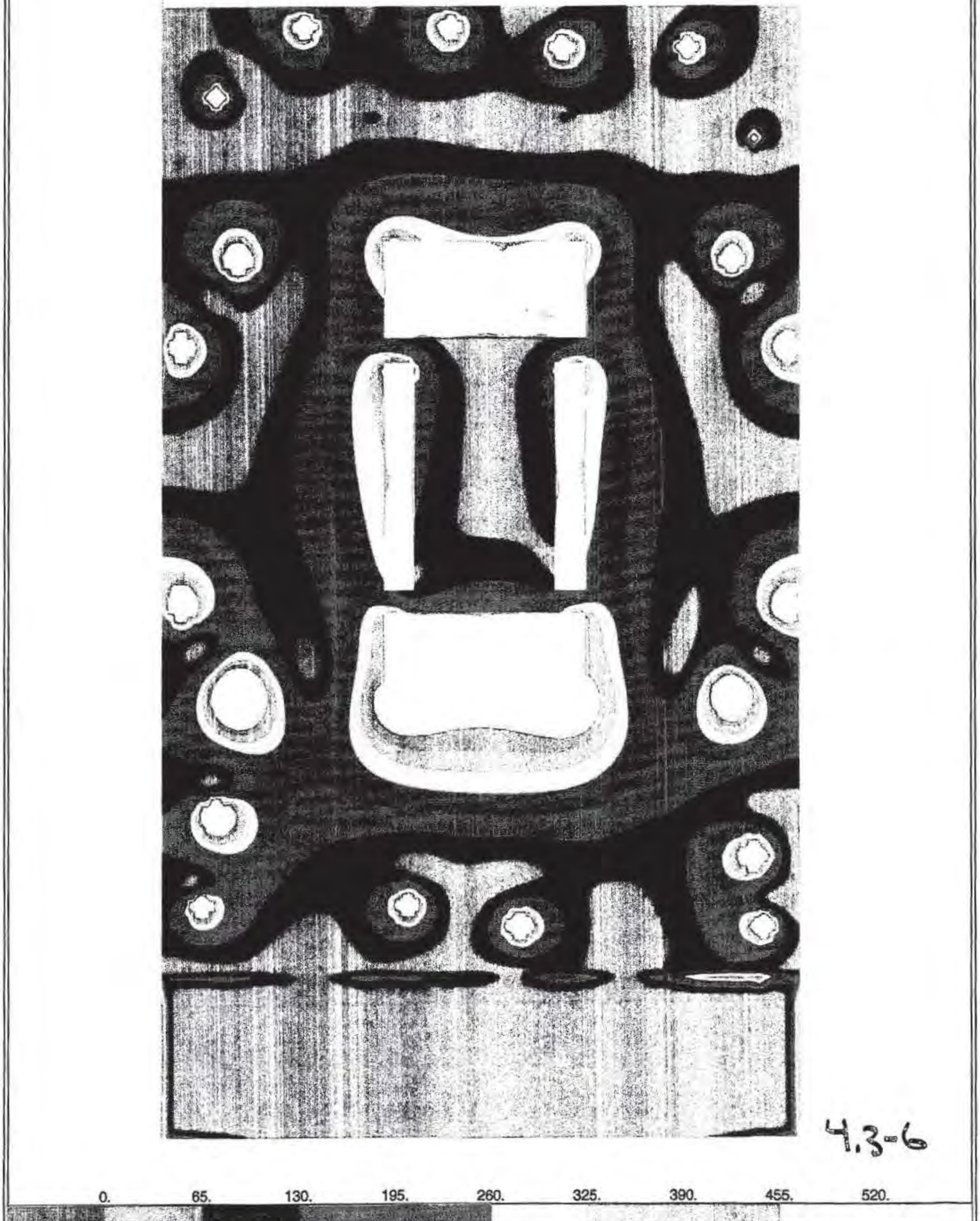
18'-0"
18'-0"
18'-0"
18'-0"
#14@24" O.C., E.W. IN SHADED AREA
#14@36" O.C., E.W., TYP.
NO SHEAR REINF.
4.3-3



SAFE v8.0.6 - File: 4069-20050518-Tower-No-Piles-E - May 19,2005 15:25 - Scale: User Scale
Slab Resultant Vmax Diagram - (1) - Kip-ft Units



SAFE v8.0.6 - File: 4069-20050518-Tower-No-Piles-E - May 19,2005 16:53 - Scale: User Scale
Slab Resultant Vmax Diagram - (2A1X1) - Kip-ft Units



SAFE v8.0.6 - File: 4069-20050518-Tower-No-Piles-E - May 19,2005 16:54 - Scale: User Scale
Slab Resultant Vmax Diagram - (2A1Y8) - Kip-ft Units

DESIMONE

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Item	TOWER FDN DESIGN - SHEAR	By ML	Ch'kd

PUNCHING SHEAR CAPACITY

CONCRETE (d = 102") $V_c = 4\sqrt{5000} = 283$ psi

#14 @ 36" O.C., E.W $A_v = 2.25\ in^2 / 3\ ft = 0.75\ in^2/ft$

$V_s = 0.75 \times 75 \times 102/36 = 159$ psi

$V_c + V_s = 283 + 159 = 442$ psi

$$\frac{\phi(V_c + V_s)}{1.428} = \frac{0.85 \times 442}{1.428} = \underline{\underline{263\ psi}}$$

4.3-7

DESIMONE

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Item: TOWER FDN DESIGN - SHEAR | By: ML. | Ch'kd ____

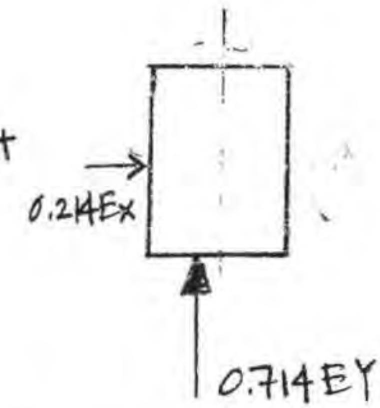
BOX WALL PUNCHING SHEAR

- Within d/2 from face of wall, has 66 piles
- Force in excess of pile capacity within b_o → punching.

<u>NORTH BOX</u>

Controlling case: 2A1Y3

D + L +



PILE CAPACITY = $66 \times 260^k = 17{,}160^k$

$V = 47{,}246^k - 4/3(17{,}160^k) = 24{,}366^k$

$M_X = 27{,}839^{k\text{-}1}$ $\qquad M_Y = 39{,}932^{k\text{-}1}$

$v = 185$ psi $\qquad DCR = \frac{185}{263} = \underline{\underline{0.70}}$ o.k

4.3-8

DESIMONE

Project 301 MISSION ST

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Item TOWER FDN DESIGN - SHEAR

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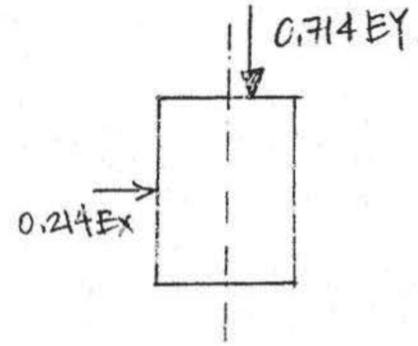
SOUTH BOX

Controlling case : 2A1Y5

D + L +



PILE CAPACITY = 66×260^k = 17,160^k

V = 46,551^k − 4/3(17,160^k) = 23,671^k

Mx = 24,860^{k-1} My = 31,100^{k-1}

v = 177 psi DCR = $\frac{177}{263}$ = 0.67 O.K.

4.3-9

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Item TOWER FDN DESIGN - SHEAR

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OUTRIGGER COLUMNS PUNCHING SHEAR

controlling case : NW column - 2A1 x 7

Pile capacity = 15 × 260^k = 3900^k

V = 15,003^k - 4/3 × 3900^k = 9808^k

Mx = 366$^{k-'}$ MY = 5479$^{k-'}$

v = 138 psi DCR = $\frac{138}{263}$ = 0.52 o.k.

MOMENT FRAME COLUMNS PUNCHING SHEAR

controlling case : col 2-G - 2B1Y2.

Pile capacity = 6 × 260^k = 1560^k

V = 9507 - 4/3 × 1560 = 7427^k

Mx = 1829$^{k-'}$ MY = 0

v = 119 psi DCR = $\frac{119}{263}$ = 0.45 o.k.

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Item: TOWER FDN DESIGN − T = 3'
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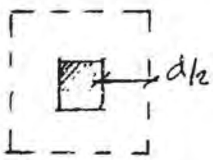
By: ML Ch'kd ______

MAT T = 3' FOR PUNCHING SHEAR

COLUMN $P_u = 1.4 \times 85 + 1.7 \times 89 = 271^k$

COLUMN SIZE : 18" × 18"

mat $d = 30''$



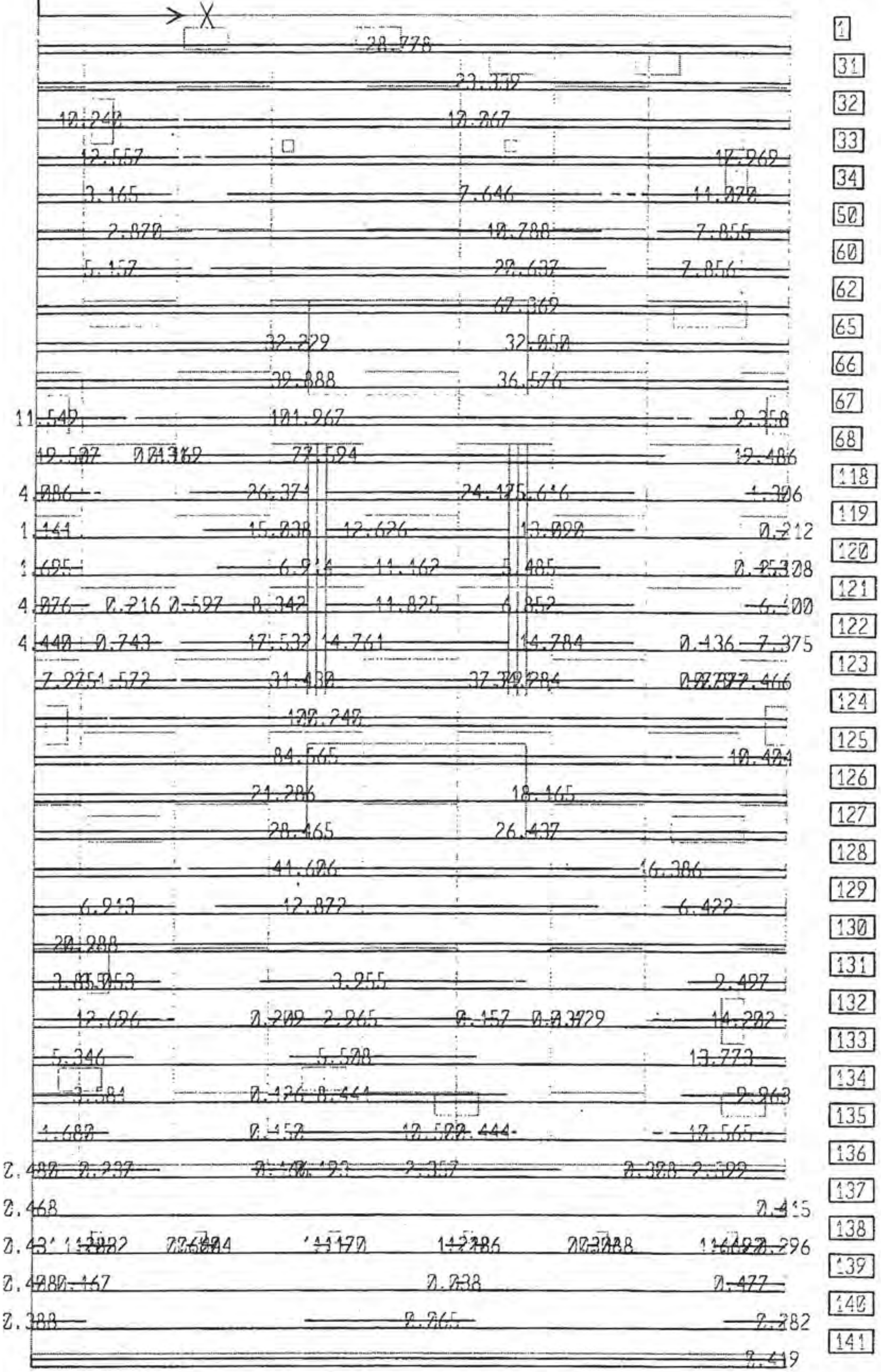
$b_o = (18 + 30) \times 4 = 192''$

$\phi V_c = 0.85 \times 4 \sqrt{5000} \times 192'' \times 30'' / 1000$

$= 1385^k >> P_u$ O.K.

4.3-11

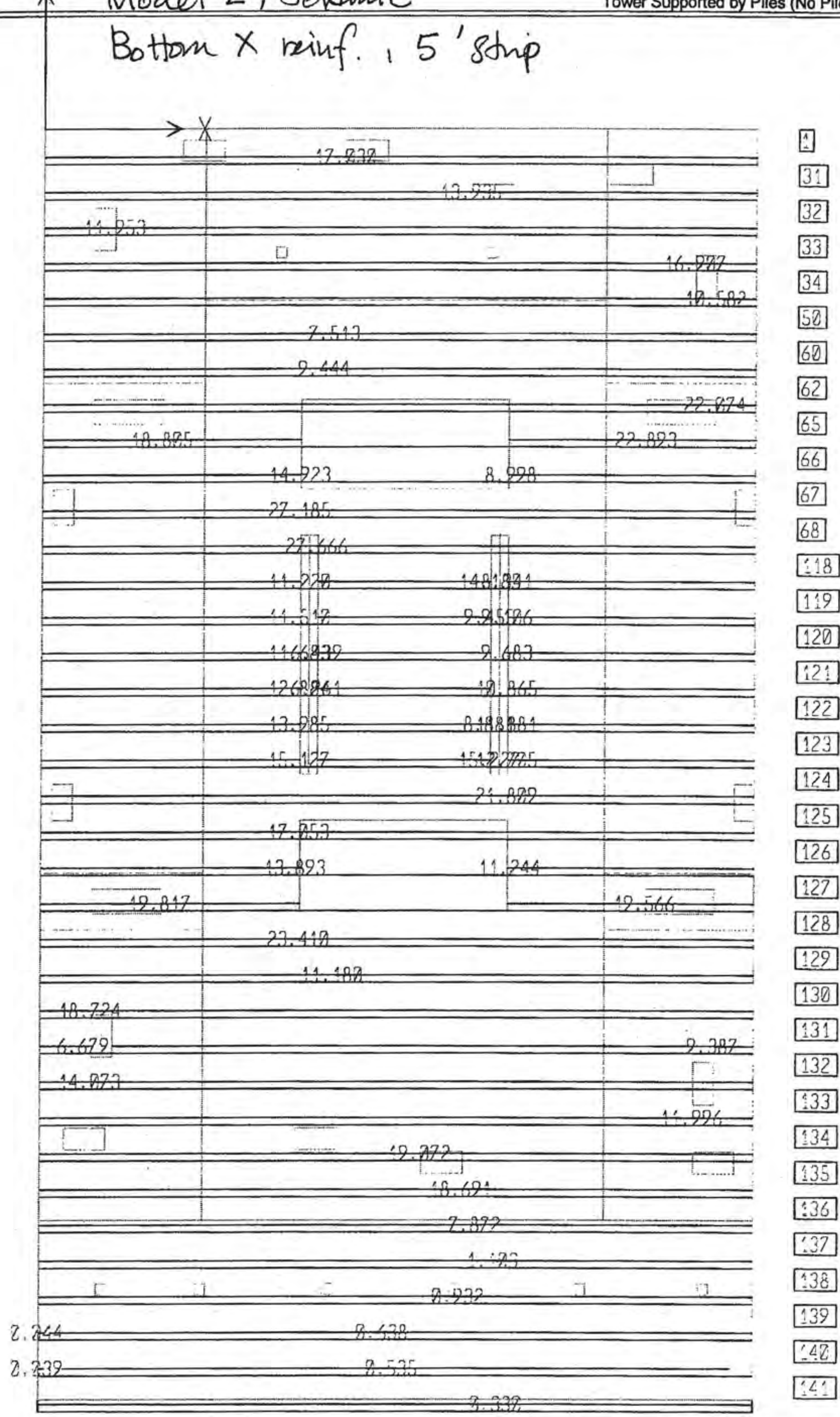
Model 1, Gravity
Bottom x reinf. (5' strip)



4.3-12

Model 2, Seismic

Bottom X reinf., 5' strip

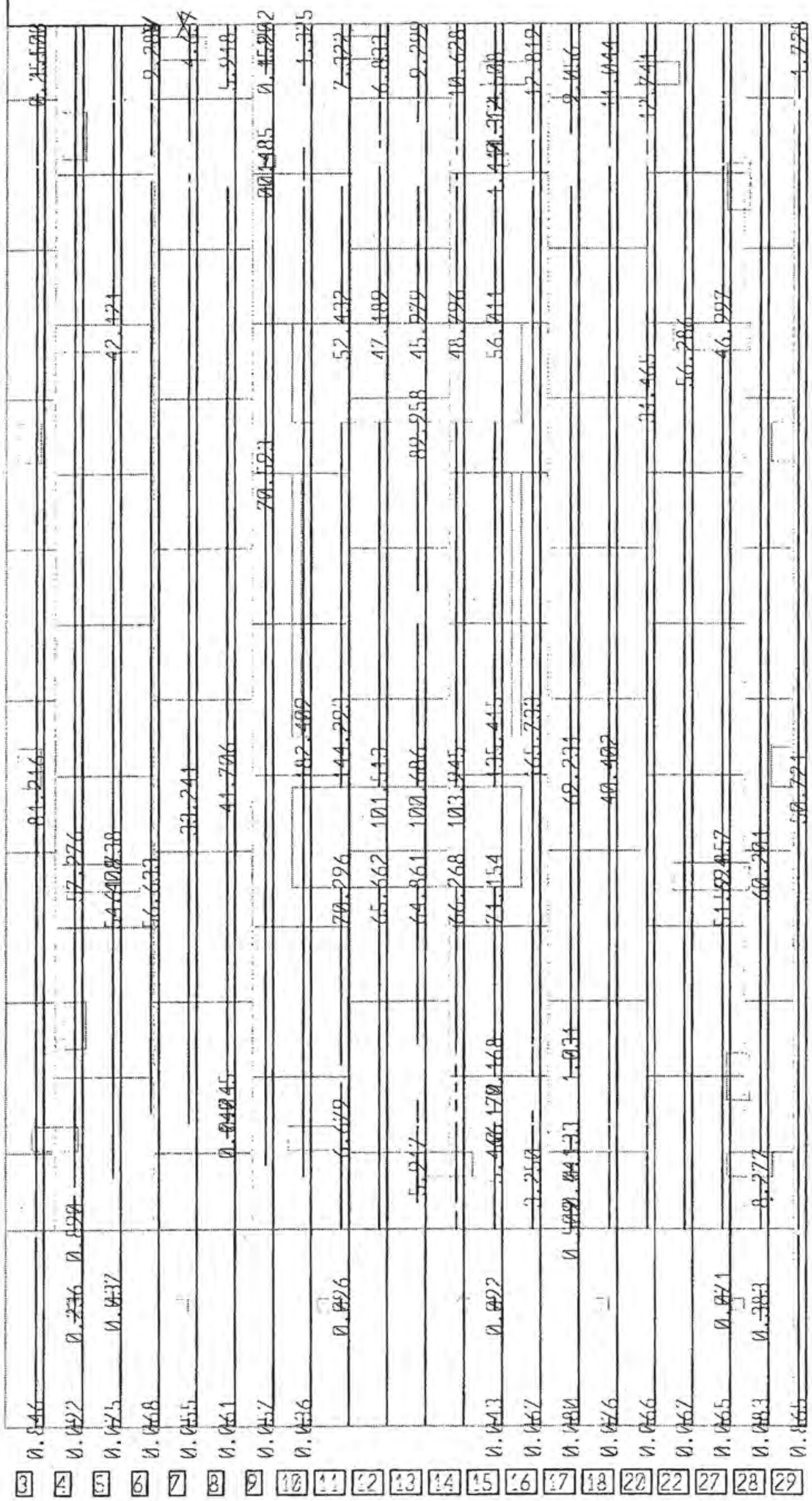


4.3-B

SAFE v8.0.6 - File: 4069-20050523-Tower-Pile-Stiffness-E-strip - May 24,2005 9:41 - Scale: User Scale
Bottom X-Strip Reinforcement (Sq-in) - Kip-ft Units

Model, Gravity

Bottom Y reinf., (5' Strip)

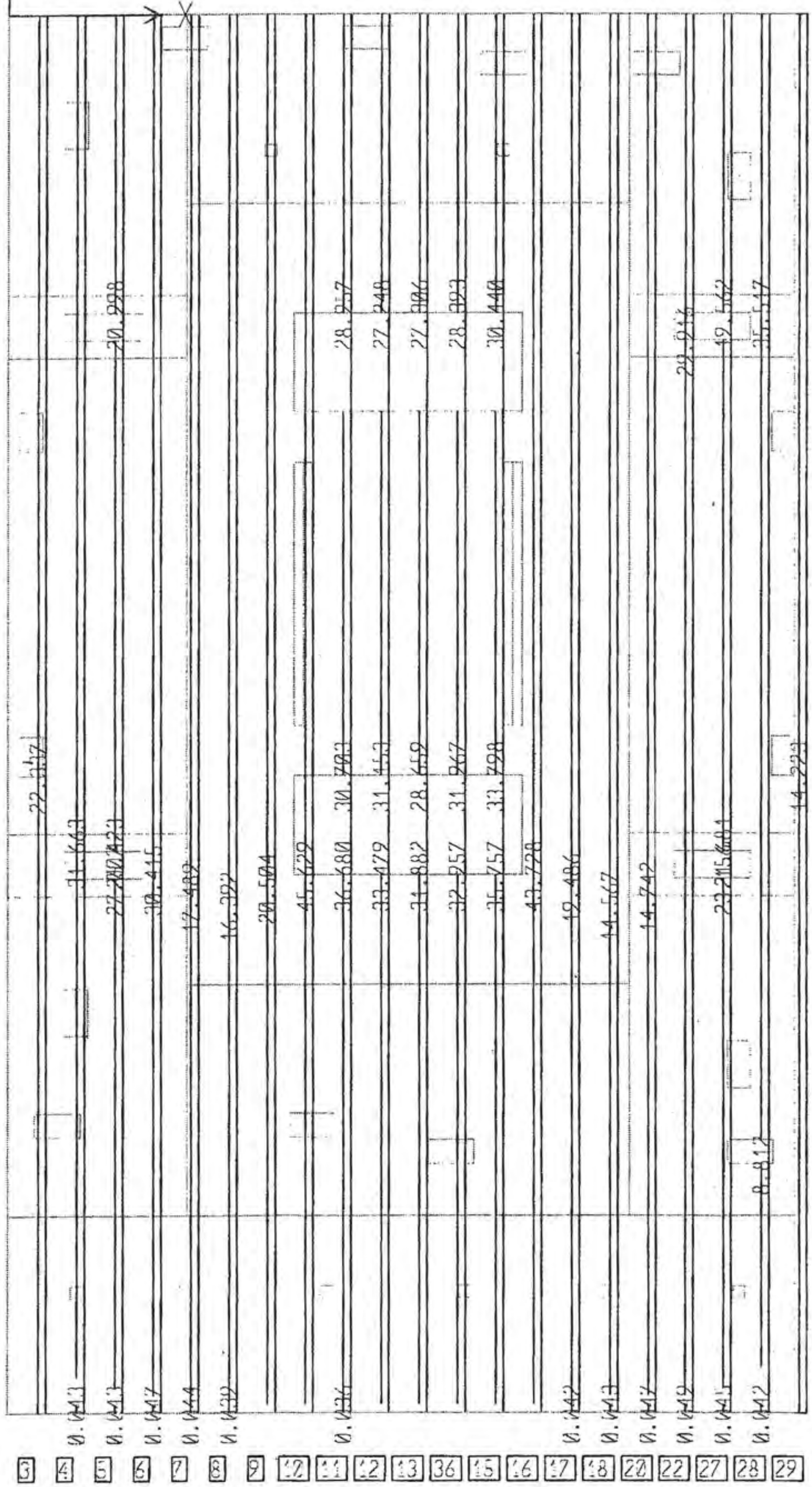


4.3-4

SAFE v8.0.6 - File: 4069-20050523-TR-Stiffness-DL-strip - May 24,2005 9:40 - Scale: User Scale
Bottom Y-Strip Reinforcement (Sq-in) - Kip-ft Units

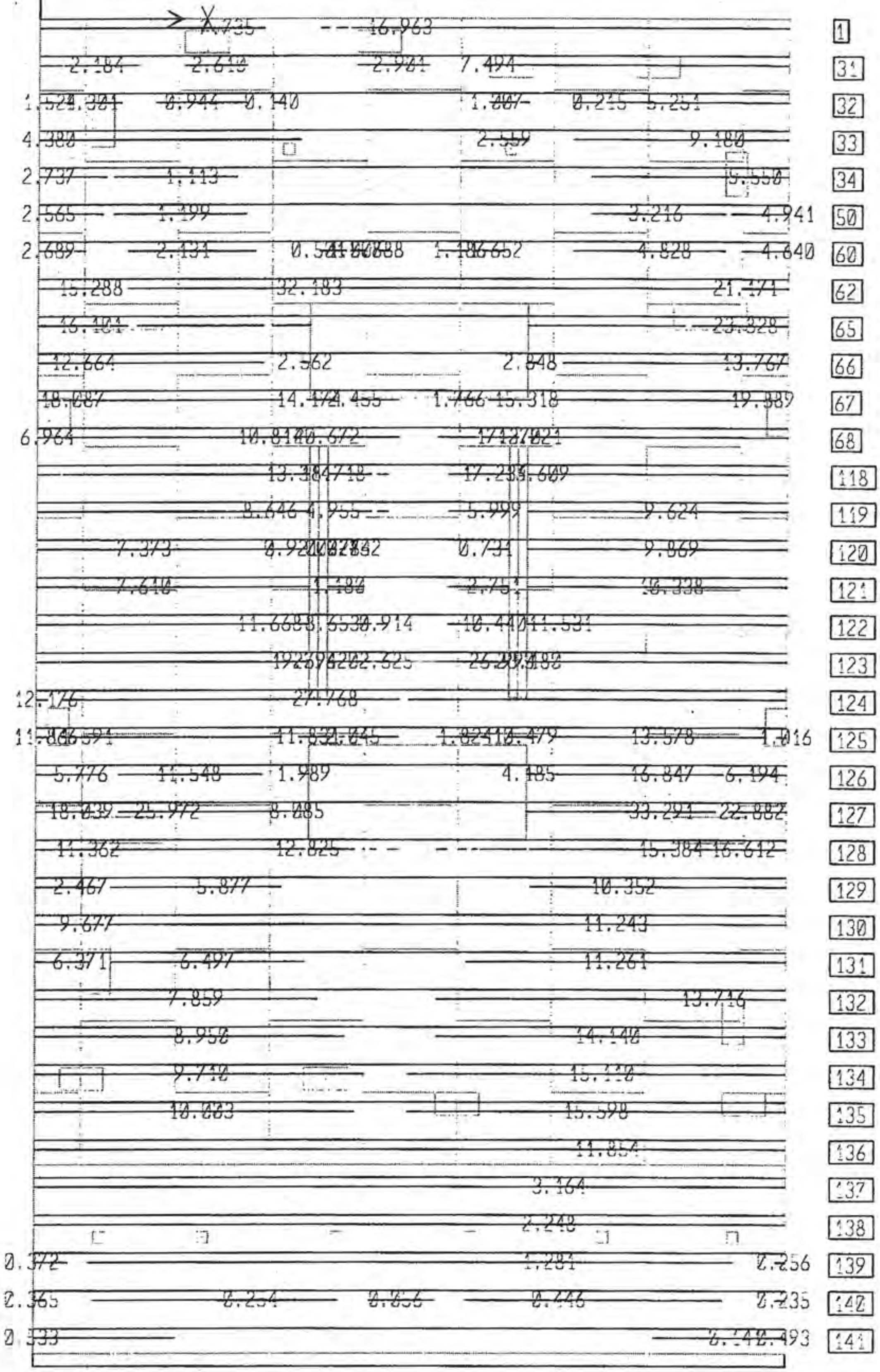
Model 2, Seismic

Bottom Y reinf., 5' strip



4.3-15

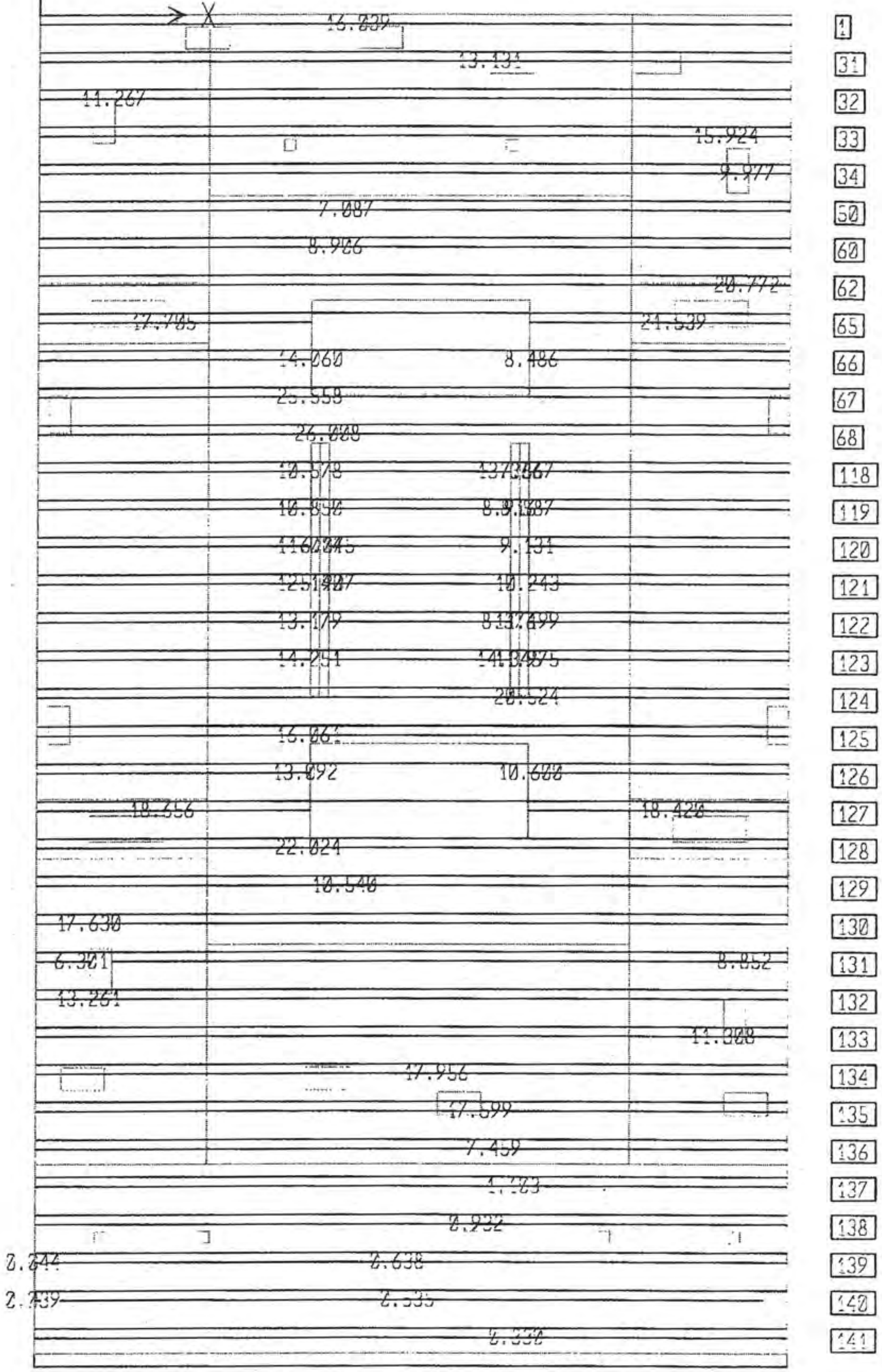
Model 1, Gravity
Top X reinf., 5' Strip



4.3-16

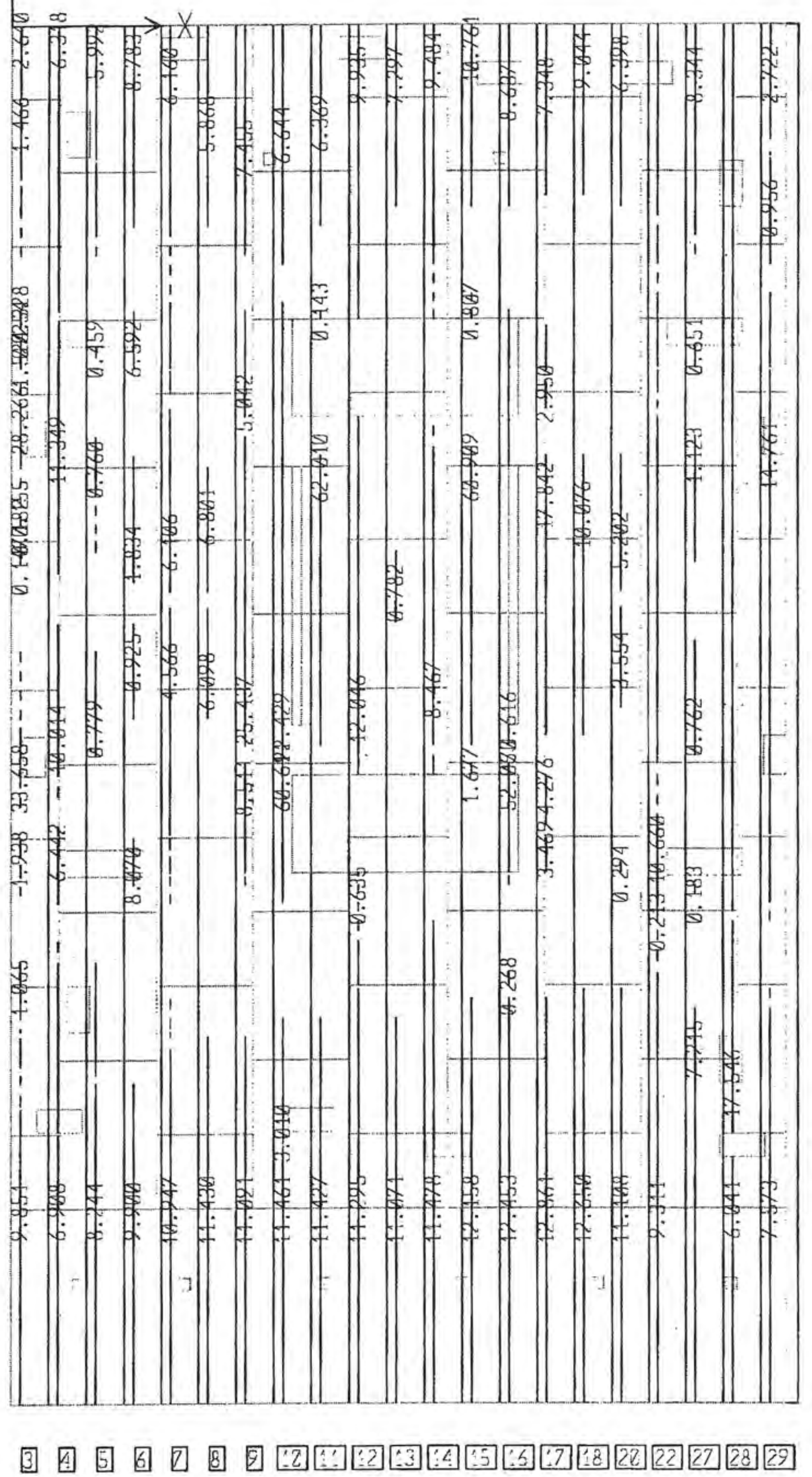
SAFE v8.0.6 - File: 4069-20050523-TR-Stiffness-DL-strip - May 24,2005 9:40 - Scale: User Scale
Top X-Strip Reinforcement (Sq-in) - Kip-ft Units

Model 2, Seismic
Top X reinf., 5' Strip



4.3-17

Model 1, Gravity,
Top Y reinf., 5 ft strip



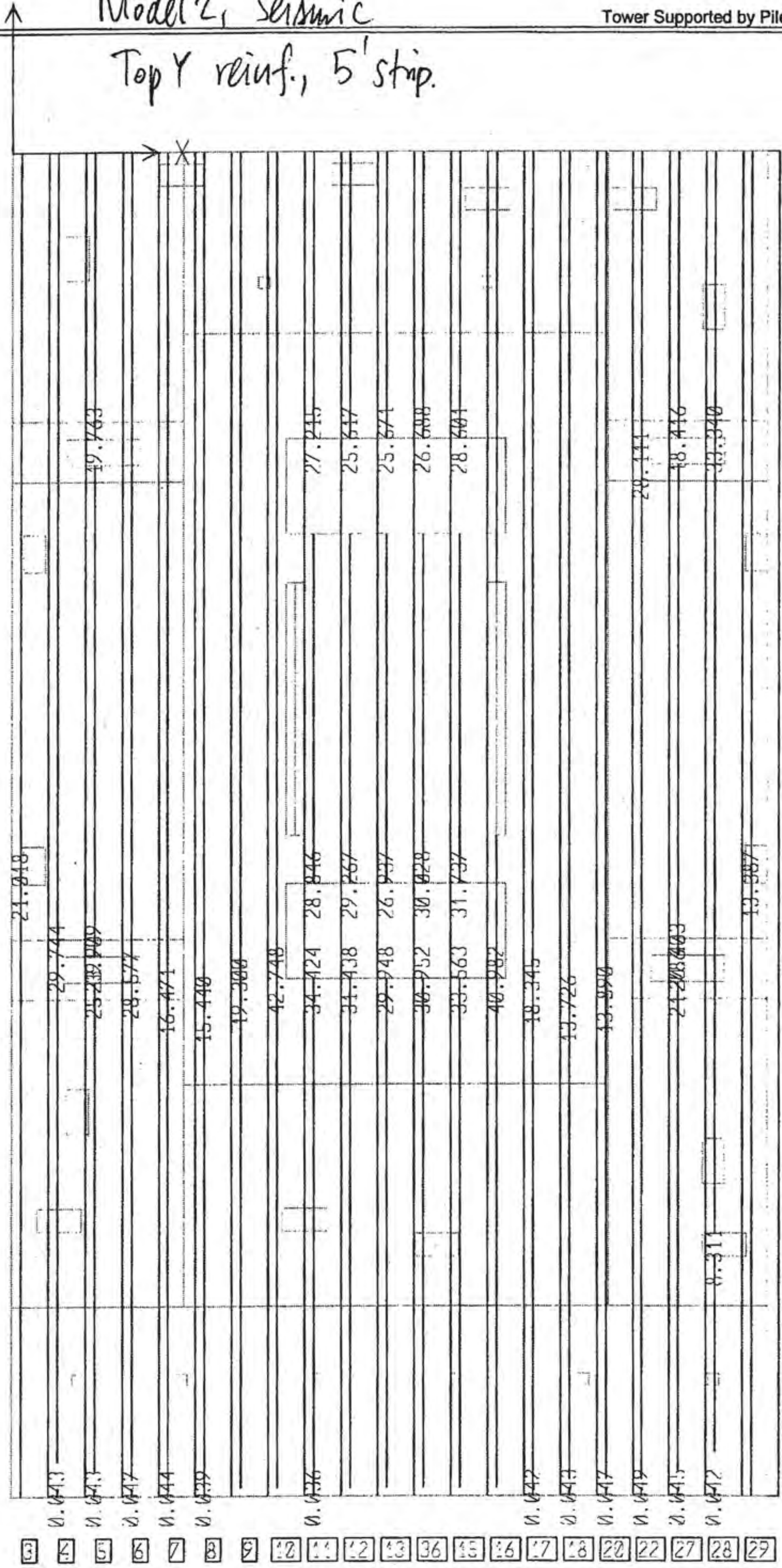
4.3-18

SAFE v8.0.6 - File: 4069-20050523-TR-Stiffness-DL-strip - May 24,2005 9:40 - Scale: User Scale
Top Y-Strip Reinforcement (Sq-in) - Kip-ft Units

SAFE

Model 2, Seismic

Top Y reinf., 5' strip.



4,3-19

SAFE v8.0.6 - File: 4069-20050523-Tower-Pile-Stiffness-E-strip - May 24,2005 9:41 - Scale: User Scale
Top Y-Strip Reinforcement (Sq-in) - Kip-ft Units

DESIMONE

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Project No.	4069	Date	5/18/05
Item	TOWER FDN DESIGN – T=3'	By ML	Ch'kd

MAT T=3' FOR HYDROSTATIC PRESSURE

wt of mat = 450 psf

H = 13.14 ft × 62.4 psf = 820 psf

Load Combo   ASD : 0.9D + H

STRENGTH : 0.9D + 1.7H

WORST CASE IS D=0 → $\frac{0.9D + 1.7H}{0.9D + H} = \underline{1.7}$

SCALE FACTOR = 1.7

Modify φ Factor   SHEAR = $\frac{0.85}{1.7} = 0.5$

FLEXURE = $\frac{0.90}{1.7} = 0.53$

NOTE: THIS IS NOT THE CONTROLING CASE FOR DESIGN OF THE T=3' PORTION.
CALCULATIONS INCLUDED HERE FOR COMPLETENESS.

4.3-20

4069-20050518-Tower-No-Piles-E-3ft.OUT
S A F E (TM)

Version 8.0.0

U Program SAFE Version 8.0.0 19 May 2005 11:08:32 File:4069-20050518-Tower-No-Piles-E-3ft.OUT Page 1

Tower Supported by Piles (No Piles Modeled)

G L O B A L   F O R C E   B A L A N C E

TOTAL FORCE AND MOMENT AT THE ORIGIN, IN GLOBAL COORDINATES

LOADDL ------------------

	FX	FY	FZ	MX	MY	MZ
APPLIED	.000000	.000000	-209627.000	1.5670E+07	1.1002E+07	.000000
SPRINGS	.000000	.000000	209626.919	-1.5670E+07	-1.1002E+07	.000000
TOTAL	.000000	.000000	-0.081391 ✓	3.479632	3.716033	.000000

LOADLL ------------------

	FX	FY	FZ	MX	MY	MZ
APPLIED	.000000	.000000	-21822.000	1.6945E+06	1.1486E+06	.000000
SPRINGS	.000000	.000000	21821.992	-1.6945E+06	-1.1486E+06	.000000
TOTAL	.000000	.000000	-0.007858 ✓	0.336135	0.358979	.000000

LOADMAT ------------------

	FX	FY	FZ	MX	MY	MZ
APPLIED	.000000	.000000	-24949.271	2.0165E+06	1.2901E+06	.000000
SPRINGS	.000000	.000000	24949.262	-2.0165E+06	-1.2901E+06	.000000
TOTAL	.000000	.000000	-0.008476 ✓	0.362520	0.387530	.000000

LOADH ------------------

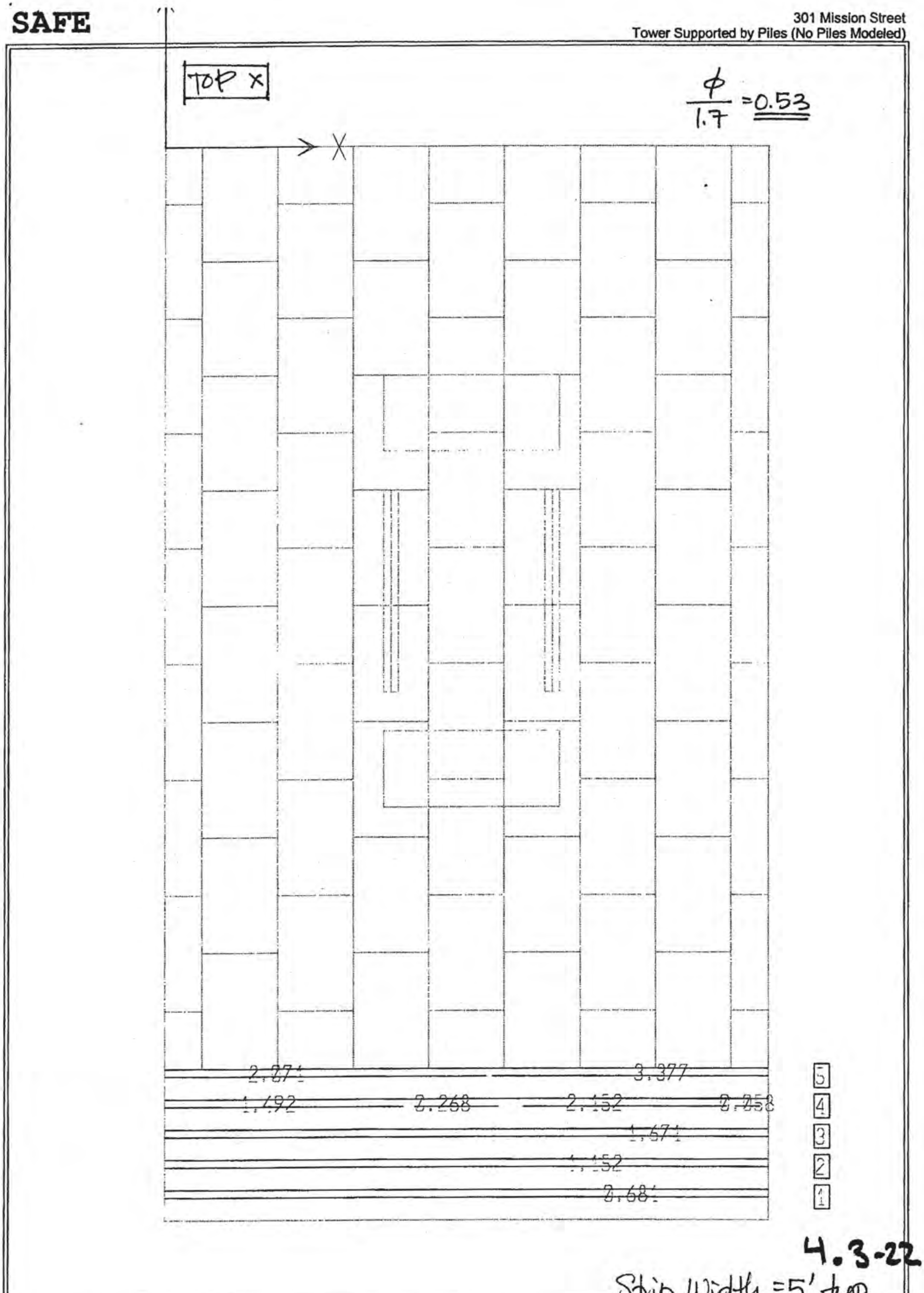
	FX	FY	FZ	MX	MY	MZ
APPLIED	.000000	.000000	22052.570	-1.8797E+06	-1.1403E+06	.000000
SPRINGS	.000000	.000000	-22052.563	1.8797E+06	1.1403E+06	.000000
TOTAL	.000000	.000000	0.006848 ✓	-0.292968	-0.313446	.000000

10' mat area = 15,857 sf
3' mat area = 2,585 sf

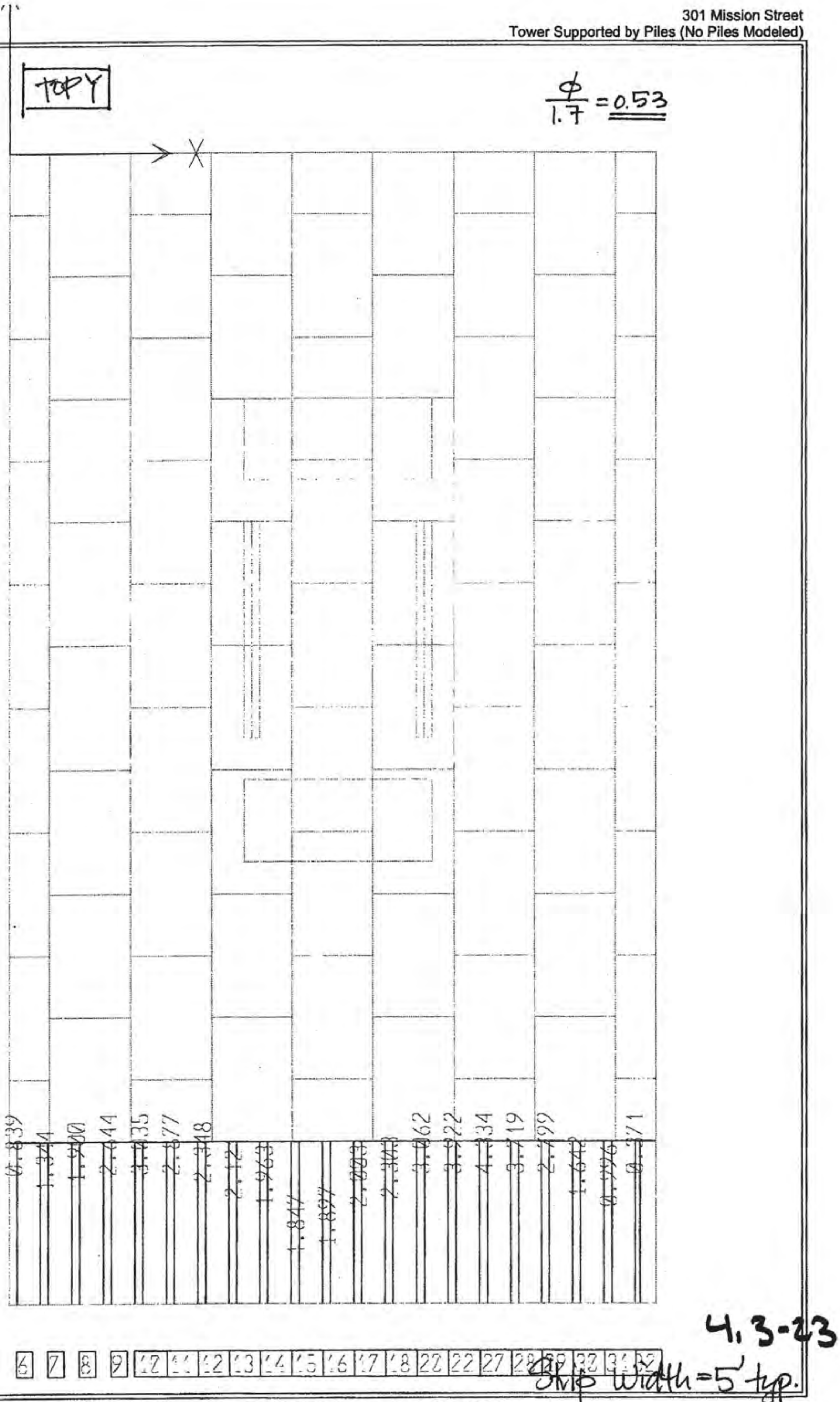
MAT = 15,857 sf × 1.5 ksf + 2,585 sf × 0.45 ksf
= 24,949 k

H = 15,857 sf × 1.257 ksf + 2,585 sf × 0.82 ksf
= 22,052 k

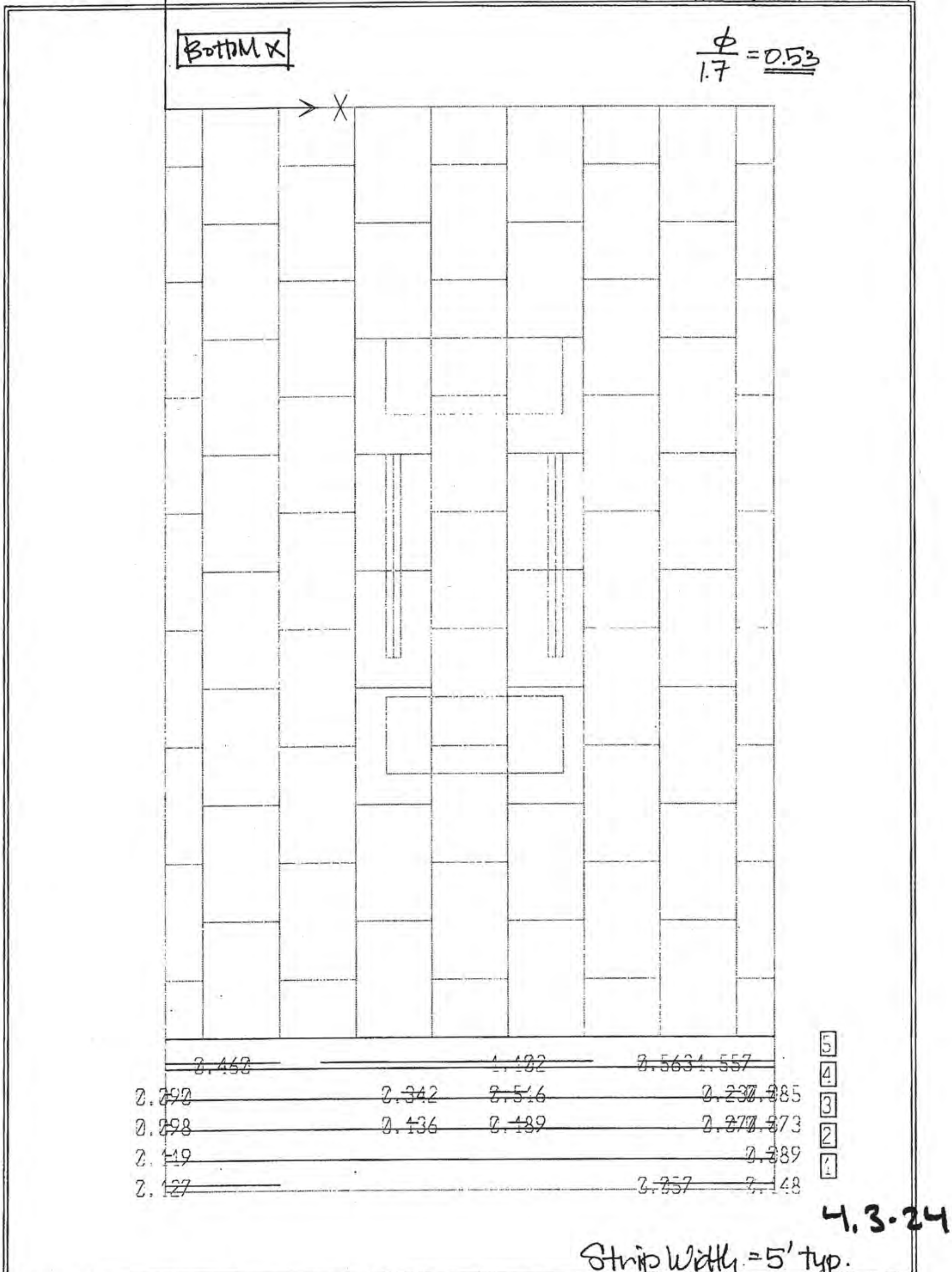
4.3-4



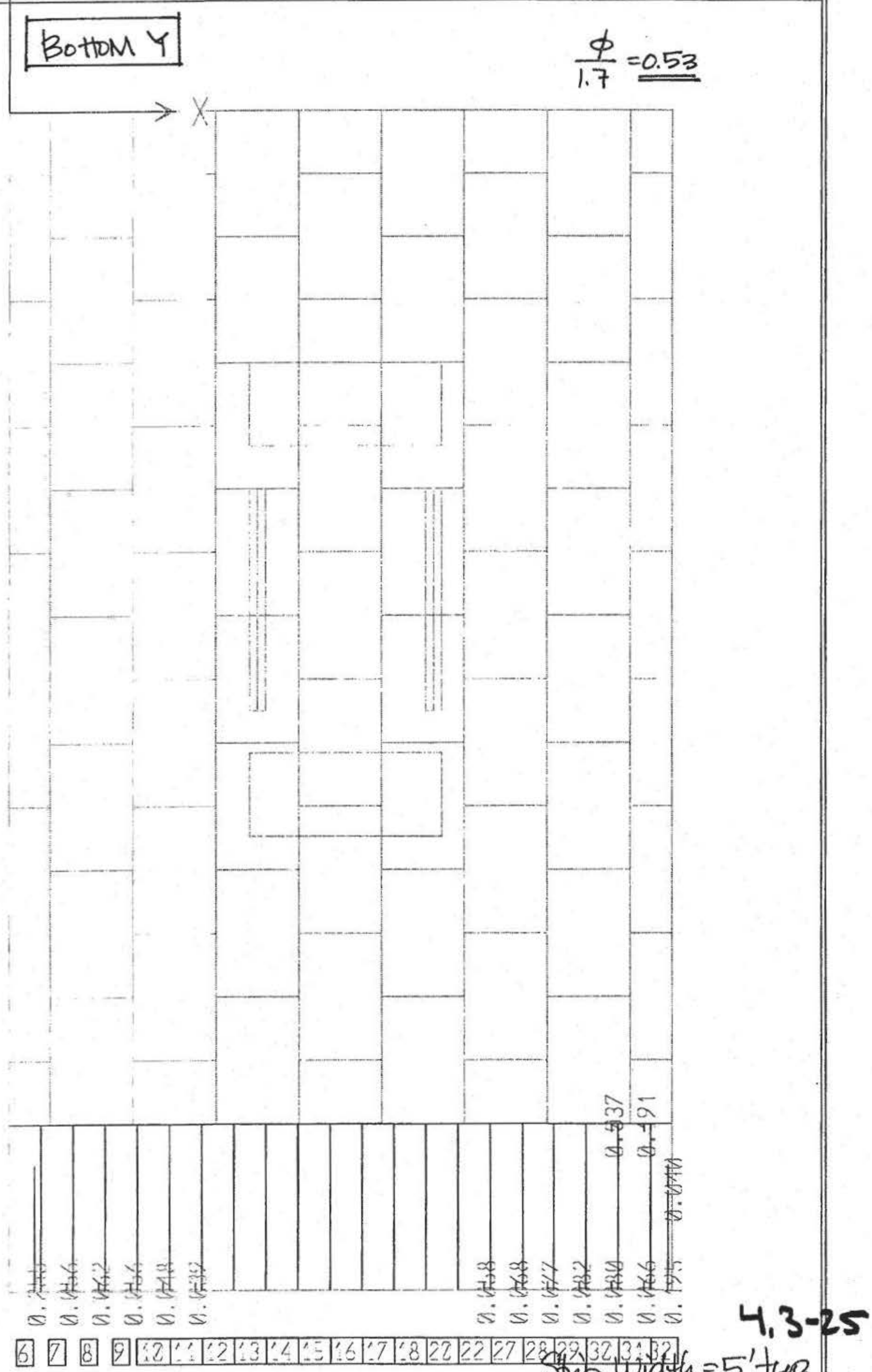
SAFE v8.0.6 - File: 4069-20050518-Tower-No-Piles-E-3ft - May 19,2005 11:43 - Scale: User Scale
Top X-Strip Reinforcement (Sq-in) - Kip-ft Units



SAFE v8.0.6 - File: 4069-20050518-Tower-No-Piles-E-3ft - May 19,2005 11:44 - Scale: User Scale
Top Y-Strip Reinforcement (Sq-in) - Kip-ft Units



SAFE v8.0.6 - File: 4069-20050518-Tower-No-Piles-E-3ft - May 19,2005 11:43 - Scale: User Scale
Bottom X-Strip Reinforcement (Sq-in) - Kip-ft Units



SAFE v8.0.6 - File: 4069-20050518-Tower-No-Piles-E-3ft - May 19,2005 11:43 - Scale: User Scale
Bottom Y-Strip Reinforcement (Sq-in) - Kip-ft Units

SECTION 5 – TOWER PERIMETER BASEMENT WALLS

5.1 North and West Perimeter Wall

5.1 North and West Perimeter Wall

The north and west perimeter walls are the same in geometry and extend from the ground floor down to level B1. The walls are 15'-9" high and braced at the ground floor at the top. The walls are 14" thick for the entire height.

One wall representing the north and west walls is modeled and analyzed using the computational program, RISA. Loads applied to the wall include the permanent and seismic soil pressure along the height of the wall. A traffic surcharge is also applied along the top 10 feet of the wall. The wall is assumed to be fixed at the base (level B1) and pinned at each level and at the top (ground floor).

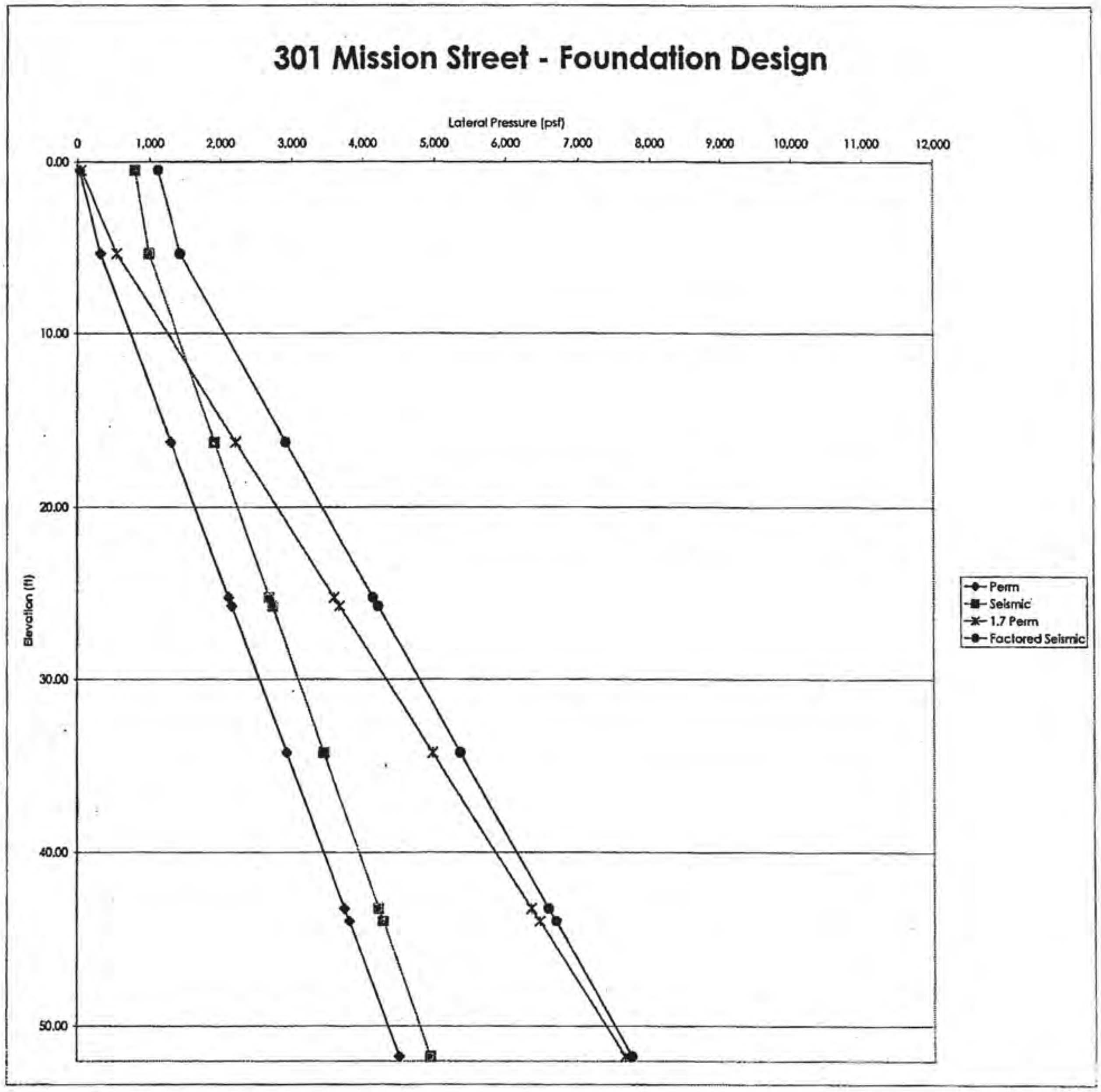
The shear in the wall due to the out-of-plane loads is checked assuming the concrete shear capacity is sufficient to take applied shear. Horizontal shear reinforcement is required for resisting the in-plane loads along the wall. The required vertical flexural reinforcement is designed for both the interior and soil faces based on the maximum moments obtained from the RISA analysis.

Lateral Earth Pressure Restrained Wall Condition
Ground Elev. = 0'-0", Design Ground Water Elev. = -5.3

	Static	Seismic	
Above -5.36	60	40	15H
Below -5.36	90	85	15H

Negative Elevation (ft)	Perm Pressure (psf)	Force (k)	1.7 Perm Pressure (psf)
0.50	30	854	51
5.36	322	8,839	547
16.25	1,302	15,360	2,213
25.25	2,112	1,067	3,590
25.75	2,157	21,583	3,666
34.25	2,922	29,940	4,967
43.25	3,732	2,824	6,344
44.00	3,799	32,147	6,459
51.75	4,497		7,644
		112,615	

Negative Elevation (ft)	Seismic Soil (psf)	Seismic Incremt (psf)	Seismic Pressure (psf)	1.6 Soil + 1.4 Seismic Force (k)	ssure (psf)
0.5	20	776	796	4,342	1,119
5.36	214	776	991	15,828	1,430
16.25	1,140	776	1,916	20,689	2,911
25.25	1,905	776	2,681	1,351	4,135
25.75	1,948	776	2,724	26,223	4,203
34.25	2,670	776	3,446	34,459	5,359
43.25	3,435	776	4,211	3,182	6,583
44.00	3,499	776	4,275	35,684	6,685
51.75	4,158	776	4,934		7,739
				141,760	



S.1-2

Foundation Wall Design Summary

Tower Foundation Walls

Foundation elevation per drawings 11/03/04
Lateral soil pressure per geotech report dated 1/13/2005
RISA model dated 1/27/2005 - Pinned at Top, Fixed at Base

DEMAND

Design Shear (k)

Grd		Perm	Seismic
B1	15'-9"	12.3	12.5

Design Moment (k-ft)
M+: Steel on Interior Face

Grd		Perm	Seismic
B1	15'-9"	16.5	17.3

M-: Steel on Soil Face

Grd		Perm	Seismic
B1	15'-9"	36.3	37.3

DESIGN FORCES

Grd		Shear	M+ Interior	M- Soil
B1	15'-9"	12.5	17.3	37.3

WALL DESIGN

f'c = 5 ksi

Grd			M+ Interior	M- Soil
B1	15'-9"	T = 14"	#5 @9"	#8 @9"

CAPACITY

Grd		Shear	M+ Interior	M- Soil
B1	15'-9"	18.4	23.6	46.8

DEMAND-CAPACITY RATIOS

Grd		Shear	M+ Interior	M- Soil
B1	15'-9"	0.68	0.73	0.80

S.1-3

Foundation Wall Design

CONCRETE SHEAR CAPACITY, k per ft

Concrete to take all shear (no shear reinf.)
Assume d = T - 1.25" at inside face for shear

	Concrete Strength			
T (in)	3 ksi	4ksi	5 ksi	6 ksi
6	5.3	6.1	6.9	7.5
8	7.5	8.7	9.7	10.7
10	9.8	11.3	12.6	13.8
12	12.0	13.9	15.5	17.0
14	14.2	16.5	18.4	20.1
16	16.5	19.0	21.3	23.3
18	18.7	21.6	24.2	26.5
20	21.0	24.2	27.0	29.6
22	23.2	26.8	29.9	32.8
24	25.4	29.4	32.8	35.9

MINIMUM HORIZONTAL STEEL REQUIREMENT
[ACI 14.3.3]

T (in)	Total As,min
6	0.18
8	0.24
10	0.30
12	0.36
14	0.42
16	0.48
18	0.54
20	0.60
22	0.66
24	0.72

Area of Steel for Each Face

Spg (in)	#4	#5	#6	#7	#8
6	0.40	0.62	0.88	1.20	1.58
8	0.30	0.47	0.66	0.90	1.19
10	0.24	0.37	0.53	0.72	0.95
12	0.20	0.31	0.44	0.60	0.79
14	0.17	0.27	0.38	0.51	0.68
16	0.15	0.23	0.33	0.45	0.59
18	0.13	0.21	0.29	0.40	0.53
20	0.12	0.19	0.26	0.36	0.47
22	0.11	0.17	0.24	0.33	0.43
24	0.10	0.16	0.22	0.30	0.40

WALL FLEXURAL CAPACITY, k-ft per ft

For M+: Assume d = T - 0.75" - dia/2 (verts outside of horiz.)

Wall T = 14 in   f'c = 5 ksi

Spg (in)	#4	#5	#6	#7	#8	#9	#10	#11
6	22.98	35.08	48.94	65.38	84.04	126.17	153.55	179.90
7	19.75	30.19	42.20	56.50	72.85	110.17	134.88	159.13
8	17.31	26.50	37.09	49.75	64.27	97.73	120.17	142.48
9	15.41	23.61	33.08	44.43	57.50	87.79	108.30	128.88
10	13.89	21.29	29.85	40.14	52.01	79.67	98.54	117.60
11	12.64	19.39	27.20	36.60	47.48	72.92	90.37	108.11
12	11.59	17.79	24.98	33.64	43.67	67.22	83.45	100.02
13	10.71	16.44	23.09	31.12	40.43	62.35	77.50	93.04
14	9.95	15.28	21.47	28.95	37.64	58.13	72.34	86.96
15	9.29	14.28	20.07	27.07	35.20	54.44	67.83	81.62
16	8.72	13.39	18.83	25.41	33.07	51.19	63.84	76.90
17	8.21	12.61	17.74	23.94	31.17	48.31	60.29	72.69
18	7.75	11.[illegible]	16.77	22.64	29.48	45.73	57.11	68.91

Area of Steel for Each Face

Spg (in)	#4	#5	#6	#7	#8	#9	#10	#11
6	0.40	0.62	0.88	1.20	1.58	2.00	2.54	3.12
7	0.34	0.53	0.75	1.03	1.35	1.71	2.18	2.67
8	0.30	0.47	0.66	0.90	1.19	1.50	1.91	2.34
9	0.27	0.41	0.59	0.80	1.05	1.33	1.69	2.08
10	[illegible]	0.37	0.53	0.72	0.95	1.20	1.52	1.87
11	[illegible]	0.34	0.48	0.65	0.86	1.09	1.39	1.70
12	[illegible]	0.31	0.44	0.60	0.79	1.00	1.27	1.56
13	[illegible]	0.29	0.41	0.55	0.73	0.92	1.17	1.44
14	[illegible]	0.27	0.38	0.51	0.68	0.86	1.09	1.34
15	[illegible]	[illegible]	0.35	0.48	0.63	0.80	1.02	1.25
16	[illegible]	[illegible]	0.33	0.45	0.59	0.75	0.95	1.17
17	[illegible]	0.22	0.31	0.42	0.56	0.71	0.90	1.10
18	0.13	[illegible]	0.29	0.40	0.53	0.67	0.85	1.04
dia (in)	0.500	0.625	0.750	0.875	1.000	1.128	1.270	1.410
fy (ksi)	60	60	60	60	60	75	75	75
Total As,min [ACI 10.5.1]	0.53	0.53	0.53	0.52	0.52	0.41	0.41	0.41

For M-: Assume d = T - 3" - dia/2 (verts outside of horiz.)

Wall T = 14 in   f'c = 5 ksi

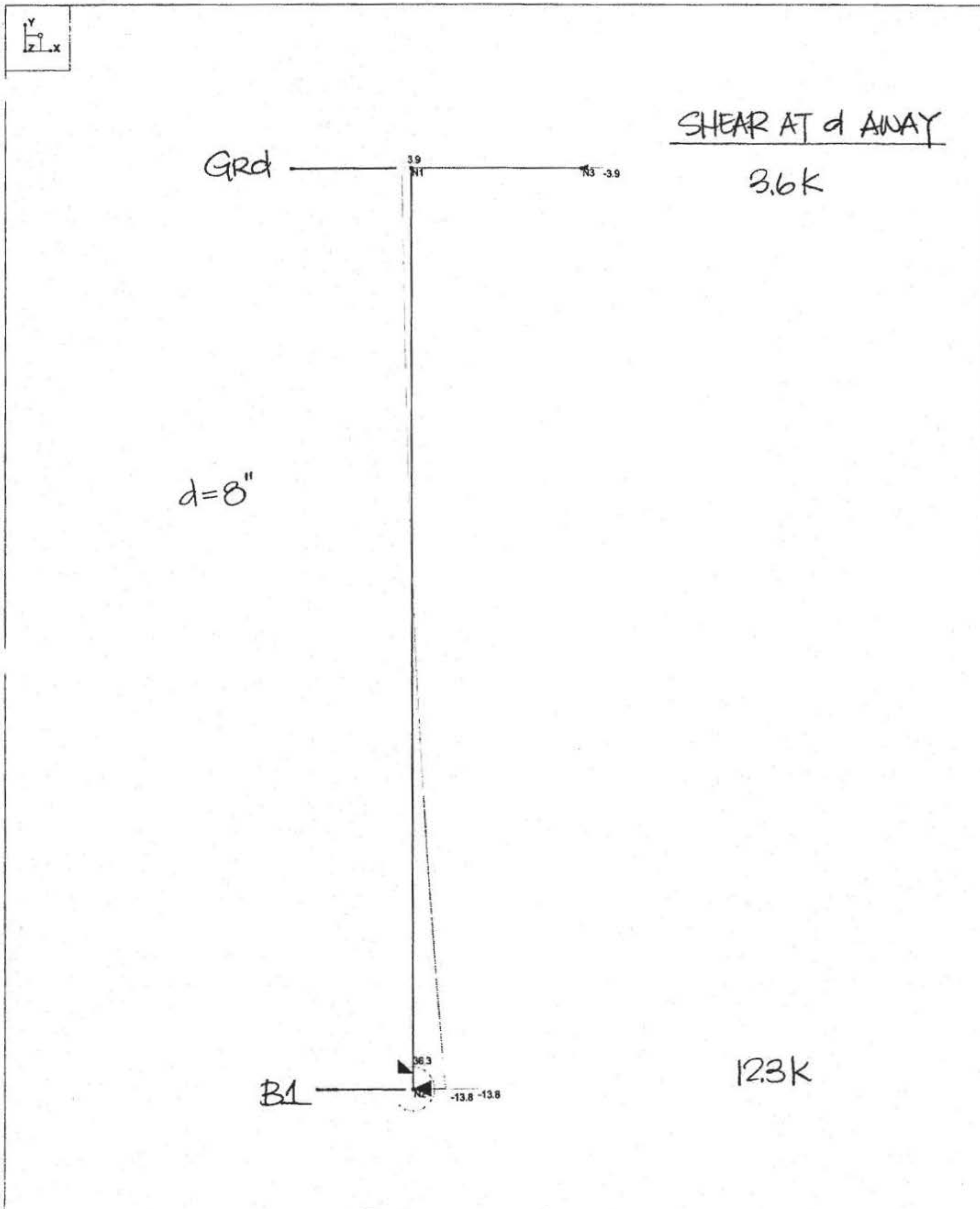
Spg (in)	#4	#5	#6	#7	#8	#9	#10	#11
6	18.93	28.80	40.03	53.23	68.05	100.86	121.41	140.42
7	16.27	24.81	34.56	46.09	59.14	88.48	107.33	125.29
8	14.27	21.79	30.40	40.63	52.27	78.75	96.06	112.86
9	12.71	19.43	27.14	36.33	46.83	70.92	86.87	102.56
10	11.46	17.52	24.51	32.85	42.41	64.49	79.25	93.91
11	10.48	15.96	22.34	29.98	38.75	59.12	72.84	86.57
12	9.57	14.65	20.53	27.57	35.68	54.57	67.37	80.27
13	8.84	13.55	18.98	25.51	33.05	50.66	62.67	74.81
14	8.22	12.59	17.66	23.74	30.78	47.28	58.57	70.04
15	7.67	11.76	16.50	22.21	28.80	44.31	54.97	65.83
16	7.20	11.04	15.49	20.85	27.07	41.70	51.78	62.09
17	6.78	10.40	14.59	19.66	25.53	39.38	48.94	58.75
18	6.40	9.83	13.80	18.59	24.15	37.30	46.40	55.75

$$V : \frac{V_u}{\phi V_c} = \frac{12.5}{18.4} = 0.68 \quad (T = 14")$$

$$M+ : \frac{M_u}{\phi M_n} = \frac{17.3}{23.61} = 0.73 \quad (\#5 \ @ \ 9" \ o.c.)$$

$$M- : \frac{M_u}{\phi M_n} = \frac{37.3}{46.83} = 0.80 \quad (\#8 \ @ \ 9" \ o.c.)$$

Tower Foundation Wall (Grd - B1)

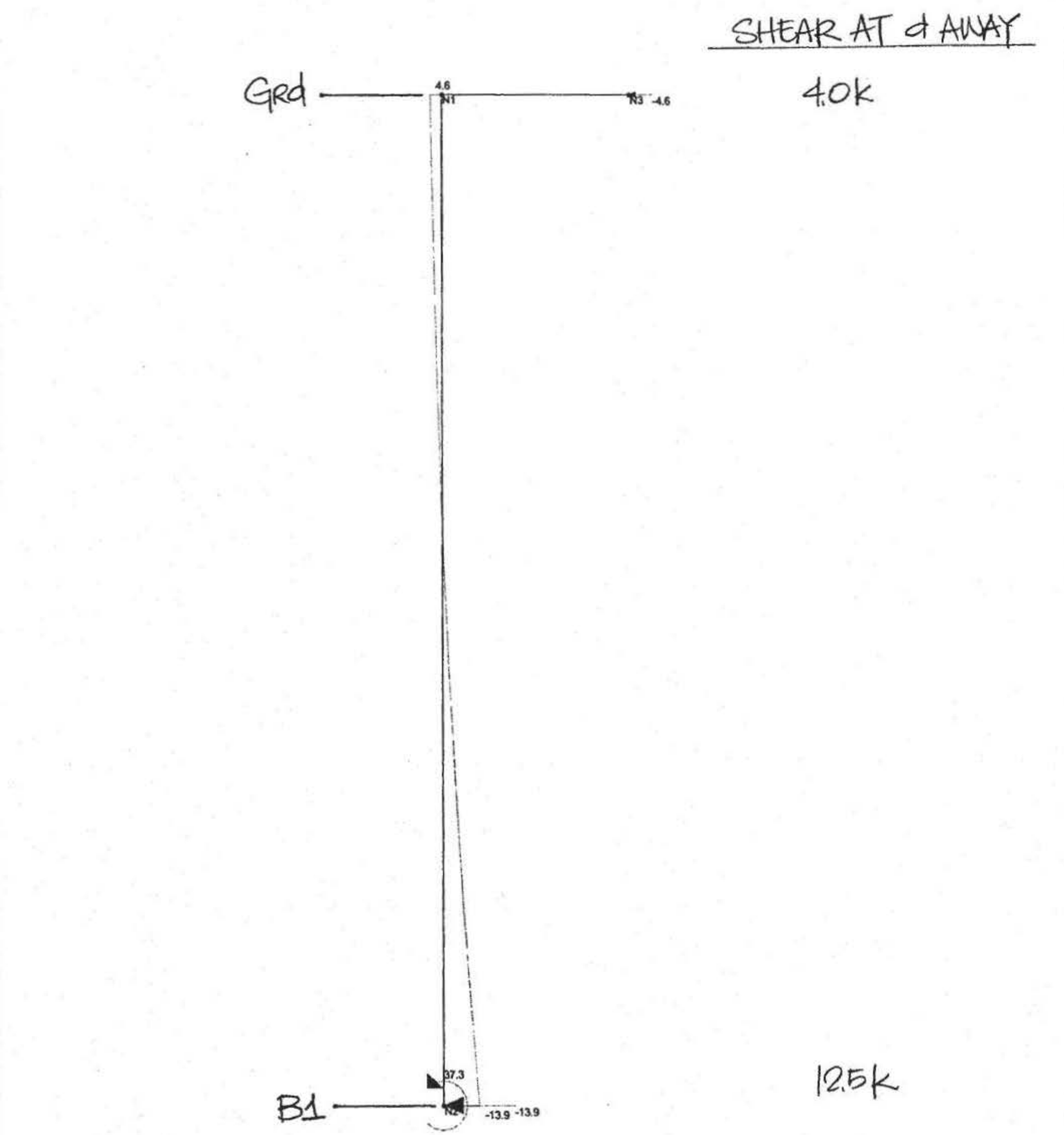


Results for LC 5, 1.7 Perm
Member y Shear Forces (k)
Reaction units are k and k-ft

1.7 Perm Soil + 1.7 Traffic Surcharge

DeSimone Consulting Eng...	301 Mission Street Tower Foundation Walls	
ML		Mar 8, 2005 at 4:25 PM
4069		4069-20050127-MKL-B1-Fdn-Wall....

5.1-5

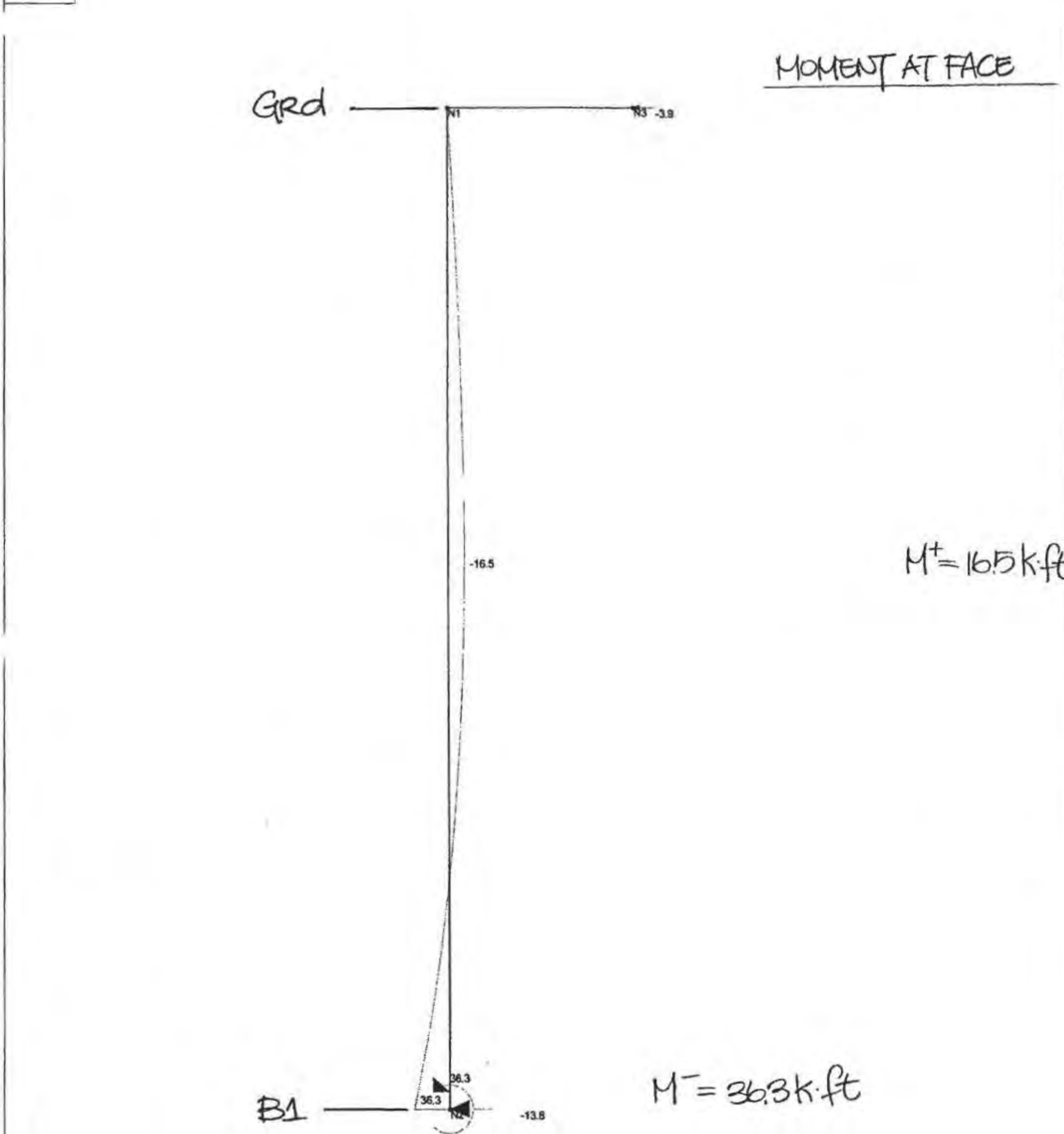


Results for LC 6, Seismic Combo
Member y Shear Forces (k)
Reaction units are k and k-ft

1.6Seismic Soil +1.4Seismic Increment +1.0Traffic Surcharge

DeSimone Consulting Eng...	301 Mission Street Tower Foundation Walls	
ML		Mar 8, 2005 at 4:26 PM
4069		4069-20050127-MKL-B1-Fdn-Wall....

5.1-6



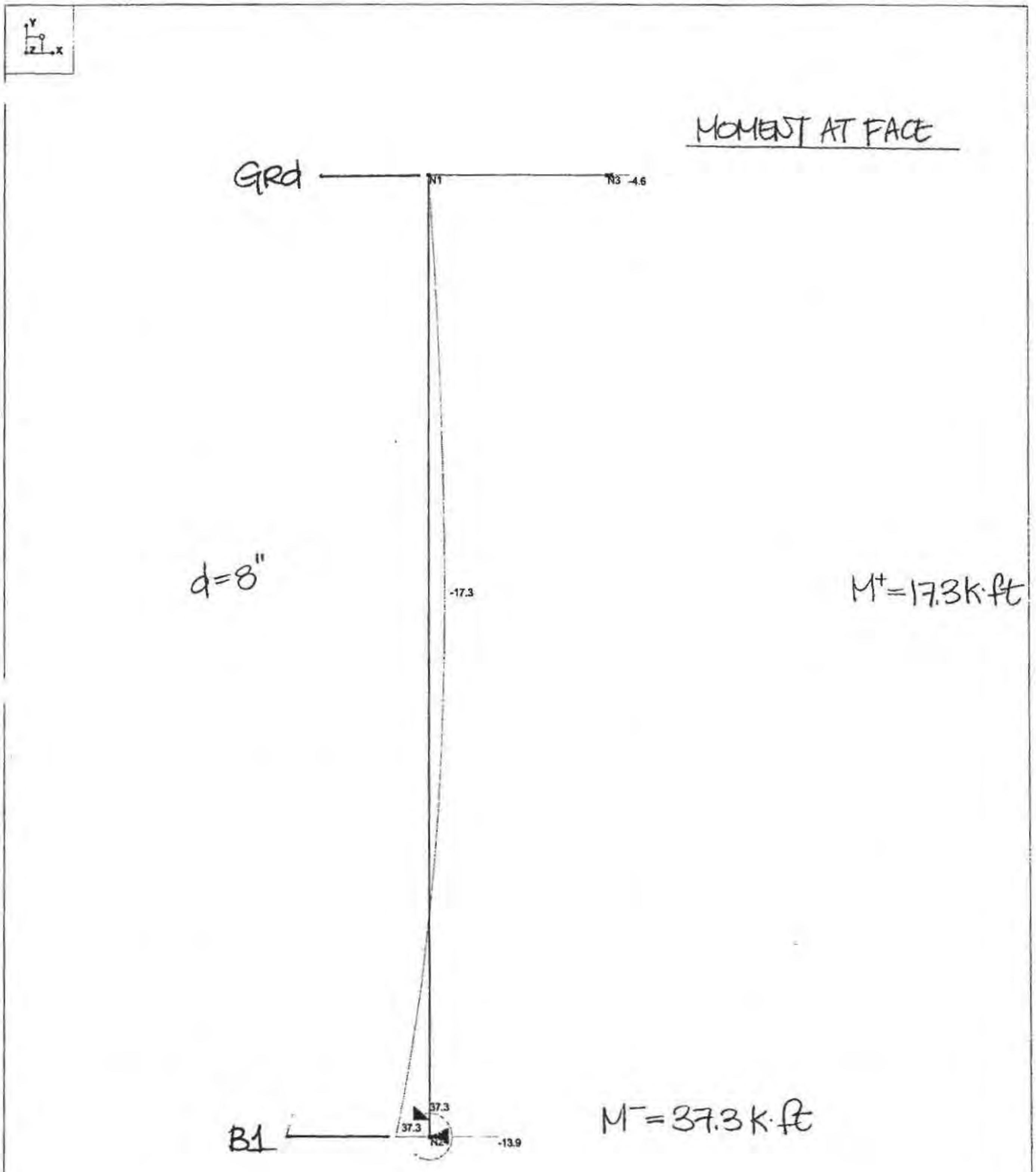
Results for LC 5, 1.7 Perm
Member z Bending Moments (k-ft)
Reaction units are k and k-ft

1.7 Perm Soil + 1.7 Traffic Surcharge

DeSimone Consulting Eng...	301 Mission Street Tower Foundation Walls	
ML		Mar 8, 2005 at 4:25 PM
4069		4069-20050127-MKL-B1-Fdn-Wall...

5.1-7

DODSONNOC00000294



Results for LC 6, Seismic Combo
Member z Bending Moments (k-ft)
Reaction units are k and k-ft

1.6 Seismic Soil + 1.4 Seismic Increment + 1.0 Traffic Surcharge

DeSimone Consulting Eng...	301 Mission Street Tower Foundation Walls	
ML		Mar 8, 2005 at 4:25 PM
4069		4069-20050127-MKL-B1-Fdn-Wall....

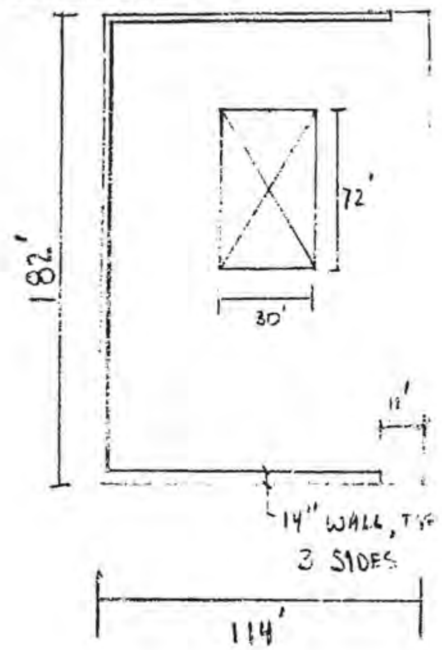
S.1-8

DODSONNOC00000295

DESIMONE

Project	301 MISSION	Page	Of
Project No.	4069B	Date	5/3/05
Item	GROUND FLR AND PERIMETER WALLS	By NJR	Ch'kd

GROUND FLR SLAB



GIVENS:

FLR TO FLR:
TOP OF MAT - G = 15.75'

$C_v = 0.681$

$C_a = 0.440$

SDL = 75 psf

FLR TO FLR G-22 = 16.58'

ASSUME:

SLAB = 14" TH

R = 4.5 (BEARING WALL)

SEISMIC LOAD

$T = C_t (h_n)^{3/4} = 0.020 (15.75)^{3/4}$

$= 0.158$ sec

$V = \frac{C_v I}{R T} W = \frac{(0.681)(1.0)}{(4.5)(0.158)} W = 0.957 W$

$V_{MAX} = \frac{2.5 C_a I}{R} W = \frac{2.5(0.440)(1.0)}{4.5} W = 0.244 W$ ← GOVERNS

$\therefore V = 0.244 W$

MASS:

— FLR AREA = $(114' \cdot 182') - (72' \cdot 30')$

= 18,588 ft^2

— AREA OF COLS + WALLS:

8(COL D) = 144 ft^2

4(COL A) = 60 ft^2

4(COL B) = 157 ft^2

4(COL C) = 72 ft^2

PERI WALLS = 452 ft^2

TOTAL = 885 ft^2

— VOLUME OF BMS @ GROUND (EXCLUDES SLAB)

MOMENT FRAME BMS = 12,692 ft^3

GRADE BMS = 4,863 ft^3

TOTAL = 17,555 ft^3

$WT_{SLAB} = (18588 ft^2 - 885 ft^2)(\frac{14"}{12"})(150 pcf) = 3098$ kips

$WT_{SDL} = (18588 - 885)(75 psf) = 1327$ kips

$WT_{VERTS} = (885 ft^2)(\frac{16.58'}{2} + \frac{15.75'}{2})(150 pcf) = 2146$ kips

$WT_{ALL BMS} = (17,555 ft^3)(150 pcf) = 2633$ kips

W = 9204 kips

GROUND FLOOR DESIGN SHEAR:

V = 0.244 W = 0.244 (9204 k)

V = 2446 kips

S.1-9

DESIMONE

Project: 301 MISSION

Project No.: 4069

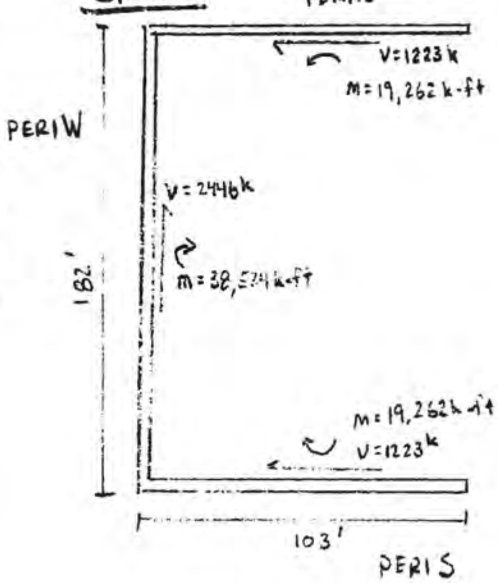
Item: PERIMETER WALL LOADS

Page ____ Of ____

Date: 5/3/05

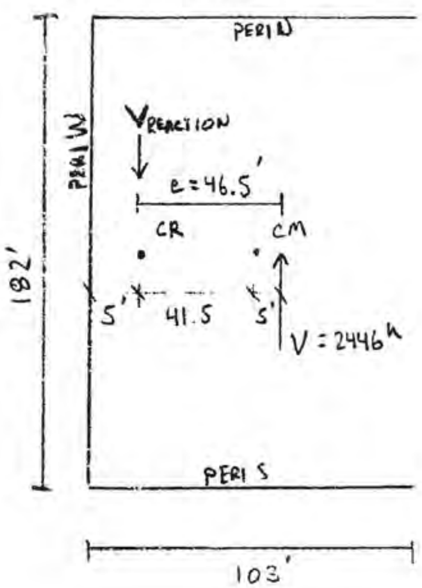
By: NJR Ch'kd ____

SHEAR



- VERTICAL BARS CONTROLLED BY RETAINING WALL DESIGN.
- SHEAR BARS - SEE ATTACHED CALC
- NO BE ELEMENT NEEDED

TORSION:



$T = Ve = 2446^k \cdot 46.5'$

$= 114{,}000 \text{ k-ft}$

- ASSUME ALL FORCE GOES TO THE NORTH AND SOUTH WALLS;

$V_{PERIN} = \frac{114{,}000 \text{ k-ft}}{182 \text{ ft}} = 630^k = V_{PERIS}$

$V_{TOTAL} = V_{SHEAR} + V_{TORSION}$ (100% BOTH DIRS)

$= 1223^k + 630^k$

$V_{u\,PERIN\ AND\ PERIS} = 1853^k$

- PERIW (NEGLIGIBLE SINCE CR IS CLOSE TO WALL)
(ALSO - MIN $\rho = 0.0025$ CONTROLS UNTIL V_U EXCEEDS 5000k)

$V_{u\,PERIW} = 2446^k$

S.1-10

dlf	dlf	dlf
1.0	1.0	1.0

Unit Wt.	0.150	kcf
Min trib area from group	Varies	ft^2 (For Tension)
Max trib area from group	Varies	ft^2 (For Compression)

PeriN and PeriS

Floor	Usage	Flr. Ht. ft.	Elevation ft.	Width in	Length in	Min Trib A. sq. ft	Min Cum Trib A. sq. ft	Max Trib A. sq. ft	Max Cum Trib A. sq. ft	Floor DL psf	Red. LL psf	Self Wt kips	Min Total DL kips	Min Cum DL kips	Max Total DL kips	Max Cum DL kips	Cum Reducible LL kips	LL % multiplier	Cum Red. LL kips	Min Cum Service kips	Max Cum Service kips	1.42(dlf)D+0.5L Design kips	0.9*D Design kips	1.4(dlf)D+1.7L Design kips
2	Cap	16.58	15.6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1.00	0	0	0	0	0	0
1	Roof	15.75	-1.0	14	1236	0	0	1000	1000	250	100	284	284	284	534	534	100	1.00	100	384	634	808	256	917
Base			-16.8																					
												284												

PeriW

Floor	Usage	Flr. Ht. ft.	Elevation ft.	Width in	Length in	Min Trib A. sq. ft	Min Cum Trib A. sq. ft	Max Trib A. sq. ft	Max Cum Trib A. sq. ft	Floor DL psf	Red. LL psf	Self Wt kips	Min Total DL kips	Min Cum DL kips	Max Total DL kips	Max Cum DL kips	Cum Reducible LL kips	LL % multiplier	Cum Red. LL kips	Min Cum Service kips	Max Cum Service kips	1.42(dlf)D+0.5L Design kips	0.9*D Design kips	1.4(dlf)D+1.7L Design kips
2	Cap	16.58	15.6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1.00	0	0	0	0	0	0
1	Roof	15.75	-1.0	14	2184	288	288	288	288	250	100	502	574	574	574	574	29	1.00	29	602	602	829	516	852
Base			-16.8																					
												502												

S.1-11

DODSONNOC00000298

SHEAR WALL SHEAR CHECK

Etabs model: None--Hand Calc
Date: 5/3/2005
By: NJR

								Shear Reinforcement of Wall							Check design					
Wall ID	Story	Width in	Length in	f'_c psi	f_{yv} ksi	ϕ	V_u kips	A_{cp} in²	$V_{n\,max}$ = 10Acp*sqrt(f'c) kips	Check size of section $V_{n\,max}<(V_u/\phi)$	ϕV_c kips	$\rho_{req'd}$	Area of steel within spacing in²	Spacing required in	Spacing provided in	$\rho_{provided}$	V_c+V_s kips	V_n = min of V_c+V_s or 10Acp*sqrt(f'c) kips	$V_u/\phi V_n$	Overstrength Provided $(V_c+V_s)/V_u$
PeriN&PeriS	B1	14	1236	5000	60	0.60	1853	17304	12236	OK	1468	0.0025	0.40	11.4	12.0	0.0024	4919	4919	0.63	2.65

								Shear Reinforcement of Wall							Check design					
Wall ID	Story	Width in	Length in	f'_c psi	f_{yv} ksi	ϕ	V_u kips	A_{cp} in²	$V_{n\,max}$ = 10Acp*sqrt(f'c) kips	Check size of section $V_{n\,max}<(V_u/\phi)$	ϕV_c kips	$\rho_{req'd}$	Area of steel within spacing in²	Spacing required in	Spacing provided in	$\rho_{provided}$	V_c+V_s kips	V_n = min of V_c+V_s or 10Acp*sqrt(f'c) kips	$V_u/\phi V_n$	Overstrength Provided $(V_c+V_s)/V_u$
PeriW	B1	14	2184	5000	60	0.60	2446	30576	21620	OK	2594	0.0025	0.40	11.4	12.0	0.0024	8692	8692	0.47	3.55

5.1-12

5.2 South Perimeter Wall

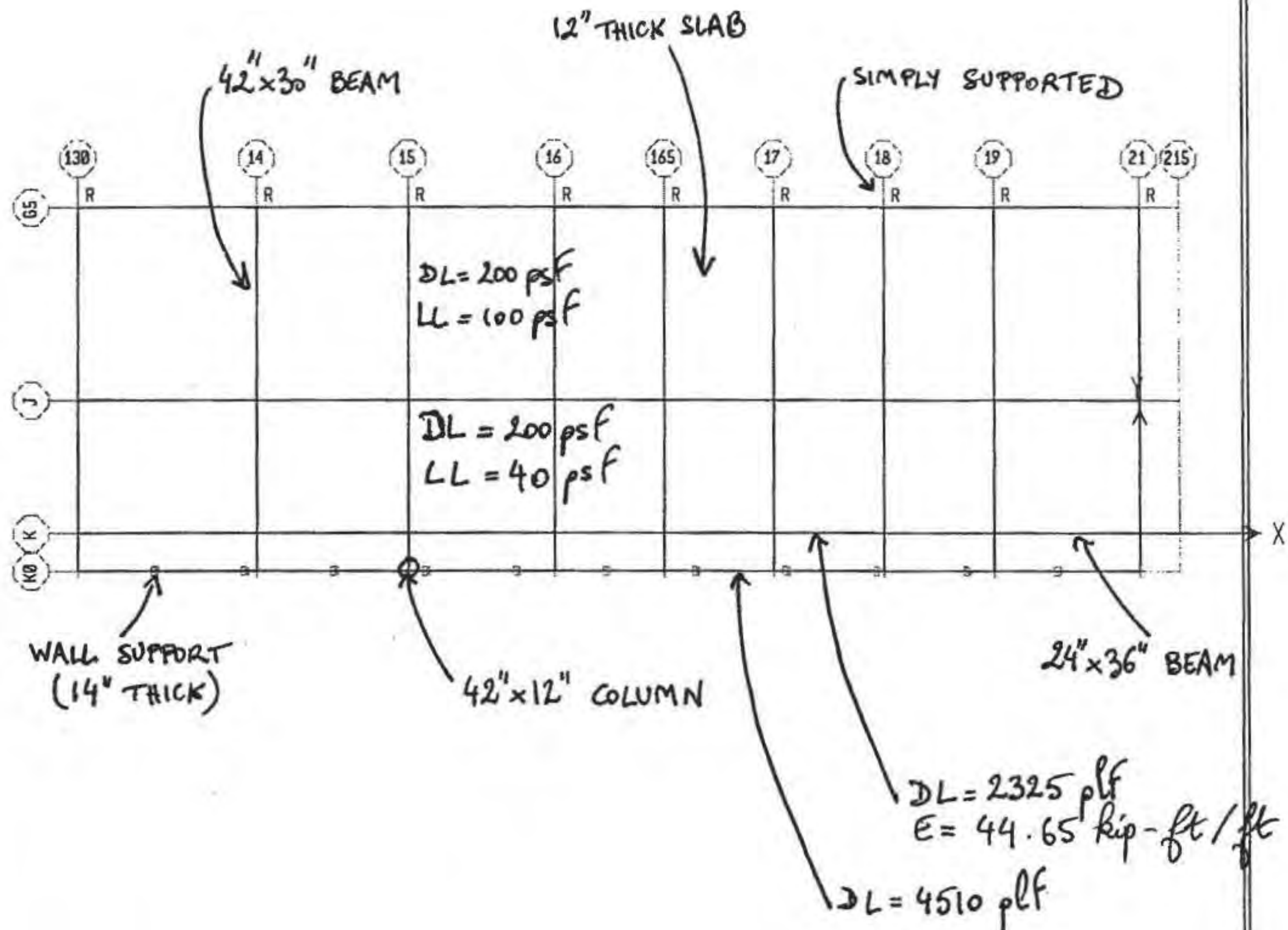
5.2 South Perimeter Wall

The out-of-plane loads are the same as for the north and west tower perimeter walls resulting in the same vertical steel I the wall.

At level 1, the south wall moves five feet further south. This setback in the wall requires a special torsion beam. This torsion beam is supported by wall below and restrained against torsion by beams B01-03.

TORSION BEAM DESIGN

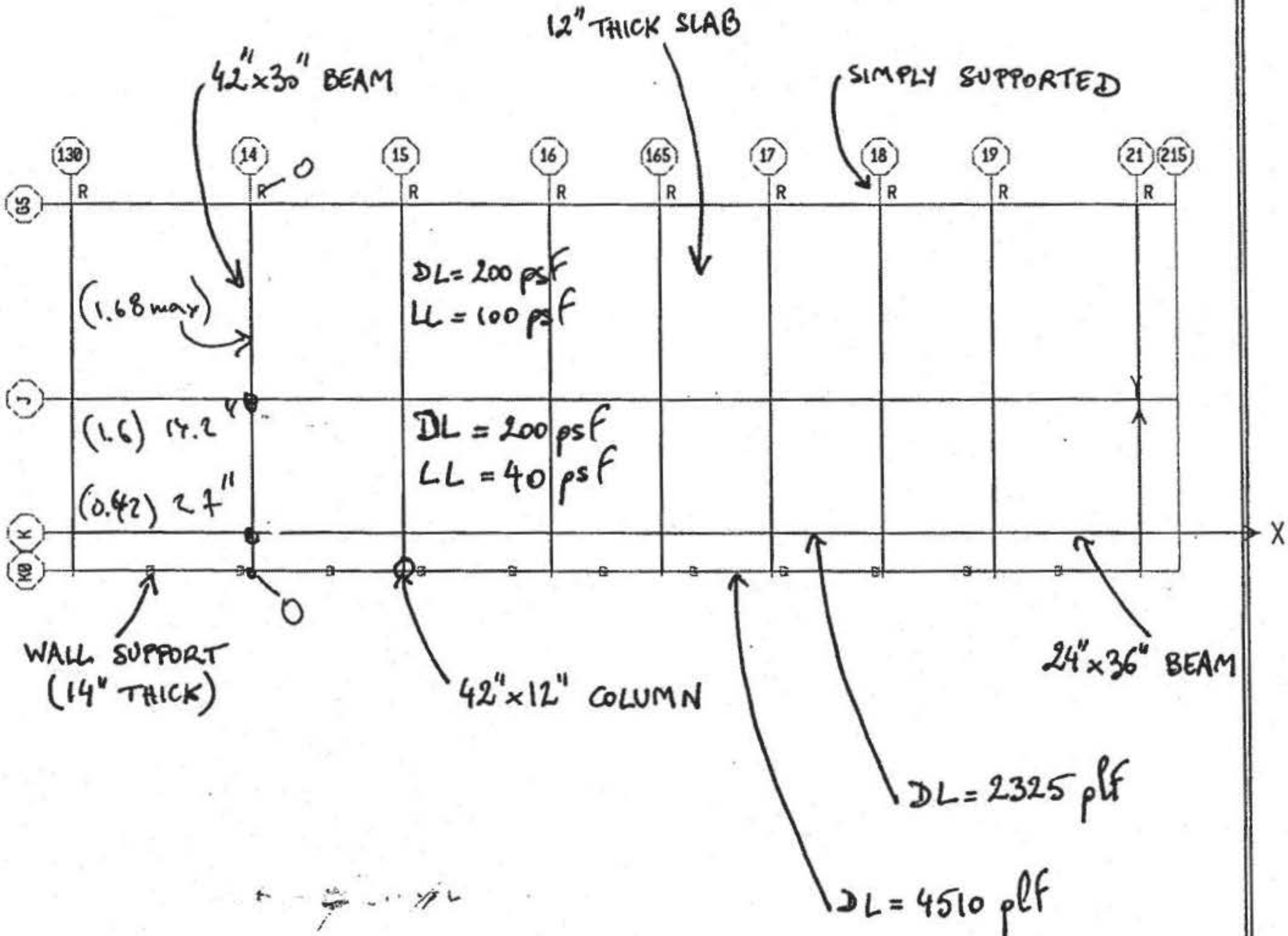
(L1) TOWER, Line K



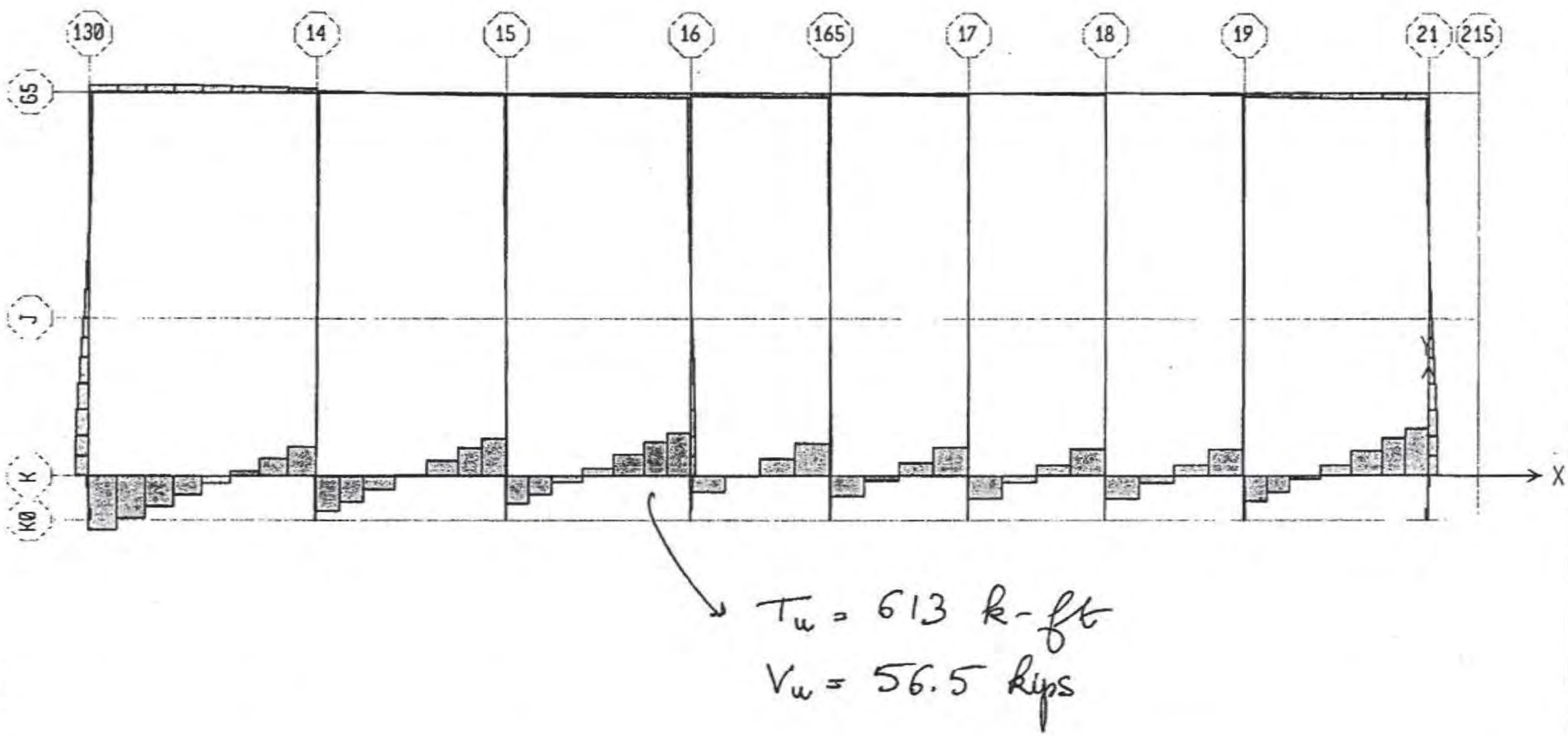
S.2-2

Defl DL+LL

(uncracked) cracked



S.2-3



S,2-4

SAFE v8.0.1 - File: 4069-20050225-NPW-level2-torsionbeam - February 28,2005 11:53 - Scale: Fit to Page
Torsion Diagram - (12DM10F) - Kip-In Units

DESIGN OF RC BEAM FOR TORSION

f'c = 5000 psi
fy = 60 ksi
ϕ = 0.75
cover = 0.75 to ties

Tu =	613.024
Vu =	56.48

←

b (ft)	h (ft)	b (in)	h (in)	d (in)	A (in^2)	Ix (in^4)	Iy (in^4)	b-cover (in)	h-cover (in)	ph (in)	Aoh (in^2)	ϕTn conc (k-ft)
5	2.5	60	30	29.25	1,800	135,000	540,000	59	29	174	1,667	60

Is member big enough? ACI Eq. 11-18 Vu / bd (ksi)	Tu*Ph/ (ksi)	left term	right term	left < right ?	ACI 11-21 At / s Req'd	ACI 11-15 Av / s Req'd	11.6.3.8 A / s Req'd	ACI 11-23 A / s code min	s Req'd #4	s Req'd #5	ACI 11-22 Aℓ Req'd (in^2)	ACI 11-24 Aℓ min (in^2)
0.032	0.271	0.27	0.53	OK	0.0577	-0.0985	0.0577	0.0250	3.5	5.4	10.04	0.57

S-2.5

SECTION 6 – MID-RISE MAT FOUNDATION SYSTEM

6.1 Design Methodology and Assumptions

6.1 Design Methodology and Assumptions

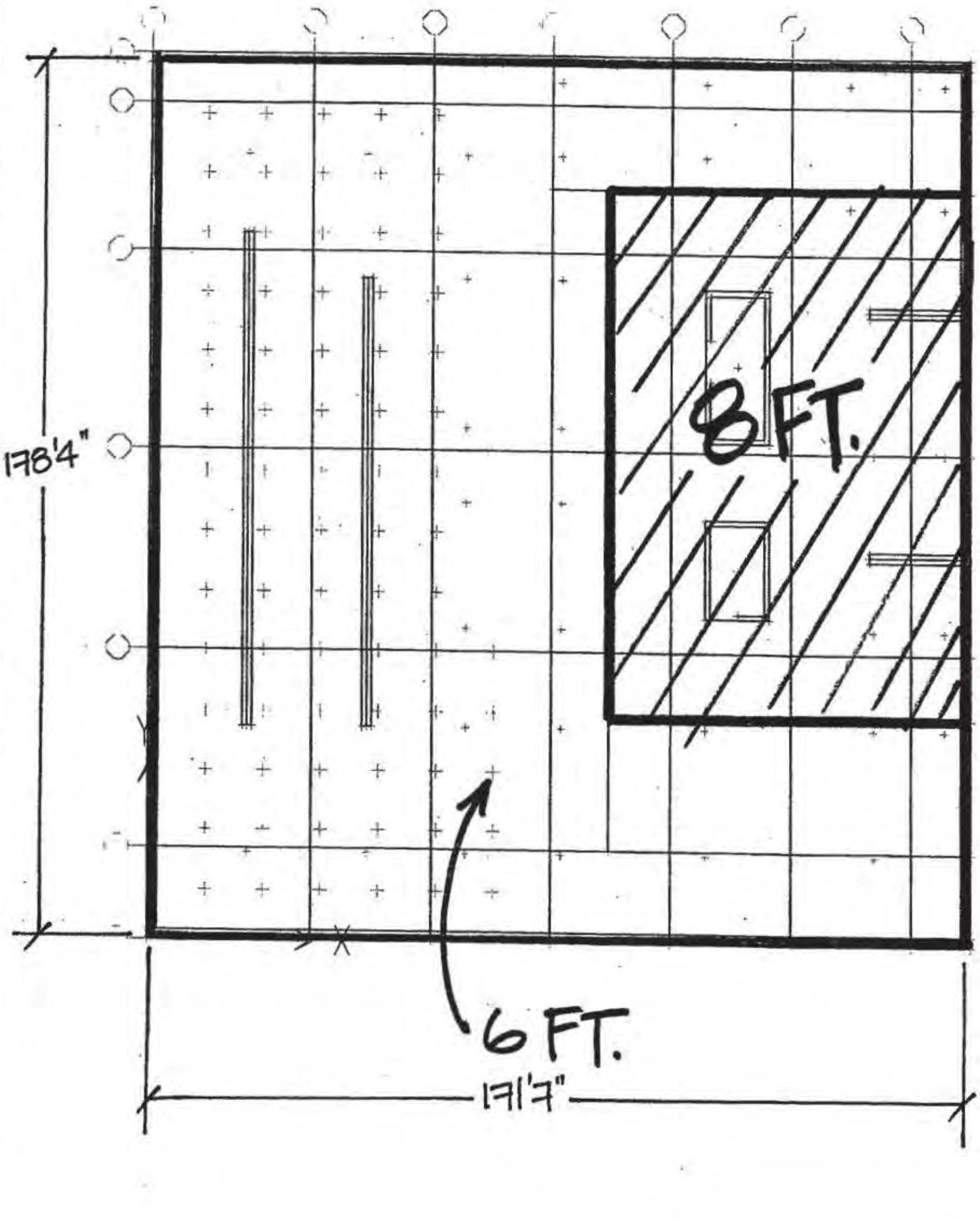
The foundation system consists of a 178'-4"(N-S) x 171'-7" (E-W) mat underneath the podium structure. The mat is 8'-0" thick directly underneath the core and 6'-0" thick in all other areas. Loads onto the foundation mat include column and wall gravity loads, wall seismic loads, and uplift due to groundwater pressure below.

Analysis and design are done with the aide of a three-dimensional computational program, SAFE. Soil subgrade moduli values are obtained from the project geotechnical engineer, Treadwell & Rollo, dated January 4, 2005. These values are established through close collaboration between the two offices.

Analysis of the foundation mat is performed using SAFE, where the soil pressures are computed and checked. Because the weight of the podium structure is relatively light and the groundwater produces uplift forces on the mat, tie-downs are used to hold down the west side of the mat. These tie-downs take tension when the surrounding mat is pushed upward and do not take any load when the surrounding mat is in compression.

Since the tie-downs are modeled as point supports and can actually take compression in the SAFE model, four models are created (all tie-downs, no tie-downs, tie-downs on the northern half, and tie-downs on the southern half) and the load cases are analyzed in the appropriate model so as to ensure proper modeling of this tension-only element.

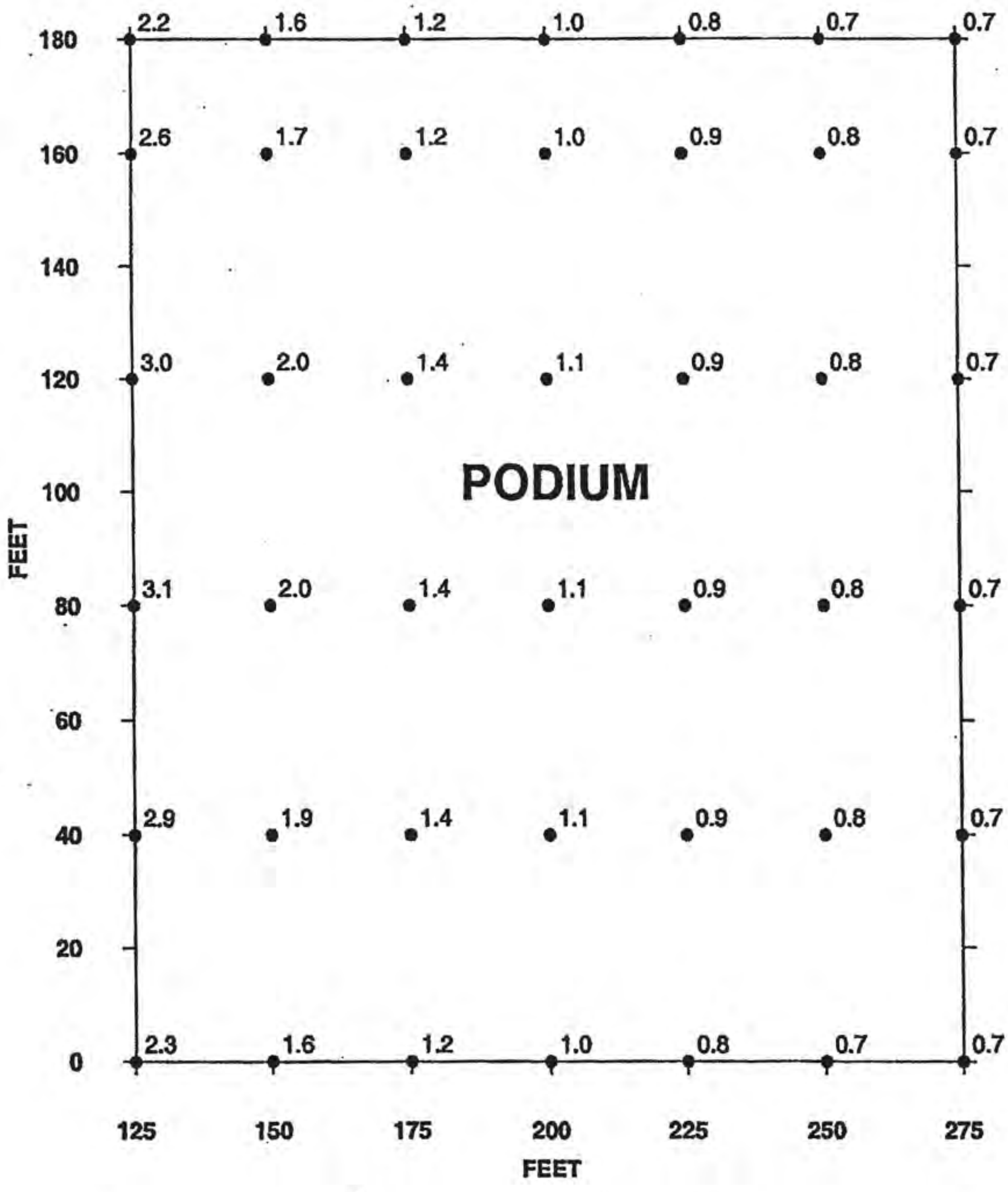
MAT THICKNESS



6.1-2

DRAFT

Estimated settlement in inches



Note: For a 60 foot excavation - Estimated settlement based on foundation pressures provided by DeSimone Consulting Engineers (DCE), dated 17 June 2004 (Podium); Assumes adjacent tower is pile supported and that the soil from a depth of 60 to 90 feet is not compressible and not improved below the Podium footprint.

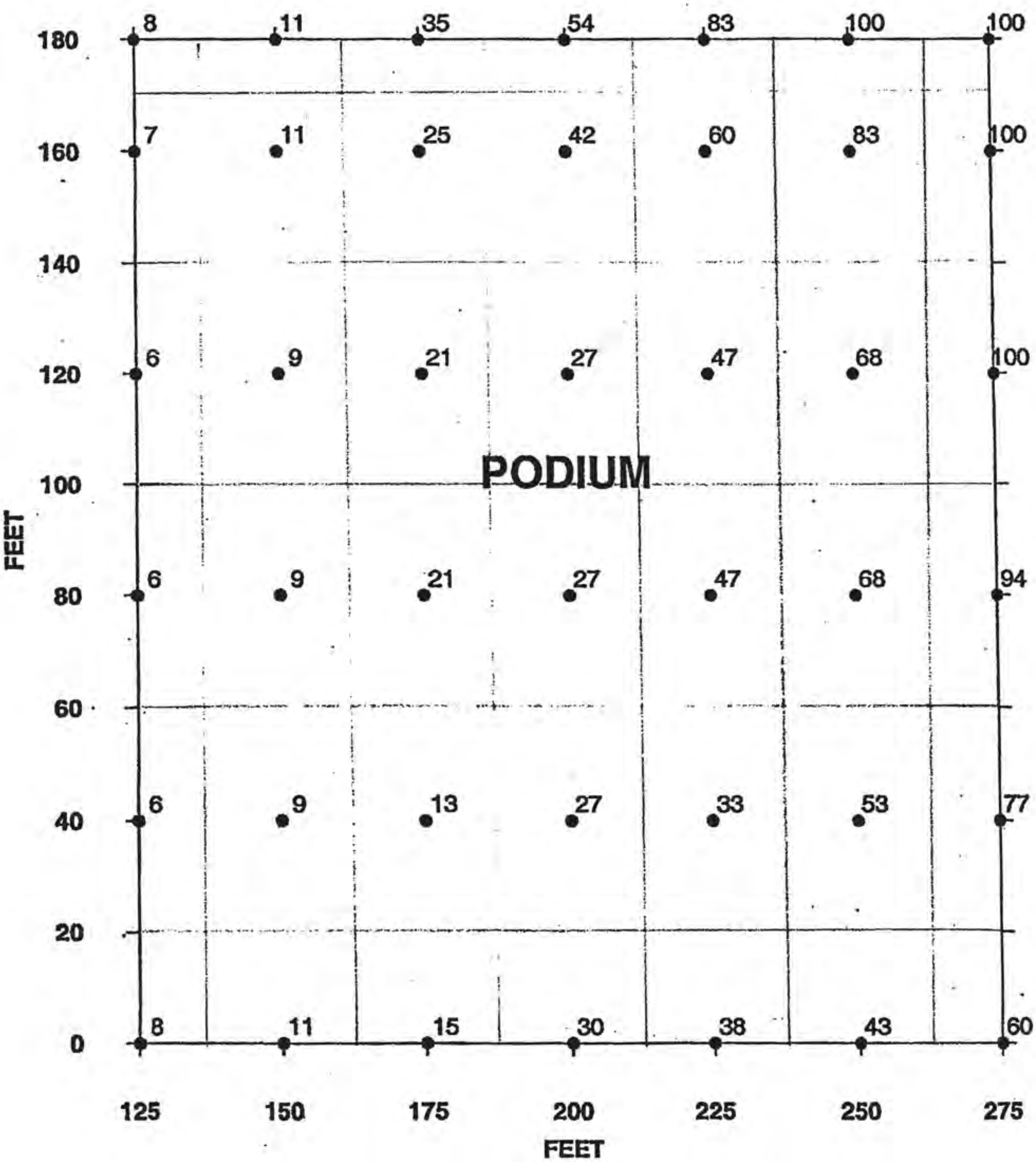
301 MISSION STREET
San Francisco, California
Project No. 3157.02
16 NOVEMBER 2004

ESTIMATED SETTLEMENT
TREADWELL & ROLLO, INC.

6.1-3

SOIL SUBGRADE MODULUS VALUES PER TREADWELL & ROLLO 1/4/05

Modulus of subgrade reaction in kips per cubic feet (kcf)



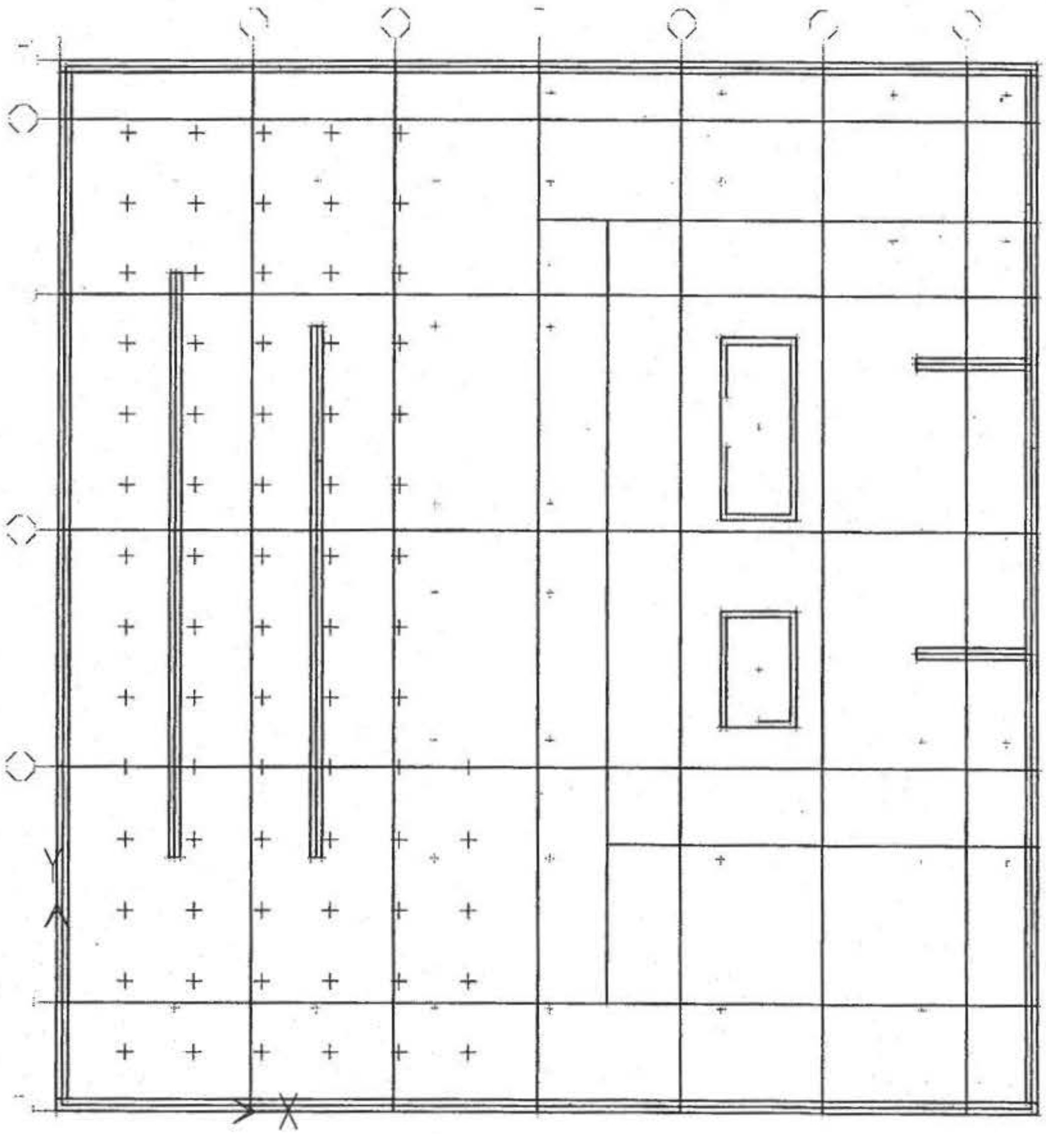
Note: For a 60 foot excavation - Estimated settlement based on foundation pressures provided by DeSimone Consulting Engineers (DCE), dated 17 June 2004 (Podium); Assumes adjacent tower is pile supported and that the soil from a depth of 60 to 90 feet is not compressible and not improved below the Podium footprint.

301 MISSION STREET
San Francisco, California
Project No. 3157.02
16 NOVEEMBER 2004

MODULI OF SUBGRADE REACTION
TREADWELL & ROLLO, INC.

6.1-4

- THIS MODEL IS USED FOR LOAD CASES IN WHICH ALL (OR MOST) OF THE TIE-DOWNS ARE IN TENSION

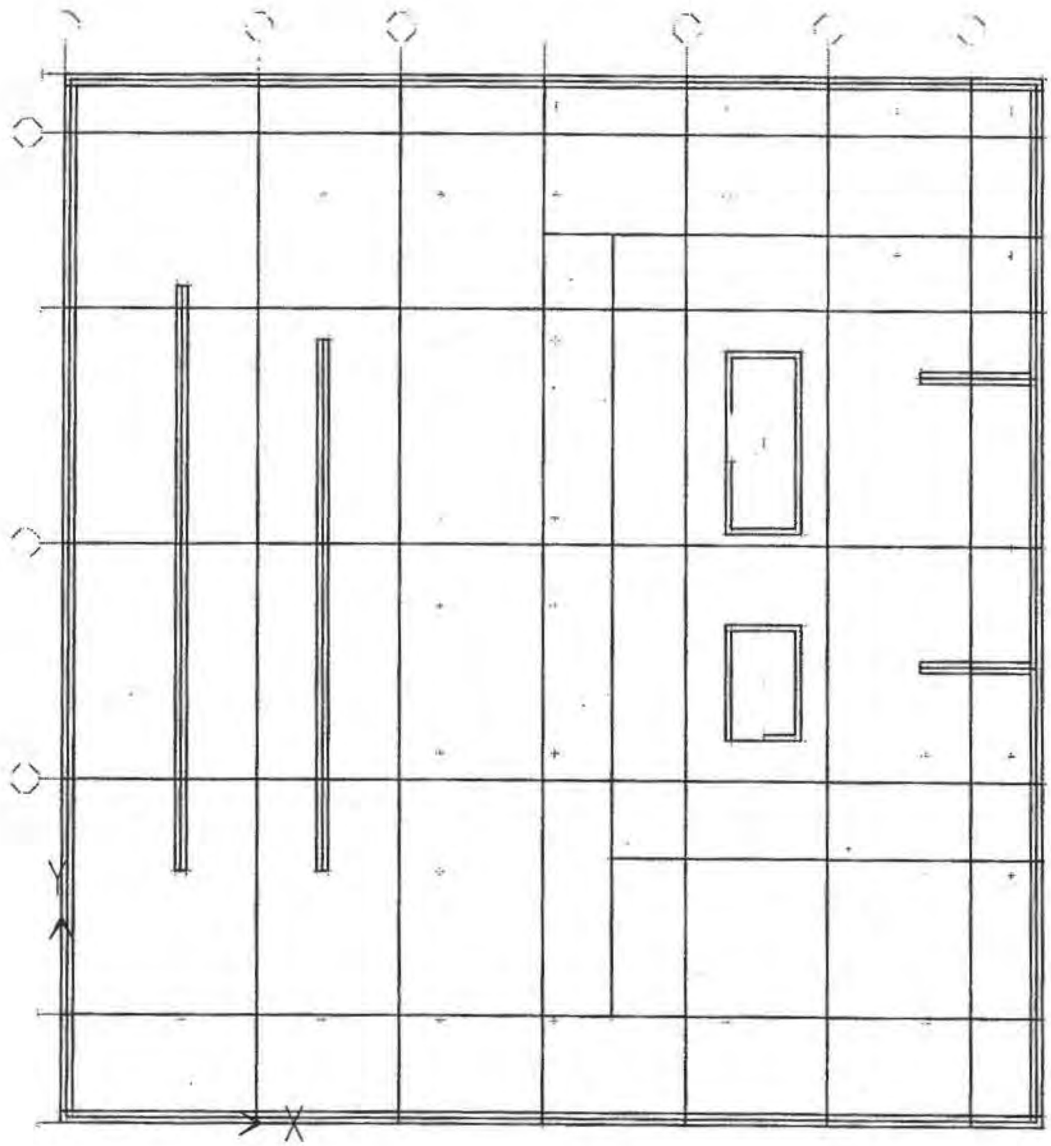


6.1-5

SAFE v8.0.6 - File: 4069-20050509-Podium-Mat-all tie-downs-align - May 23,2005 20:05 - Scale: 1 in = 32 ft
Structural Layer Plan View - Kip-ft Units

SAFE MODEL W/ NO TIE-DOWNS

- THIS MODEL IS USED FOR LOAD CASES IN WHICH NONE OF THE TIE-DOWNS ARE IN TENSION

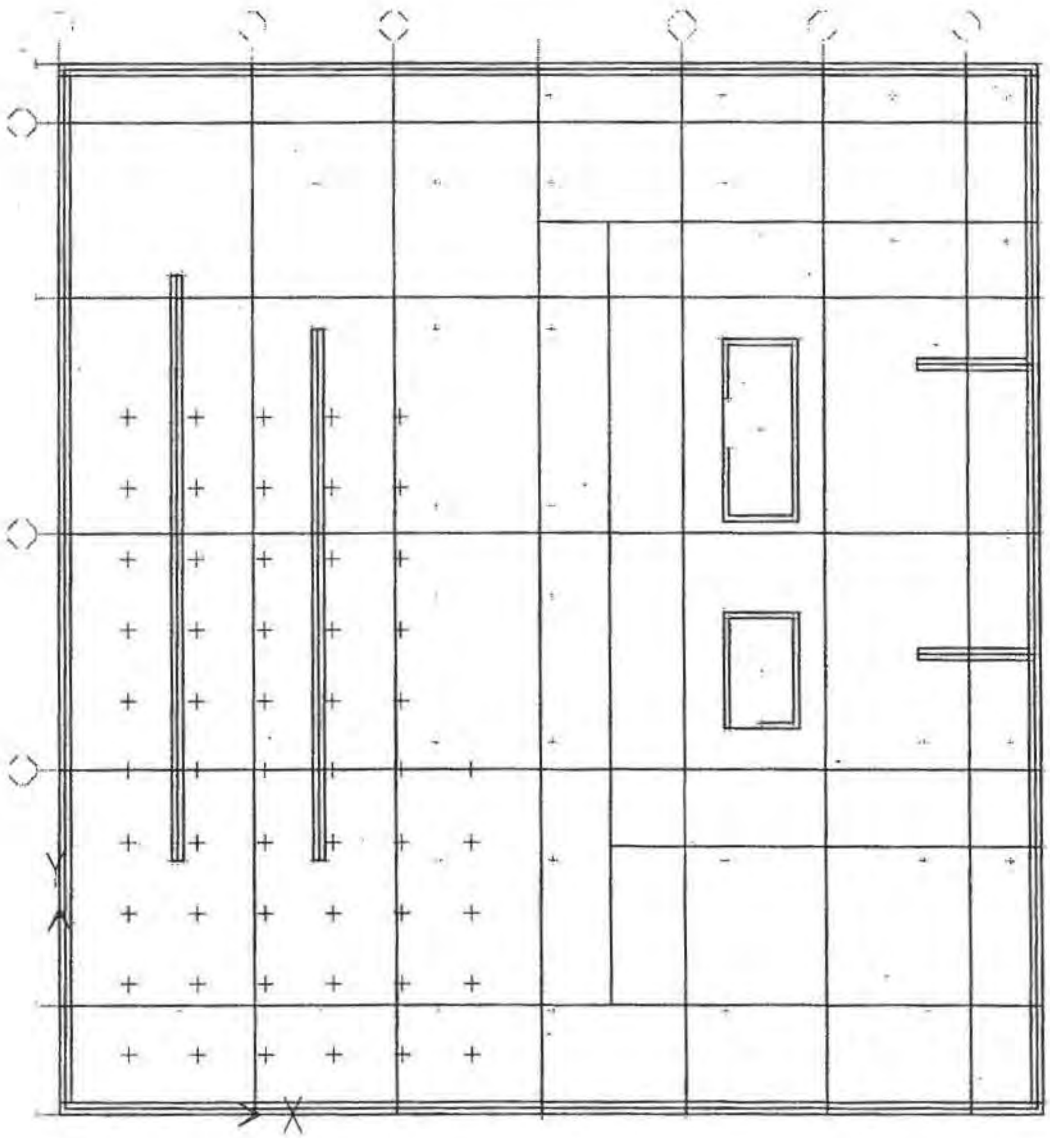


6.1-6

SAFE v8.0.6 - File: 4069-20050509-Podium-Mat-no tie-downs-align - May 23,2005 20:05 - Scale: 1 in = 32 ft
Structural Layer Plan View - Kip-in Units

SAFE MODEL W/ TIE-DOWNS ON SOUTHERN HALF

- THIS MODEL IS USED FOR LOAD CASES IN WHICH TIE-DOWNS ON THE SOUTHERN HALF ARE IN TENSION

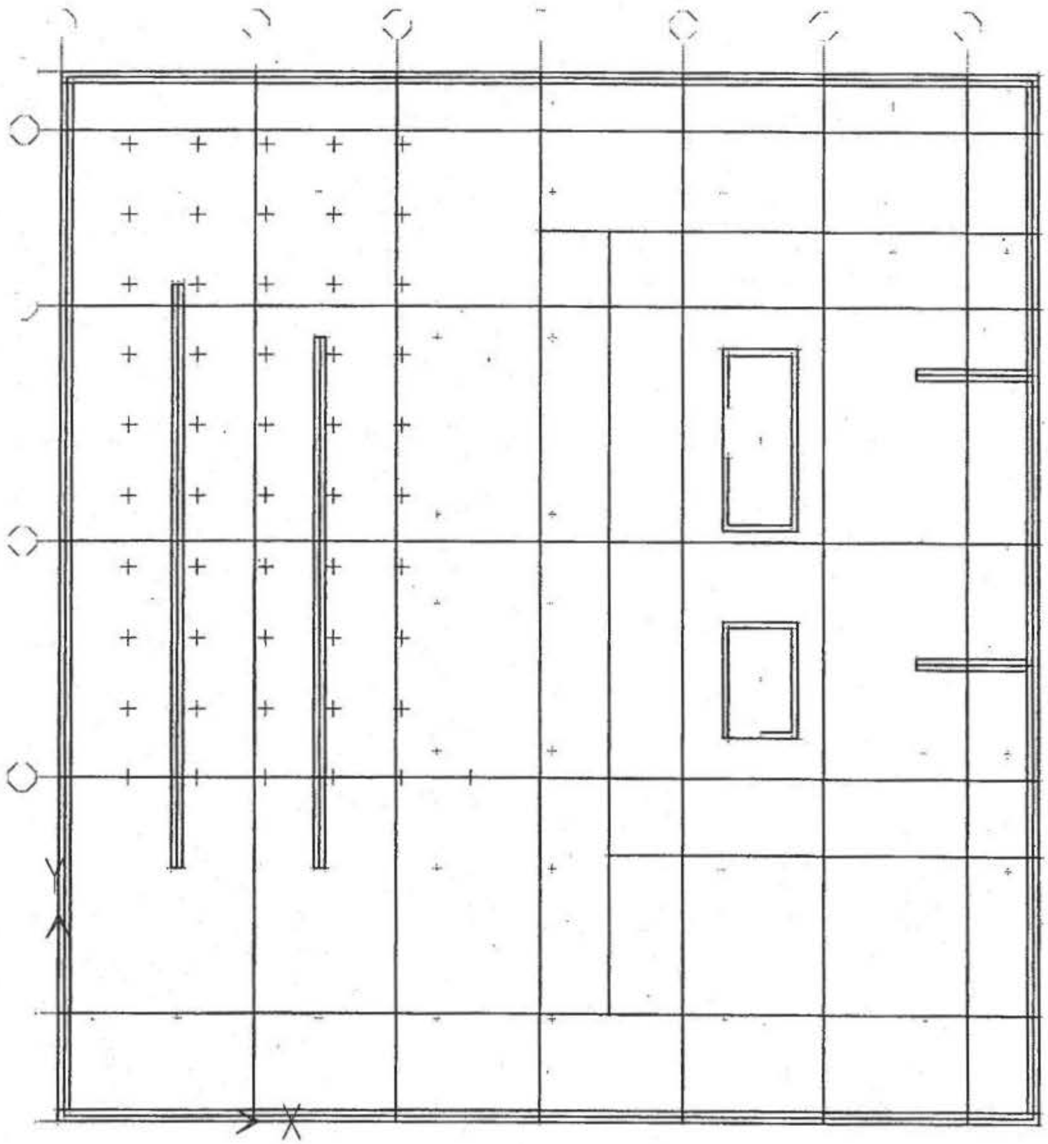


6.1-7

SAFE v8.0.6 - File: 4069-20050509-Podium-Mat-tie-downs bottom-align - May 23,2005 20:05 - Scale: 1 in = 32 ft
Structural Layer Plan View - Kip-in Units

SAFE MODEL W/ TIE-DOWNS ON NORTHERN HALF

- THIS MODEL IS USED FOR LOAD CASES IN WHICH TIE-DOWNS ON THE NORTHERN HALF ARE IN TENSION



6.1-8

SAFE v8.0.6 - File: 4069-20050509-Podium-Mat-tie-downs top-align - May 23,2005 20:08 - Scale: 1 in = 32 ft
Structural Layer Plan View - Kip-in Units

6.2 Design Forces And Load Combinations

6.2 Design Forces and Load Combinations

Loads onto the foundation mat include gravity loads from the columns and walls and seismic loads from the shear walls. Uplift forces on the mat due to groundwater pressure are also included.

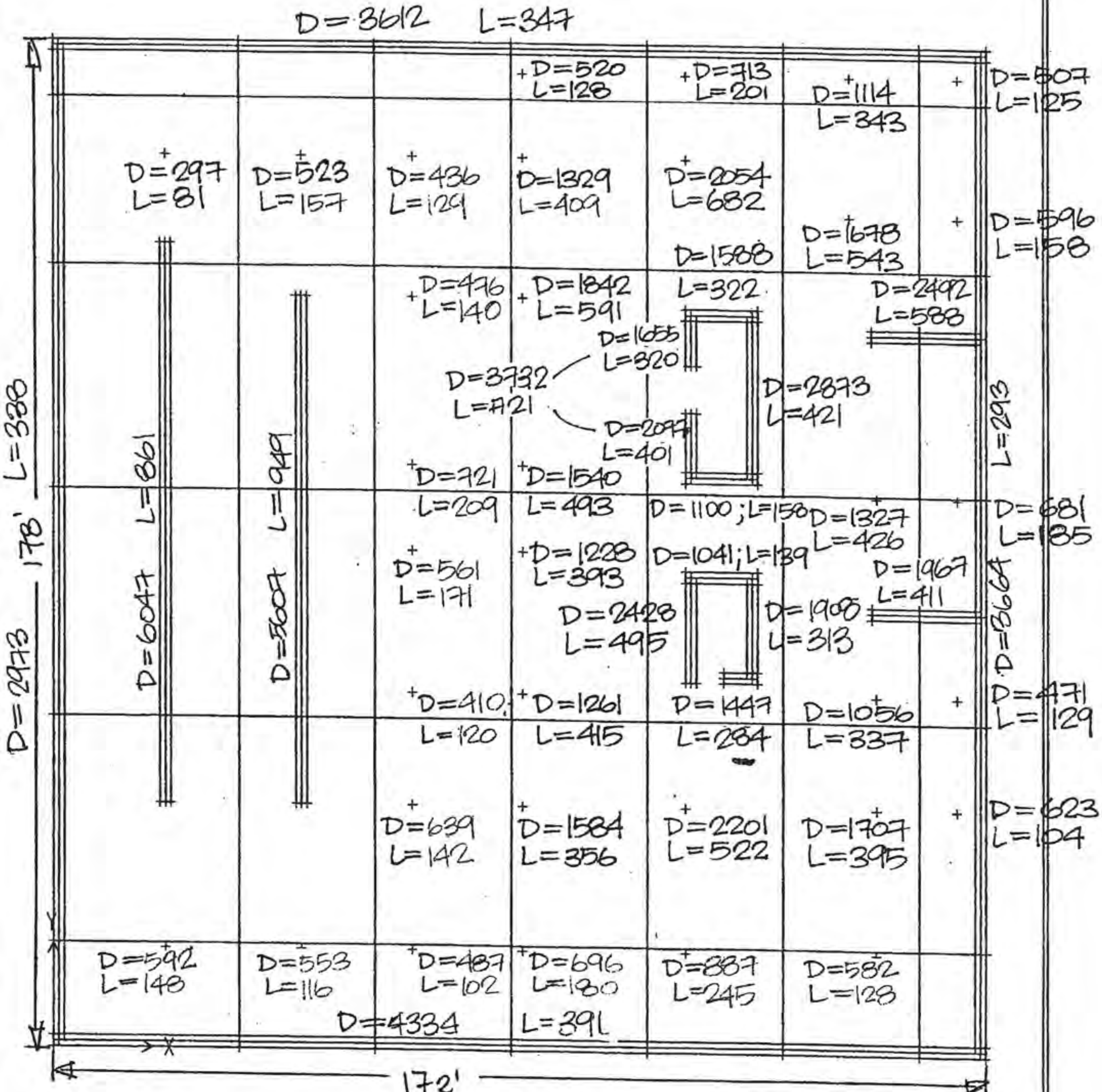
ASD load combinations per UBC-97 are used for the analysis of the foundation mat. Load combinations include seismic loads in both directions, including orthogonal and torsional effects. Combinations also include the effects of the groundwater, both during dewatering (no water pressure) and after dewatering has been stopped and full water pressure is developed.

Strength design of concrete requires the amplification of the loads. However, in this case amplifying the loads will result in a quasi "unstable" condition of the structure and a meaningless soil pressure distribution. In lieu of amplifying the loads, and then reducing the strength of the reinforced concrete mat, the design is done with ASD load cases with modified phi factors to account for both the reduction in strength and the amplification of the load effects.

SAFE TOTAL GRAVITY LOADS AT FOUNDATION 5/16/05

301 Mission Street
Podium Foundation Mat

• VALUES FROM JP'S GRAVITY CALCS (see 4069-JP-gravity columns.xls)
— COLUMN DEAD & LIVE LOADS
— WALL DEAD & LIVE LOADS



TOT DL = 31,892 K + 46,813 K = 78,705 K

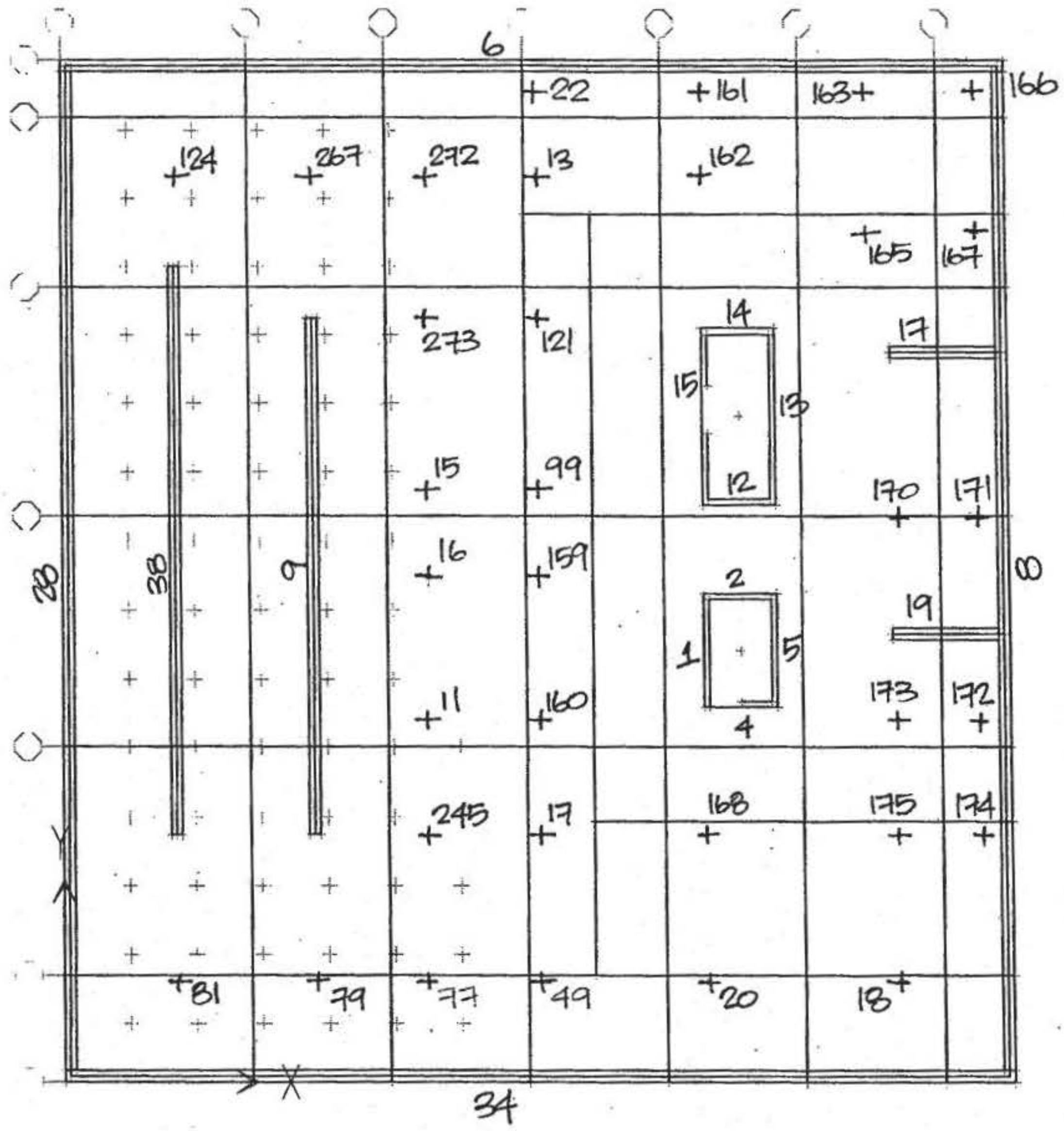
TOT LL = 9,003 K + 7,031 K = 16,034 K

94,739 K

6.2-2

SAFE v8.0.1 - File: 4069-20041214-Podium - December 20,2004 14:47 - Scale: User Scale

– REFER TO 4069-JP-Podium gravity columns and walls.xls for column & wall dead/live loads



6.2-3

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Job no. : 4069
Client : Handel Architects
Project : 301 Mission street
Engineer : Jiri Partoid
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COLUMN REACTIONS SUMMARY

L4 COLUMN

I.D.	Shape	Diameter (in)	Section (in2)	Height (ft)	Weight (k)	SAFE FZ DL (k)	SAFE FZ LL (k)
13	COL30DIA	30	709	10	7	53	17
17	COL24DIA	24	454	10	5	32	9
22	COL30DIA	30	709	10	7	23	6
99	COL24DIA	24	454	10	5	83	20
121	COL24DIA	24	454	10	5	72	22
159	COL24DIA	24	454	10	5	49	15
160	COL24DIA	24	454	10	5	54	17
161	COL24DIA	24	454	10	5	40	12
162	COL24DIA	24	454	10	5	119	40
163	COL24DIA	24	454	10	5	59	18
165	COL24DIA	24	454	10	5	92	31
166	COL30DIA	30	709	10	7	25	7
167	COL30DIA	30	709	10	7	30	9
168	COL24X48	N/A	1152	10	12	69	21
170	COL30DIA	30	709	10	7	72	24
171	COL30DIA	30	709	10	7	38	11
172	COL24DIA	24	454	10	5	25	7
173	COL30DIA	30	709	10	7	60	20
174	COL24DIA	24	454	10	5	16	4
175	COL24DIA	24	454	10	5	50	15
SUM:						1039	328

L3 COLUMN

I.D.	Shape	Diameter (in)	Section (in2)	Height (ft)	Weight (k)	SAFE FZ DL (k)	SAFE FZ LL (k)
13	COL30DIA	30	709	14	11	178	61
14	COL24DIA	24	454	14	7	51	21
17	COL30DIA	30	709	14	11	98	36
20	COL24DIA	24	454	14	7	196	90
22	COL24DIA	24	454	14	7	34	8
49	COL24DIA	24	454	14	7	127	58
50	COL24DIA	24	454	14	7	140	65
51	COL24DIA	24	454	14	7	113	50
52	COL24DIA	24	454	14	7	58	25
53	COL24DIA	24	454	14	7	59	26
54	COL24DIA	24	454	14	7	59	25
99	COL30DIA	30	709	14	11	182	61
121	COL30DIA	30	709	14	11	349	117
124	COL24X48	N/A	1152	14	17	42	17
159	COL30DIA	30	709	14	11	128	49
160	COL30DIA	30	709	14	11	134	52
161	COL24DIA	24	454	14	7	77	19
162	COL30DIA	30	709	14	11	131	34
163	COL24DIA	24	454	14	7	91	23
165	COL24DIA	24	454	14	7	111	30
166	COL24DIA	24	454	14	7	49	12
167	COL24DIA	24	454	14	7	54	13
168	COL30DIA	30	709	14	11	103	25
170	COL24DIA	24	454	14	7	86	23
171	COL24DIA	24	454	14	7	64	16
172	COL24DIA	24	454	14	7	43	11
173	COL24DIA	24	454	14	7	70	19
174	COL24DIA	24	454	14	7	34	8
175	COL24DIA	24	454	14	7	80	20
SUM:						2939	1012

L2 COLUMN

I.D.	Shape	Diameter (in)	Section (in2)	Height (ft)	Weight (k)	SAFE FZ DL (k)	SAFE FZ LL (k)
7	24X24	N/A	576	16	9	68	25
8	HSS16X16	16	202	16	3	58	21
9	HSS16X16	16	202	16	3	68	24
10	HSS16X16	16	202	16	3	89	26
13	36X36	N/A	1296	16	21	79	40
14	24DIA	24	454	16	7	73	20
15	24DIA	24	454	16	7	178	65
16	24DIA	24	454	16	7	177	62
17	30DIA	30	709	16	12	502	77
22	24DIA	24	454	16	7	48	22
50	24DIA	24	454	16	7	92	36
51	24DIA	24	454	16	7	289	47
52	24DIA	24	454	16	7	21	7
53	24DIA	24	454	16	7	114	35
54	24DIA	24	454	16	7	105	31
99	30DIA	30	709	16	12	129	61
121	36X36	N/A	1296	16	21	110	65
159	30DIA	30	709	16	12	115	52
160	30DIA	30	709	16	12	100	37
161	24DIA	24	454	16	7	82	36
162	30DIA	30	709	16	12	148	75
163	24DIA	24	454	16	7	94	44
165	87X24	N/A	2088	16	34	119	61
166	24DIA	24	454	16	7	56	24
167	24DIA	24	454	16	7	67	30
168	30DIA	30	709	16	12	462	74
170	30DIA	30	709	16	12	91	46
171	24DIA	24	454	16	7	59	31
172	30DIA	30	709	16	12	30	20
173	24DIA	24	454	16	7	40	31
174	24DIA	24	454	16	7	206	28
175	87X24	N/A	2088	16	34	401	82
SUM:						4281	1326

L1 COLUMN

I.D.	Shape	Diameter (in)	Section (in2)	Height (ft)	Weight (k)	SAFE FZ DL (k)	SAFE FZ LL (k)
7	24X24	N/A	576	16	9	180	48
8	HSS16X16	16	202	16	3	101	36
9	HSS16X16	16	202	16	3	118	41
10	HSS16X16	16	202	16	3	134	61
11	24DIA	24	454	16	7	146	49
13	24X48	N/A	1152	16	19	117	54
14	24DIA	24	454	16	7	254	78
15	36DIA	36	1021	16	17	103	37
16	24DIA	24	454	16	7	102	38
17	36DIA	36	1021	16	17	203	41
18	24X48	N/A	1152	16	19	185	18
20	24X48	N/A	1152	16	19	206	19
22	24DIA	24	454	16	7	29	13
49	24X48	N/A	1152	16	19	180	17
52	24DIA	24	454	16	7	121	32
53	24DIA	24	454	16	7	253	74
54	24DIA	24	454	16	7	204	57
77	24X48	N/A	1152	16	19	187	18
79	24X48	N/A	1152	16	19	206	22
81	24X48	N/A	1152	16	19	219	48
99	36DIA	36	1021	16	17	130	61
121	36DIA	36	1021	16	17	141	67
159	30DIA	30	709	16	12	112	52
160	30DIA	30	709	16	12	105	50
161	24DIA	24	454	16	7	27	13
162	30DIA	30	709	16	12	146	73
163	30DIA	30	709	16	12	75	37
165	X48Y24	N/A	1152	16	19	119	69
166	24DIA	24	454	16	7	24	11
167	24DIA	24	454	16	7	24	12
168	36DIA	36	1021	16	17	309	64
170	30DIA	30	709	16	12	96	48
171	24DIA	24	454	16	7	30	14
172	24DIA	24	454	16	7	24	12
173	30DIA	30	709	16	12	68	35
174	X48Y24	N/A	1152	16	19	59	9
175	X48Y24	N/A	1152	16	19	235	46
245	36DIA	36	1021	16	17	307	53
267	30DIA	30	709	16	12	146	53
272	30DIA	30	709	16	12	123	44
273	30DIA	30	709	16	12	102	36
SUM:						9633	1634

B2 COLUMN

I.D.	Shape	Diameter (in)	Section (in2)	Height (ft)	Weight (k)	SAFE FZ DL (k)	SAFE FZ LL (k)
11	COL24X48	N/A	1152	8	10	65	18
13	COL24X48	N/A	1152	8	10	81	26
15	COL24X48	N/A	1152	8	10	71	23
16	COL24X48	N/A	1152	8	10	54	18
17	COL24X48	N/A	1152	8	10	93	30
18	COL24X48	N/A	1152	8	10	85	27
20	COL24X48	N/A	1152	8	10	105	34
22	COL30DIA	30	709	8	6	22	7
49	COL24X48	N/A	1152	8	10	81	28
77	COL24X48	N/A	1152	8	10	68	21
79	COL24X48	N/A	1152	8	10	72	23
81	COL24X48	N/A	1152	8	10	79	25
99	COL24X48	N/A	1152	8	10	103	33
121	COL24X48	N/A	1152	8	10	116	38
124	COL24X48	N/A	1152	8	10	50	16
159	COL24X48	N/A	1152	8	10	79	25
160	COL24X48	N/A	1152	8	10	80	26
161	COL24DIA	24	454	8	4	22	7
162	COL24X48	N/A	1152	8	10	110	36
163	COL30DIA	30	709	8	6	57	18
165	COLX48Y24	N/A	1152	8	10	90	29
166	COL24DIA	24	454	8	4	13	4
167	COL24DIA	24	454	8	4	19	6
168	COL36DIA	36	1021	8	9	131	42
170	COL30DIA	30	709	8	6	72	23
171	COL24DIA	24	454	8	4	23	7
172	COL24DIA	24	454	8	4	17	5
173	COL30DIA	30	709	8	6	56	18
174	COLX48Y24	N/A	1152	8	10	17	5
175	COLX48Y24	N/A	1152	8	10	100	32
245	COL24X48	N/A	1152	8	10	69	22
267	COL30DIA	30	1152	8	10	81	26
272	COL30DIA	30	1152	8	10	66	21
273	COL24X48	N/A	1152	8	10	81	26
SUM:						2316	746

9*(L4)+(L3)+(L2)+(L1)+4*(B2) — REACTION AT B5

Column I.D.	FZ DL (k)	FZ LL (k)	1.4DL+1.7LL (k)
11	410	120	777
13	1329	408	2556
15	721	209	1364
16	601	171	1077
17	1584	356	2823
18	582	128	1031
20	687	245	1656
22	528	126	947
49	690	150	1201
77	467	102	855
79	553	116	971
81	592	145	1081
99	1540	483	2994
121	1642	591	3583
124	297	81	553
159	1228	393	2388
160	1261	415	2470
161	713	201	1340
162	2054	682	4036
163	1114	343	2142
165	1678	643	3273
166	507	125	922
167	596	158	1104
168	2201	622	3968
170	1327	426	2582
171	881	185	1269
172	471	129	880
173	1056	337	2052
174	623	104	1049
175	1707	395	3060
245	639	142	1136
267	523	157	1000
272	436	129	830
273	478	140	905
0	31893	9004	59957

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Job no. : 4069
Client : Handel Architects
Project : 301 Mission street
Engineer : Jiri Partoid
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WALL REACTIONS SUMMARY

L4 WALL

I.D.	Type	SAFE FZ DL (k)	SAFE FZ LL (k)
1	WALL24	147	30
2	WALL24	57	7
4	WALL24	86	17
5	WALL24	110	18
12	WALL24	83	9
13	WALL24	156	21
14	WALL30	90	18
15	WALL24	218	41
17	WALL24	157	34
18	WALL24	121	22
SUM:		1205	216

L3 WALL

I.D.	Type	SAFE FZ DL (k)	SAFE FZ LL (k)
1	WALL24	181	21
2	WALL24	76	8
4	WALL24	109	18
5	WALL24	136	17
12	WALL24	79	9
13	WALL24	196	22
14	WALL30	109	17
15	WALL24	238	29
17	WALL24	209	38
19	WALL24	158	25
38	WALL24	502	157
SUM:		1974	360

L2 WALL

I.D.	Type	SAFE FZ DL (k)	SAFE FZ LL (k)
1	WALL24	172	51
2	WALL24	84	18
3	WALL24	1271	261
4	WALL24	83	25
5	WALL24	149	34
12	WALL24	86	17
13	WALL24	219	42
14	WALL30	118	31
15	WALL24	294	80
17	WALL24	229	75
19	WALL24	187	52
38	WALL24	1205	151
47	WALL14ATK	2013	122
SUM:		6113	958

L1 WALL

I.D.	Type	SAFE FZ DL (k)	SAFE FZ LL (k)
1	WALL24	284	51
2	WALL24	142	19
3	WALL24	1710	328
4	WALL24	155	26
5	WALL24	232	34
6	WALL24	1047	116
8	WALL24	1115	89
12	WALL24	138	17
13	WALL24	383	57
14	WALL30	170	32
15	WALL24	425	75
17	WALL24	246	51
19	WALL24	235	44
34	WALL24	1632	117
38	WALL24	1677	240
SUM:		9873	1297

B2 WALL

I.D.	Type	SAFE FZ DL (k)	SAFE FZ LL (k)
1	WALL24	127	25
2	WALL24	56	8
3	WALL24	656	90
4	WALL24	80	16
5	WALL24	100	17
6	WALL24	641	58
8	WALL24	637	51
12	WALL24	58	9
13	WALL24	168	29
14	WALL30	96	21
15	WALL24	202	40
17	WALL24	99	30
19	WALL24	75	22
28	WALL24	743	84
34	WALL24	576	69
38	WALL24	666	78
SUM:		5080	849

9*(L4)+(L3)+(L2)+(L1)+4*(B2) — REACTION AT B5

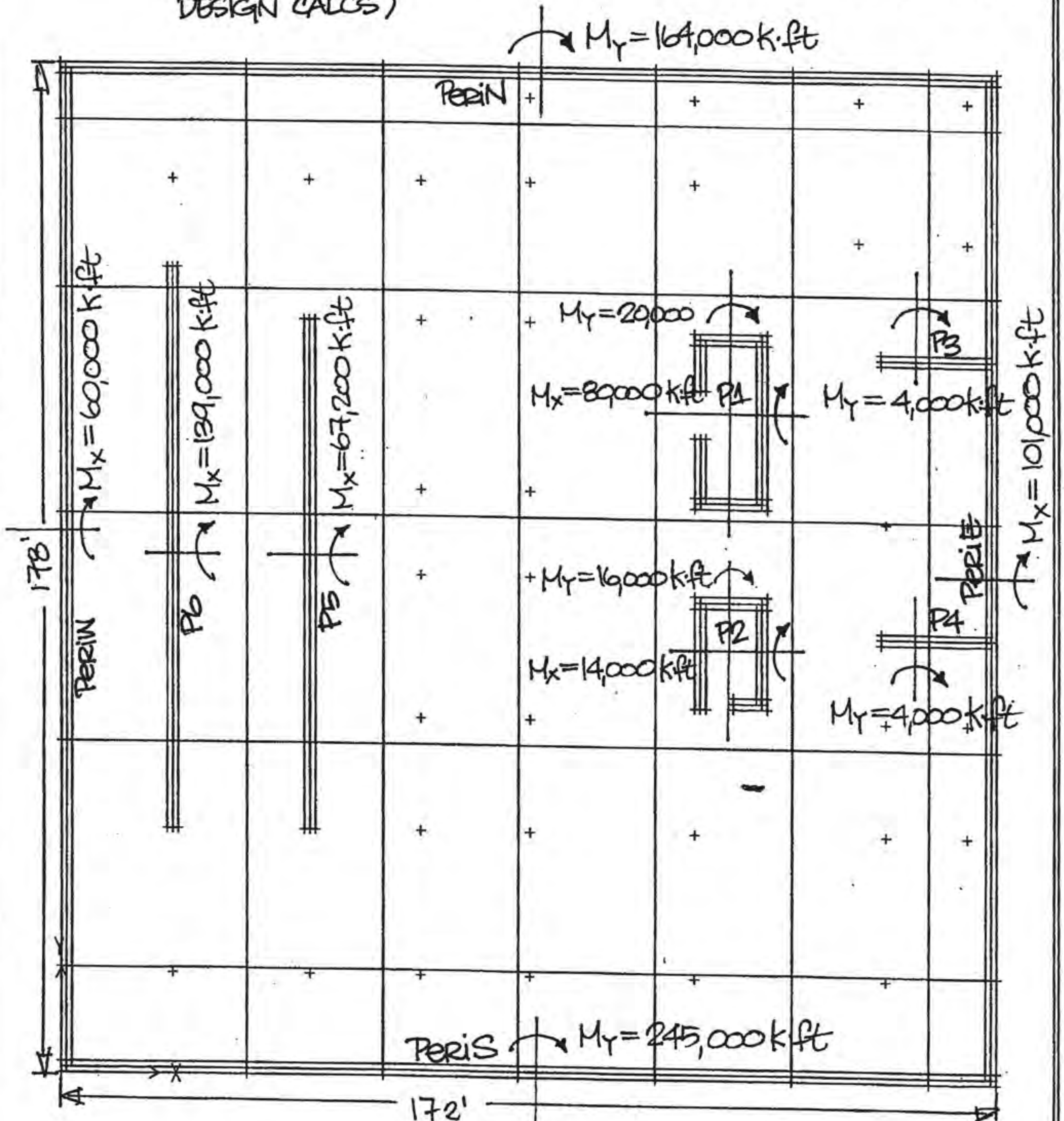
Wall I.D.	FZ DL (k)	FZ LL (k)	1.4DL+1.7LL (k)
1	2426	465	4240
2	1041	139	1694
3	5607	849	9462
4	1447	284	2508
5	1908	313	3293
6	3612	347	5647
8	3664	293	5628
12	1100	158	1810
13	2573	421	4738
14	1588	322	2770.8
15	3732	721	6450
17	2492	568	4408
19	1967	411	3452
28	2973	338	4736
34	4334	391	6732
38	6647	881	9930
	46613	7030	77490

6.2-4

SAFE SEISMIC LOADS @ FOUNDATION 5/11/05

DUE TO → EQ IN X-DIRECTION (EQ_x)

- INCLUDES ↑ 30% EQ IN Y-DIRECTION FOR ORTHOGONAL EFFECTS, 5% TORSIONAL EFFECTS, AND OVERSTRENGTH EFFECTS
- VALUES FROM NJR'S ETABS OUTPUT (SEE MIDRISE SHEARWALL DESIGN CALCS)



— MOMENTS DUE TO EQ_x →

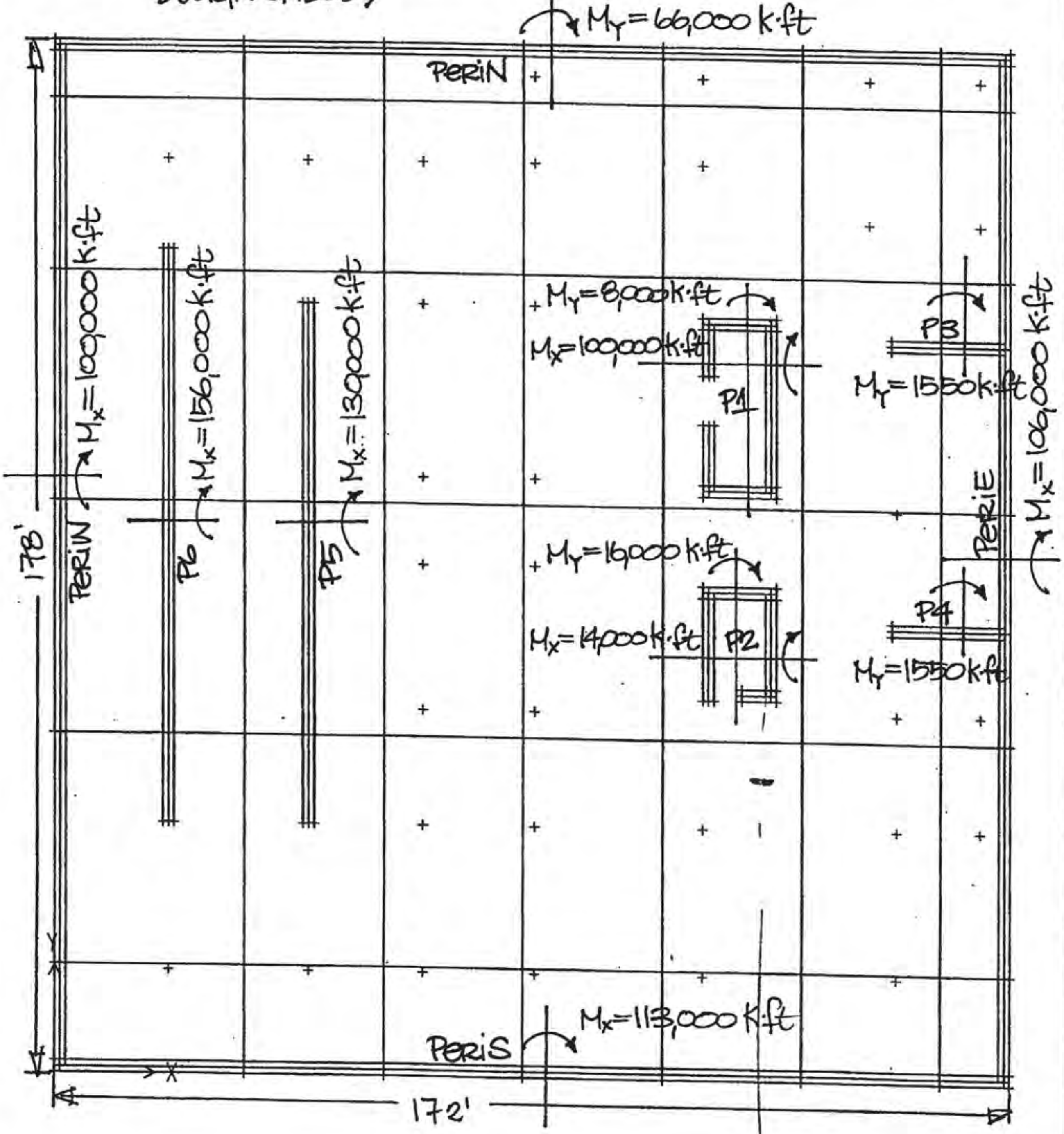
— MOMENTS ESSENTIALLY DUE TO 30% LOAD IN Y-DIRECTION ↑

6.2 S

SAFE SEISMIC LOADS AT FOUNDATION 5/11/05

DUE TO ↑ EQ IN Y-DIRECTION (EQ_Y)

- INCLUDES → 30% EQ IN X-DIRECTION FOR ORTHOGONAL EFFECTS, 5% TORSIONAL EFFECTS, AND OVERSTRENGTH EFFECTS
- VALUES FROM NJR'S ETABS OUTPUT (SEE MIDRISE SHEARWALL DESIGN CALCS)

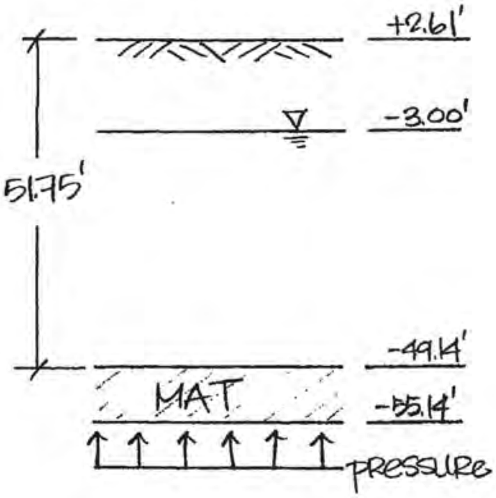


— MOMENTS DUE TO EQ_Y ↑
— MOMENTS ESSENTIALLY DUE TO 30% LOAD IN X-DIRECTION →

DESIMONE

Project: 301 Mission St.
Project No.: 4069
Item: Podium Mat Water Pressure

Page: 1 Of:
Date: 3/1/05
By: MKL Ch'kd:



SFC Datum

@ bottom of mat, waterhead = 52.14'

$$p = (52.14\,ft)(62.4\,lb/ft^3) = 3254\,lb/ft^2$$

→ say $p = 3300\ psf$

↖ water pressure

6.2-7

Project: 301 MISSION	Page 1 Of
Project No. 4069	Date 5/11/05
Item: PODIUM MAT DESIGN COMBOS	By MF Ch'kd

ALLOWABLE STRESS DESIGN PER UBC97

- NON-SEISMIC: $D_{full} + L$
  $0.9[D_{full} + D_{mat}] + H$ *
  $[D_{full} + D_{mat}] + L + H$

  *ADJUSTED FROM 1.0H TO 0.96H FOR SAFE TO ITERATE FOR CONVERGENCE

- SEISMIC W/ WATER:

$0.9D + H \pm E/1.4 \longrightarrow 0.9[D_{full} + D_{mat}] + H \pm EQ_x/1.4$ * (2 CASES)

$0.9[D_{full} + D_{mat}] + H \pm [EQ_x \pm 0.3EQ_y]/1.4$ * (4)

$0.9[D_{full} + D_{mat}] + H \pm EQ_y/1.4$ * (2)

$0.9[D_{full} + D_{mat}] + H \pm [EQ_y \pm 0.3EQ_x]/1.4$ * (4)

$D + L + H \pm E/1.4 \longrightarrow [D_{full} + D_{mat}] + L + H \pm EQ_x/1.4$ (2)

$[D_{full} + D_{mat}] + L + H \pm [EQ_x \pm 0.3EQ_y]/1.4$ (4)

$[D_{full} + D_{mat}] + L + H \pm EQ_y/1.4$ (2)

$[D_{full} + D_{mat}] + L + H \pm [EQ_y \pm 0.3EQ_x]/1.4$ (4)

- SEISMIC W/O WATER:

$0.9D \pm E/1.4 \longrightarrow 0.9D_{full} \pm EQ_x/1.4$ (2)

$0.9D_{full} \pm [EQ_x \pm 0.3EQ_y]/1.4$ (4)

$0.9D_{full} \pm EQ_y/1.4$ (2)

$0.9D_{full} \pm [EQ_y \pm 0.3EQ_x]/1.4$ (4)

$D + L \pm E/1.4 \longrightarrow D_{full} + L \pm EQ_x/1.4$ (2)

$D_{full} + L \pm [EQ_x \pm 0.3EQ_y]/1.4$ (4)

$D_{full} + L \pm EQ_y/1.4$ (2)

$D_{full} + L \pm [EQ_y \pm 0.3EQ_x]/1.4$ (4)

6.2-8

DESIMONE

Project	301 MISSION	Page 1	Of
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Item	PHI FACTORS	By MF	Ch'kd

SINCE CONCRETE USES STRENGTH DESIGN, USE ASD LOAD CASES WITH ϕ FACTORS REDUCED APPROPRIATELY FOR DESIGNING THE MAT REINFORCEMENT:

- LC1: ASD = D + L

  STRENGTH = 1.4D + 1.7L

  $$\text{SCALE FACTOR} = \frac{1.4D + 1.7L}{D+L} = \frac{(78705)(1.4) + 1.7(16034)}{78705 + 16034}$$

  $$= \underline{1.45}$$

- LC2: ASD = 0.9D + H ± E/1.4

  STRENGTH = 0.9 + 1.6H ± 1.0E

  SINCE D ≈ H IN MODEL, SCALE FACTOR $\cong \frac{0.9+1.6}{2} = 1.25$

  SEISMIC SCALE FACTOR = 1.4

  → USE SCALE FACTOR = $\underline{1.4}$

- LC3: ASD = D + L + H ± E/1.4

  STRENGTH = 1.42D + 0.5L + 1.6H ± 1.0E

  SINCE D ≈ H AND LL IS INSIGNIFICANT IN COMPARISON

  $$\text{SCALE FACTOR} = \frac{1.42 + 1.6}{2} = 1.51$$

  SEISMIC SCALE FACTOR = 1.4

  → USE SCALE FACTOR = $\underline{1.51}$

6.2-9

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Project No.	4069	Date 5/18/05	
Item	PHI FACTORS	By MF	Ch'kd

⟹ FOR CONSERVATIVENESS, & SIMPLICITY, MODIFY ϕ BY 1.51 TO COMPARE ASD LOADS FOR CONCRETE DESIGN

SHEAR: $\phi = \frac{0.85}{1.51} = 0.56$

FLEXURE: $\phi = \frac{0.9}{1.51} = 0.60$

6.2-10

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6.3 Detailed Design

6.3 Detailed Design

Tie-Downs – The tie-downs are designed using the maximum tension forces from all load cases. Maximum forces from transient load cases (seismic) are decreased by 75% (equivalent of a 1/3-stress increase in the capacity) and compared to the maximum forces due to any permanent load cases.

One-way Shear – 1-way shear in the foundation mat is checked by inspecting the shear stress contours of the various load combinations. At most locations, the concrete shear capacity is adequate for the respective loads. Some shear reinforcement, however, are required at various locations around shear walls.

Two-way Shear – 2-way shear in the mat is checked by calculating the punching shear capacity for various column sizes found on the podium foundation mat. At all columns, the 2-way shear capacity is greater than the applied load.

Flexure – Flexural reinforcement is designed using all four models in SAFE for both directions on both the top and bottom of the mat.

S A F E (TM)
Version 8.0.0

17 May 2005 14:27:09

Foundation Mat with Tie-Downs @ 60 k/in

G L O B A L F O R C E B A L A N C E

TOTAL FORCE AND MOMENT AT THE ORIGIN, IN GLOBAL COORDINATES

LOADFDL

	FX	FY	FZ	MX	MY	MZ
APPLIED	.000000	.000000	-78704.955	-8.8349E+07	9.3952E+07	.000000
SPRINGS	.000000	.000000	78699.386	8.8341E+07	-9.3943E+07	.000000
TOTAL	.000000	.000000	-5.568654	-7308.213	8573.537	.000000

LOADLL

	FX	FY	FZ	MX	MY	MZ
APPLIED	.000000	.000000	-16033.994	-1.8764E+07	1.9861E+07	.000000
SPRINGS	.000000	.000000	16032.745	1.8762E+07	-1.9859E+07	.000000
TOTAL	.000000	.000000	-1.249038	-1646.571	1923.489	.000000

LOADMAT

	FX	FY	FZ	MX	MY	MZ
APPLIED	.000000	.000000	-29937.179	-3.2294E+07	3.2204E+07	.000000
SPRINGS	.000000	.000000	29935.352	3.2292E+07	-3.2201E+07	.000000
TOTAL	.000000	.000000	-1.826370	-2374.234	2810.456	.000000

LOADWATER

	FX	FY	FZ	MX	MY	MZ
APPLIED	.000000	.000000	101775.596	1.0899E+08	-1.0524E+08	.000000
SPRINGS	.000000	.000000	-101769.857	-1.0898E+08	1.0523E+08	.000000
TOTAL	.000000	.000000	5.739567	7501.840	-8829.959	.000000

LOADGDL

	FX	FY	FZ	MX	MY	MZ
APPLIED	.000000	.000000	-37164.986	-4.0221E+07	4.0014E+07	.000000
SPRINGS	.000000	.000000	37163.095	4.0219E+07	-4.0011E+07	.000000
TOTAL	.000000	.000000	-1.890770	-2554.315	2908.568	.000000

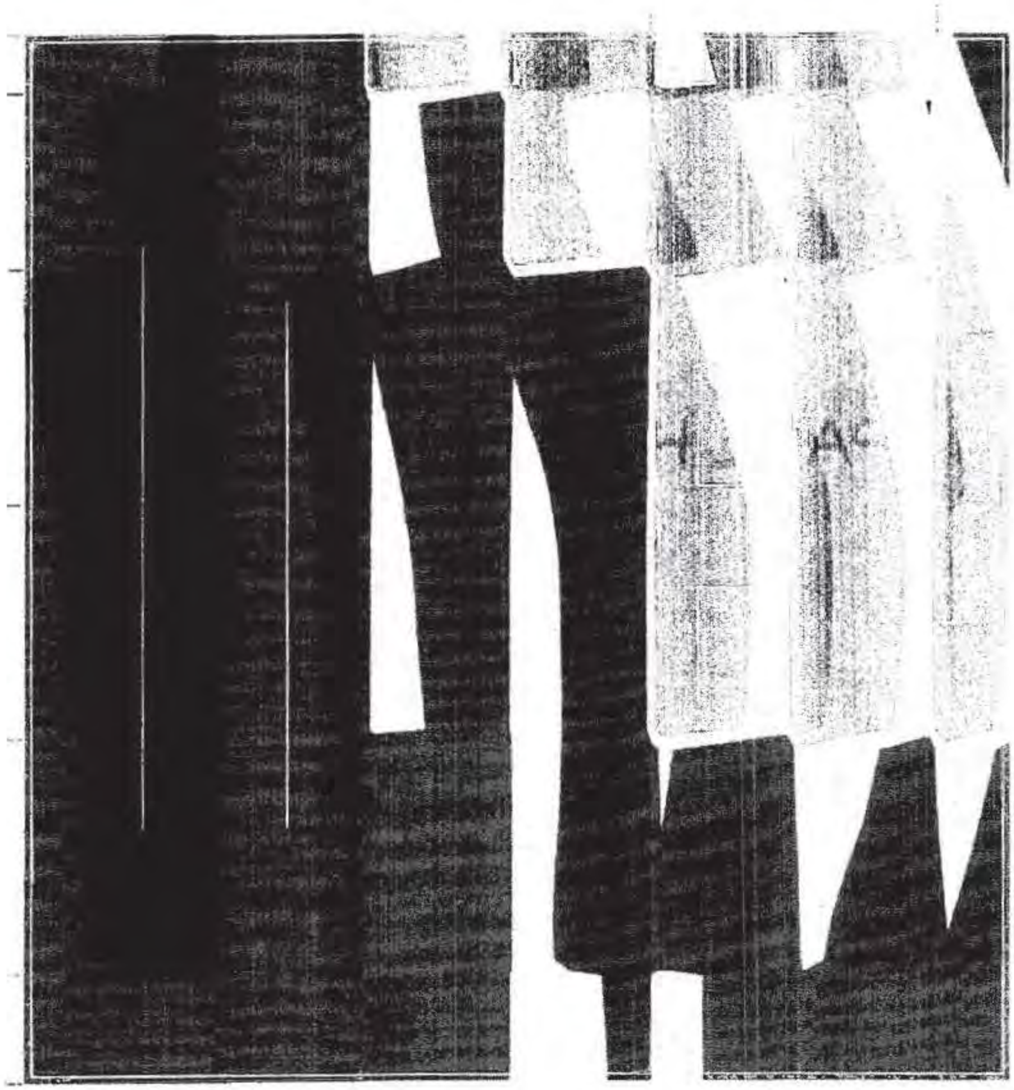
LOAD7DL

	FX	FY	FZ	MX	MY	MZ
APPLIED	.000000	.000000	-57512.964	-6.3328E+07	6.7010E+07	.000000
SPRINGS	.000000	.000000	57509.264	6.3323E+07	-6.7004E+07	.000000
TOTAL	.000000	.000000	-3.699854	-4892.762	5695.348	.000000

6.3-2

$D_{full} + L_{full}$

max = 7.02 ksf



0.00	0.88	1.75	2.63	3.50	4.38	5.25	6.13	7.00

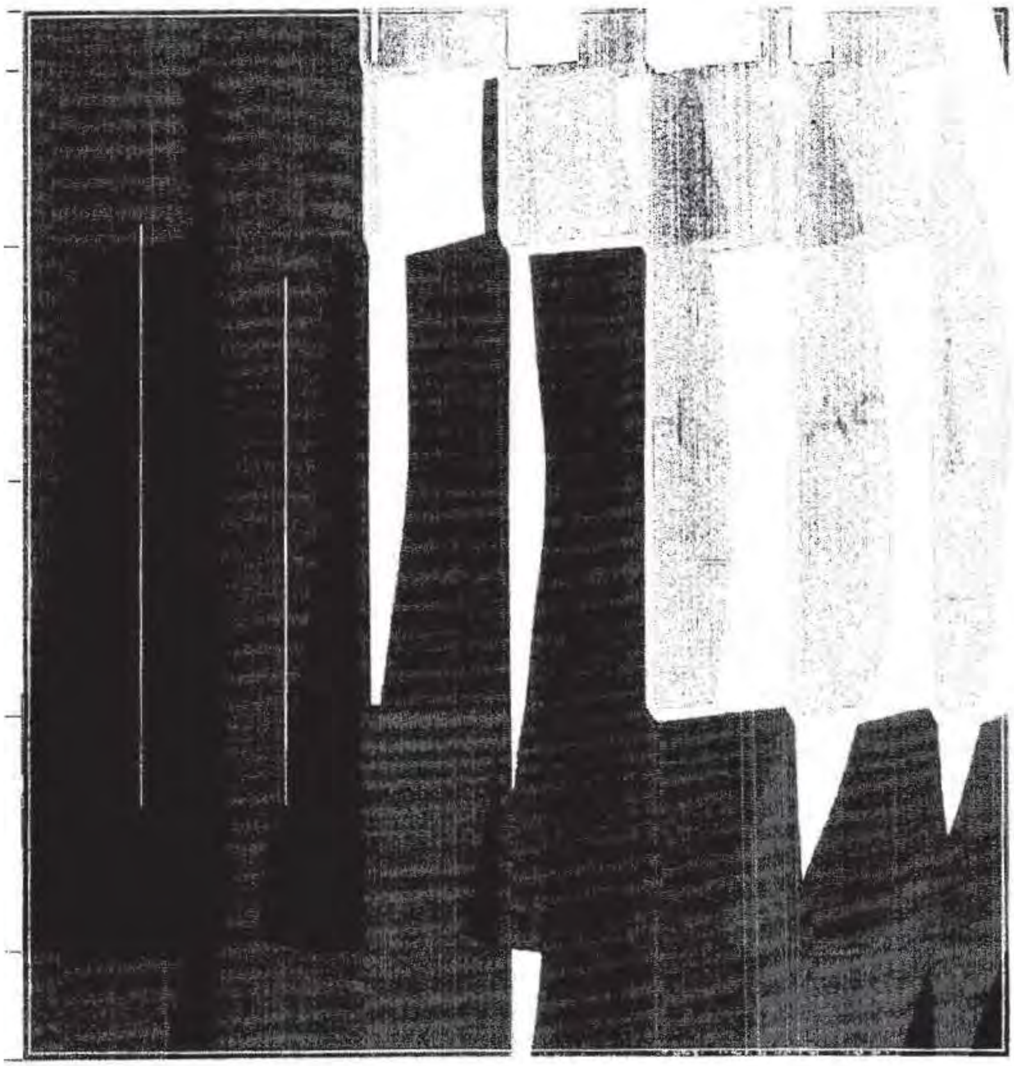
0.9D + H + (EQY + 0.3EQX)/1.4

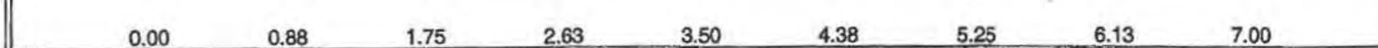
max = 6.67 ksf

0.00	0.88	1.75	2.63	3.50	4.38	5.25	6.13	7.00

D+L+EQY/14

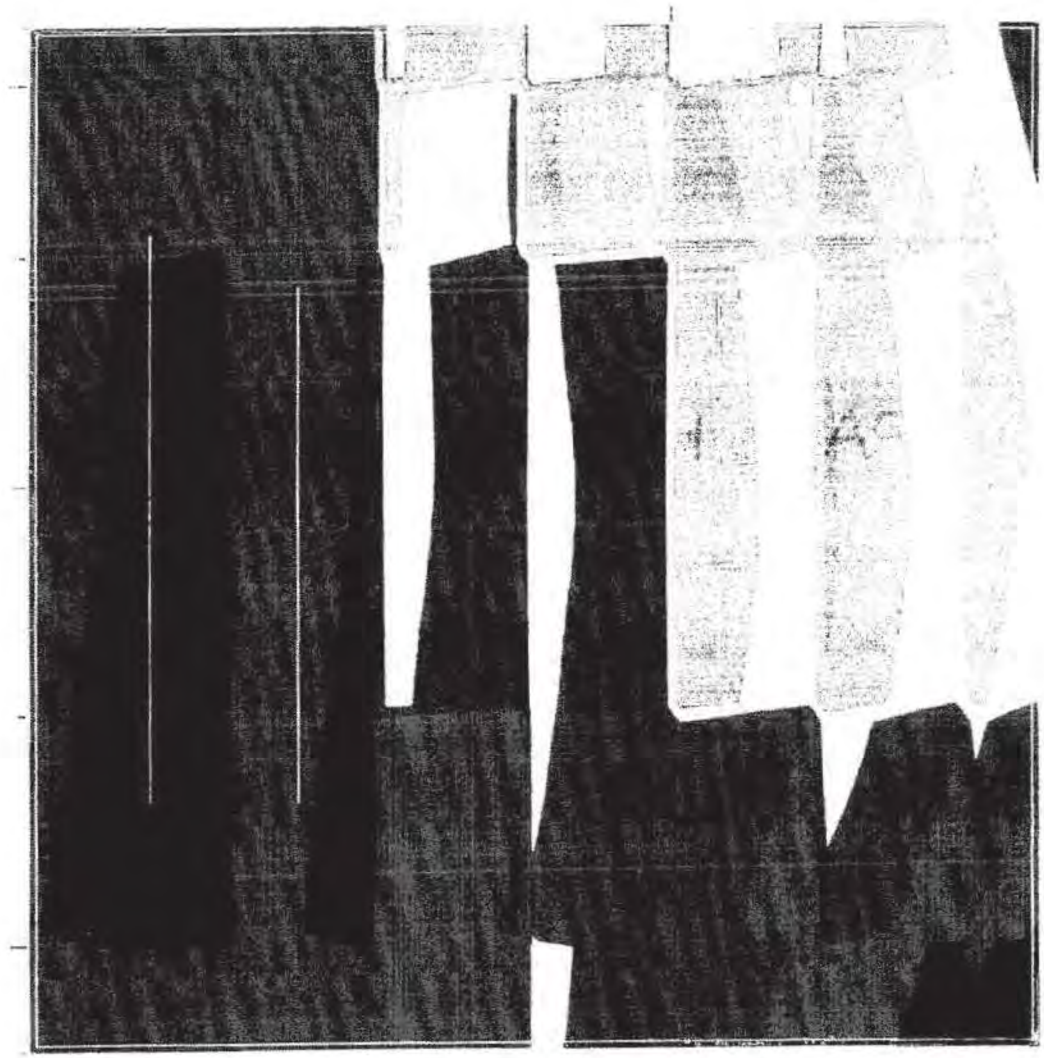
max = 7.95ksf





$D+L+(EQ_Y \pm 0.3EQ_X)/1.4$

max=7.96ksf



0.00	0.88	1.75	2.63	3.50	4.38	5.25	6.13	7.00

SAFE v8.0.6 - File: 4069-20050509-Podium-Mat-no tie-downs-align - May 23,2005 19:16 - Scale: 1 in = 32 ft
Soil Pressure Diagram - (DLEYP2) - Kip-ft Units

6.3-6

SAFE TIE-DOWN FORCES 5/16/05

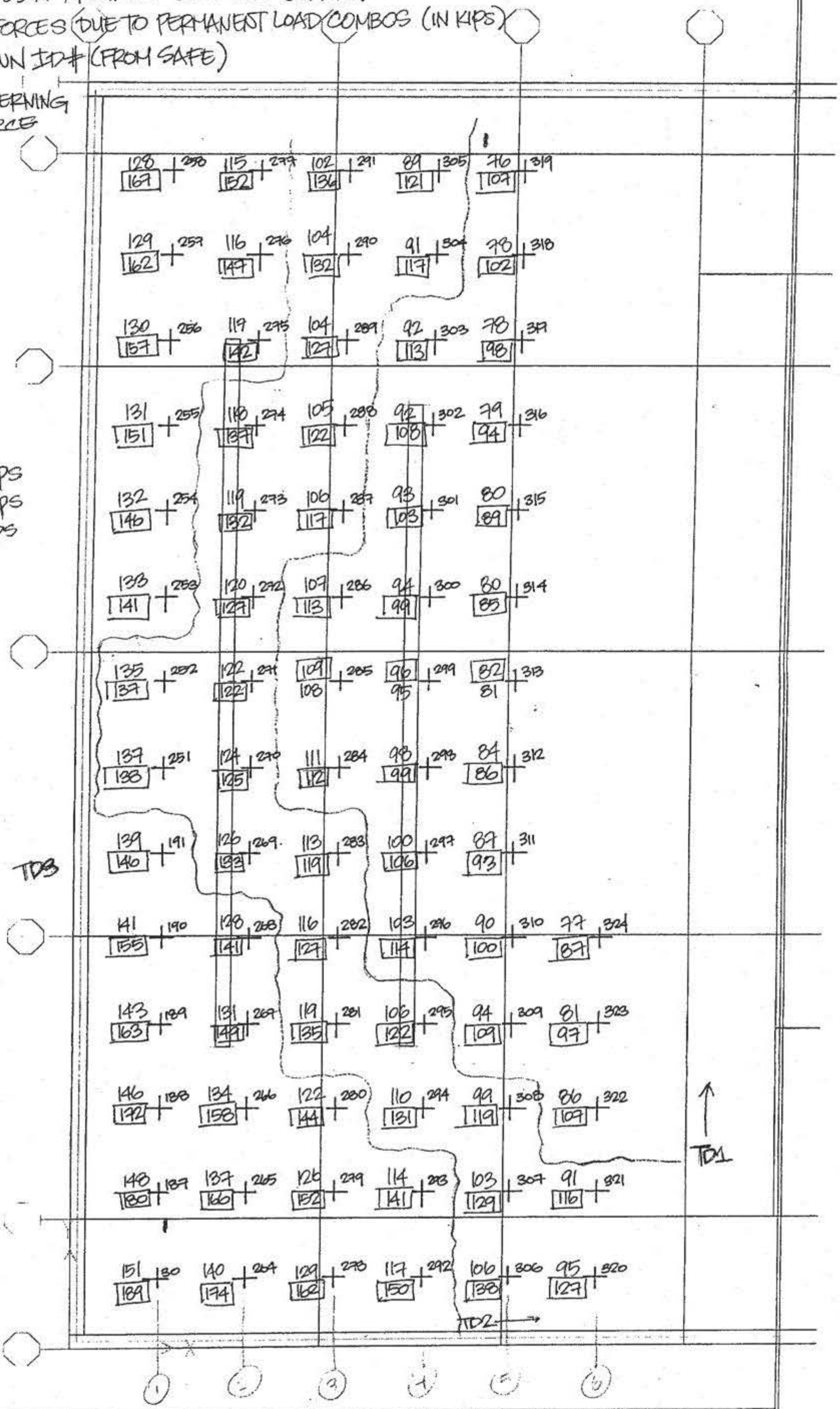
— MAX FORCES (OUT OF ALL LOAD CASES) DUE TO TRANSIENT (SEISMIC) COMBINATIONS; INCLUDES A 3/4 STRESS DECREASE (IN KIPS)

— MAX FORCES DUE TO PERMANENT LOAD COMBOS (IN KIPS)

— TIE-DOWN ID# (FROM SAFE)

☐ GOVERNING FORCE

TD1 – 115 kips
TD2 – 140 kips
TD3 – 190 kips



6.3-7

Project	301 MISSION	Page	Of
Project No.	4069	Date	5/18/05
Item	SHEAR REINFORCEMENT	By	MF Ch'kd

SHEAR CAPACITY – 1 way shear

CONCRETE (8FT. MAT) $\phi V_c = 0.56(2\sqrt{f'_c}\,b_w d)$

$= 0.56(2)\sqrt{5000}(12")(90")$

$\rightarrow \phi V_c = 85.5$ kips

MAX V = 125 kips (at "d" away from walls/columns)

TRY #8 bars @ 24" O.C.:

$\phi V_s = 0.56\left(\frac{A_v f_y d}{s}\right)$ $A_v = \frac{0.79 in^2}{2 bars/ft} = 0.40\ in^2/ft$

$= 0.56\left(\frac{(0.40 in^2)(75 ksi)(90 in)}{24 in}\right)$ $\rightarrow \phi V_s = 63.0$ kips

$\phi(V_c + V_s) = 85.5 + 63.0$ $\rightarrow \phi(V_c + V_s) = 148.5$ kips ✓ OK

CONCRETE (6 FT. MAT) $\phi V_c = 62.7$ kips

MAX V = 85 kips

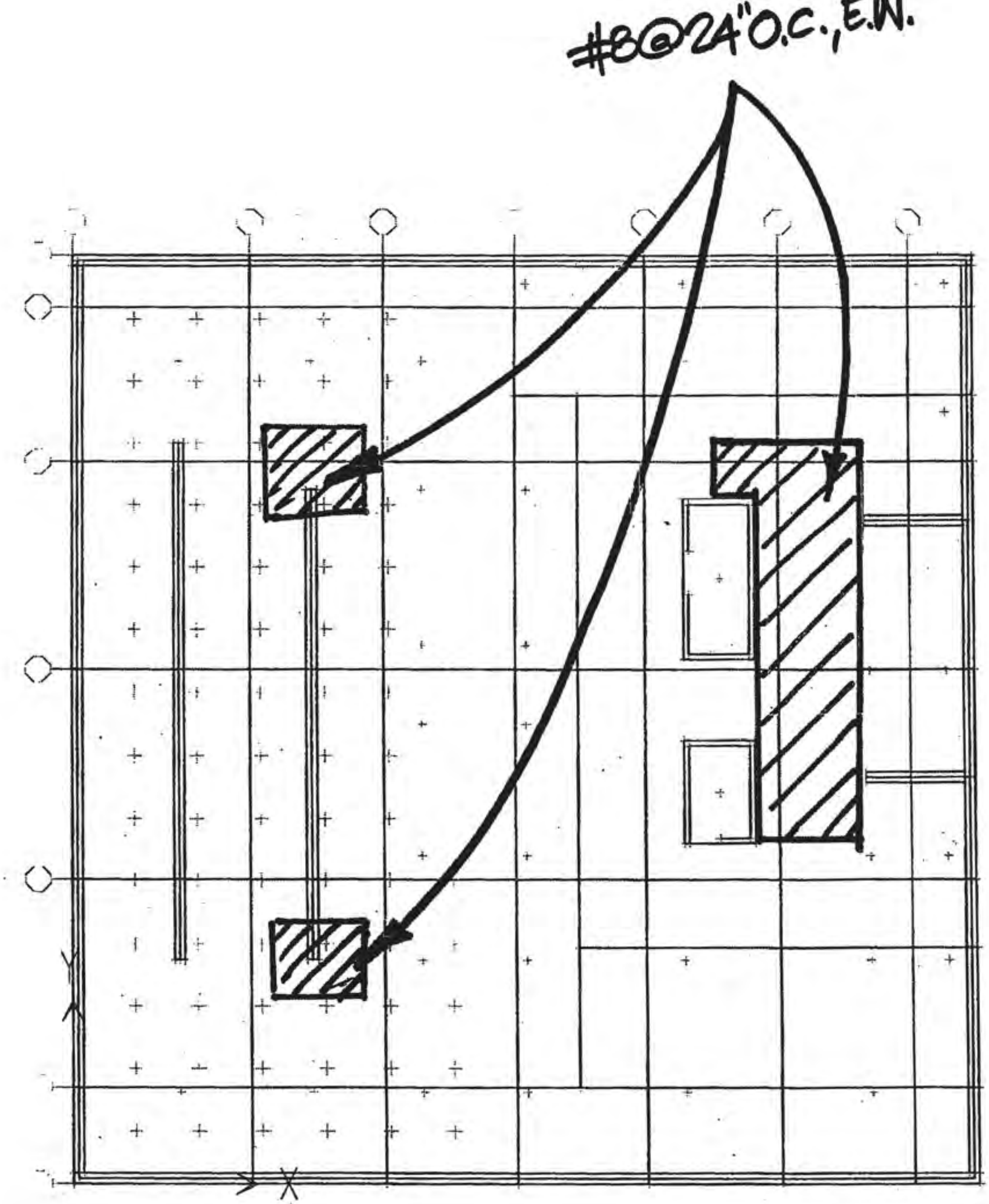
w/ #8 bars @ 24" O.C., $\phi V_s = 46.2$ kips

$\phi(V_c + V_s) = 62.7 + 46.2$ $\rightarrow \phi(V_c + V_s) = 108.9$ kips ✓ OK

6.3-8

SAFE SHEAR REINFORCEMENT

301 Mission Street - Podium Foundation Mat
Foundation Mat with Tie-Downs @ 60 k/in



- NO SHEAR REINF. NEEDED ELSEWHERE

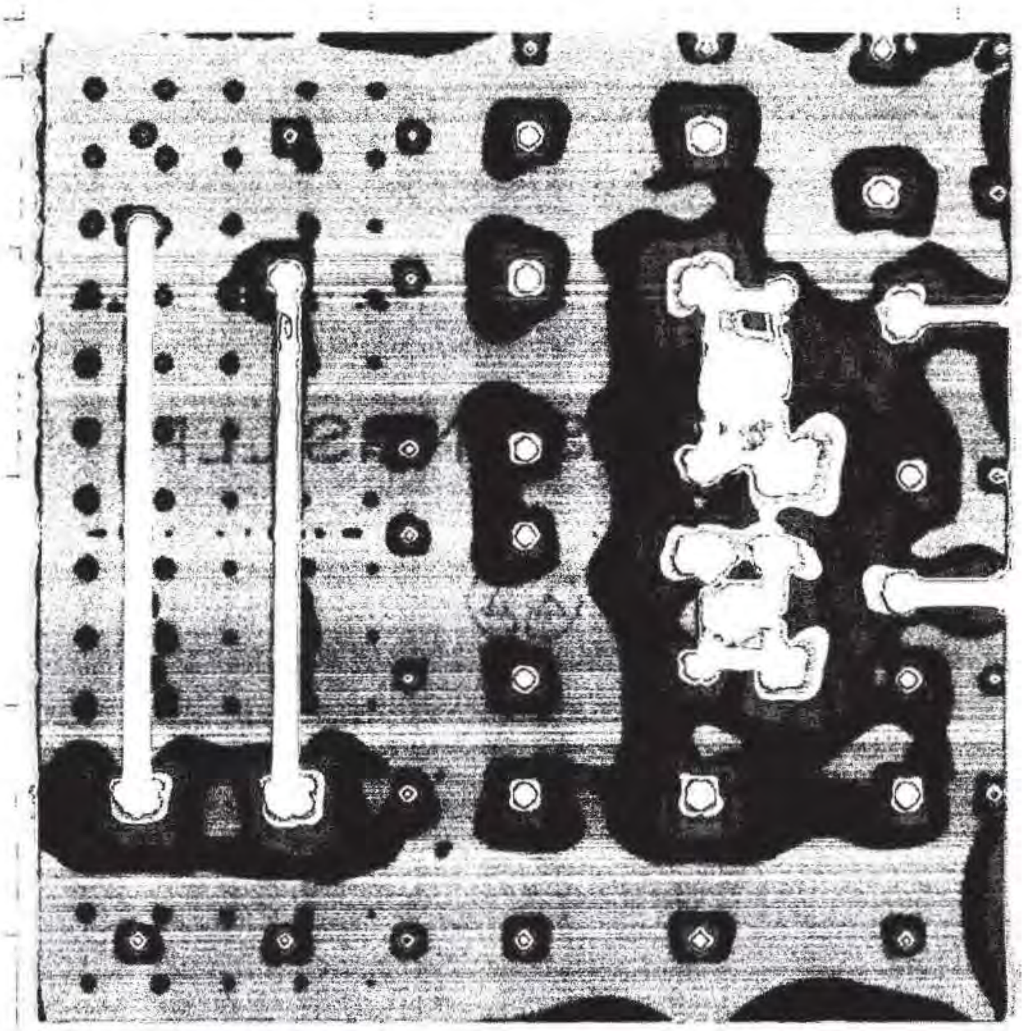
6.3-9

SAFE v8.0.6 - File: 4069-20050509-Podium-Mat-all tie-downs-align - May 23,2005 17:07 - Scale: 1 in = 32 ft
Structural Layer Plan View — Kip-in Units

SAFE MAX SHEARS IN MAT

301 Mission Street - Podium Foundation Mat
Foundation Mat with Tie-Downs @ 60 k/in

0.9D+H+(EQx -0.3EQy)/1.4



CONCRETE (6FT. MAT) | #8@24 (6FT. MAT)

CONCRETE (8FT MAT) | #8@24"O.C. (8FT. MAT)

0. 19. 38. 56. 75. 94. 113. 131. 150.

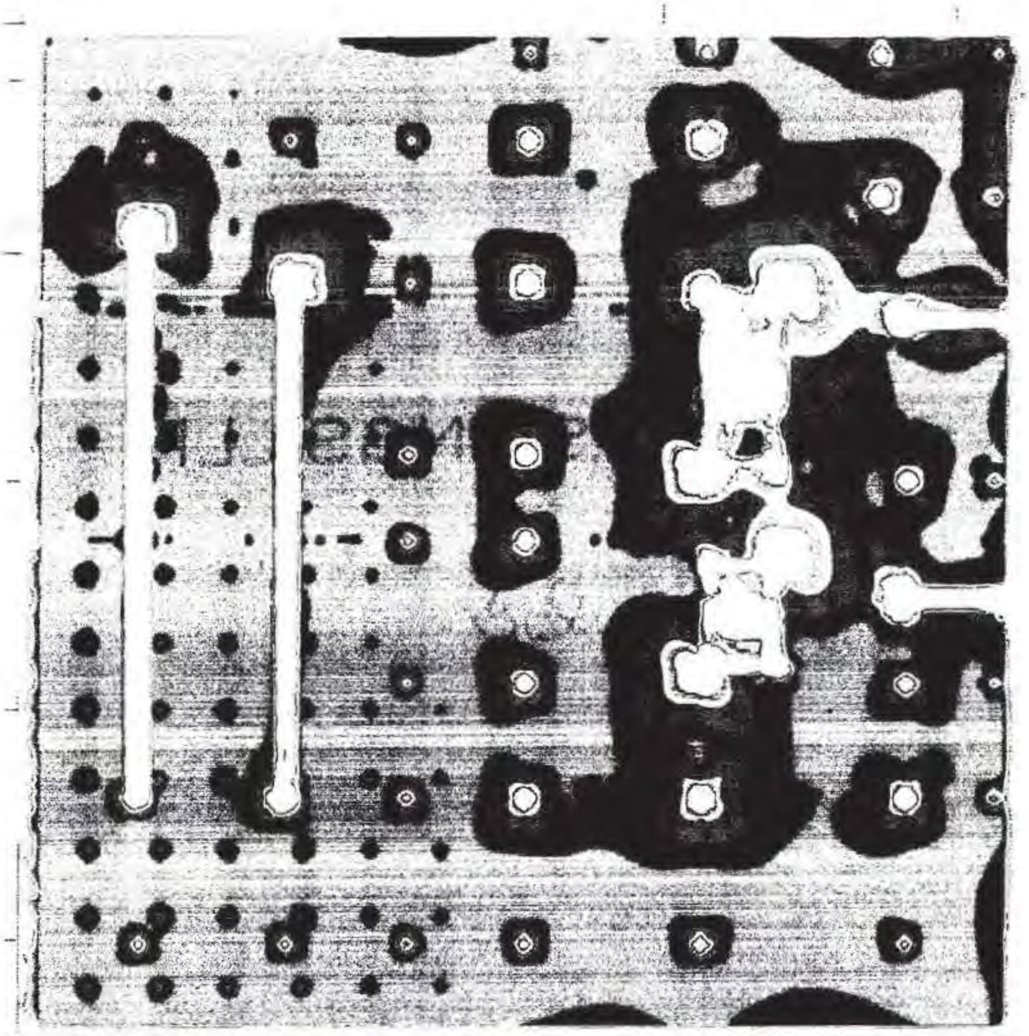
SAFE v8.0.6 - File: 4069-20050509-Podium-Mat-all tie-downs-align - May 23,2005 19:58 - Scale: 1 in = 32 ft
Slab Resultant Vmax Diagram - (9DHEXP2) - Kip-ft Units

6.3-10

SAFE MAX SHEARS IN MAT

301 Mission Street - Podium Foundation Mat
Foundation Mat with Tie-Downs @ 60 k/in

0.9D + H + (EQx + 0.3EQy)/1.4



0.	19.	38.	56.	75.	94.	113.	131.	150.

SAFE v8.0.6 - File: 4069-20050509-Podium-Mat-all tie-downs-align - May 23,2005 19:54 - Scale: 1 in = 32 ft
Slab Resultant Vmax Diagram - (9DHEXP1) - Kip-ft Units

6.3-11

DESIMONE

Project	301 Mission	Page 1	Of
Project No.	4069	Date 5/18/05	
Item	COLUMN PUNCHING SHEAR	By MF	Ch'kd

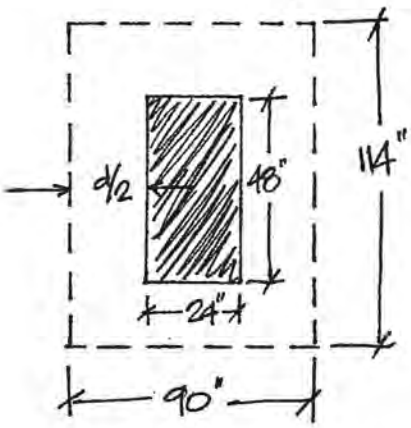
- CHECK COLUMN PUNCHING SHEAR FOR VARIOUS COLUMNS (CHECK WORST LOAD FOR EACH SIZE COLUMN)

• 24X48 COLUMN

ID #162: $D = 2054$, $L = 682$ ON 6FT. MAT

$P_u = 1.4D + 1.7L$

$= 1.4(2054k) + 1.7(682k) \longrightarrow P_u = 4035k$



for 6 ft. MAT, $d = 66"$ so $d/2 = 33"$

$$V_c = \left(2 + \frac{4}{\beta_c}\right)\sqrt{f'_c}\,b_o d \le 4\sqrt{f'_c}\,b_o d$$

$$\beta_c = \frac{48}{24} = 2 \qquad b_o = 2(90" + 114") = 408"$$

$$\longrightarrow V_c = 4\sqrt{f'_c}\,b_o d$$

$$\phi V_c = \phi(4\sqrt{f'_c}\,b_o d)$$

$$= 0.85(4)\sqrt{5000}(408)(66) \longrightarrow \phi V_c = 6474 \text{ kips}$$

$$DCR = \frac{4035k}{6474k} \longrightarrow \underline{\underline{DCR = 0.62}} \quad \checkmark OK$$

6.3-12

• 30 DIA. COLUMN

ID#163: D = 1114 ON 6 FT. MAT
L = 343

$P_u = 1.4(1114) + 1.7(343) \longrightarrow P_u = 2143$ kips

for 6 ft. mat, d = 66"

$\beta_c = \frac{30"}{30"} = 1$ (CIRCULAR)

$\longrightarrow V_c = 4\sqrt{f'_c}\, b_o d$

$b_o = \pi d = \pi(30" + 66") = 301.6$ in

$\phi V_c = 0.85(4)\sqrt{5000}\,(301.6)(66) \longrightarrow \phi V_c = 4785.6$ kips

$DCR = \frac{2143k}{4785.6k} \longrightarrow \underline{\underline{DCR = 0.45}}$ ✓ ok

• 36 DIA. COLUMN

ID # 168: D = 2201 ON 6 FT. MAT
L = 522

$P_u = 1.4(2201) + 1.7(522) \longrightarrow P_u = 3968.8k$

SINCE CIRCULAR, $\beta_c = 1$ AND $V_c = 4\sqrt{f'_c}\, b_o d$

6.3-13

$b_o = \pi d = \pi(36" + 66") \rightarrow b_o = 320.4"$

$\phi V_c = 0.85(4)\sqrt{5000}(320.4")(66") \rightarrow \phi V_c = 5085$ kips

$DCR = \frac{3968.8}{5085} \rightarrow \underline{\underline{DCR = 0.78}}$ ✓ OK

• 24 DIA. COLUMN

ID #161: D = 713 ON 6 FT. MAT
L = 201

$P_u = 1.4(713) + 1.7(201) \rightarrow P_u = 1340k$

SINCE CIRCULAR, $\beta_c = 1$ AND $V_c = 4\sqrt{f'_c}\, b_o d$

$b_o = \pi d = \pi(24" + 66") \rightarrow b_o = 282.7"$

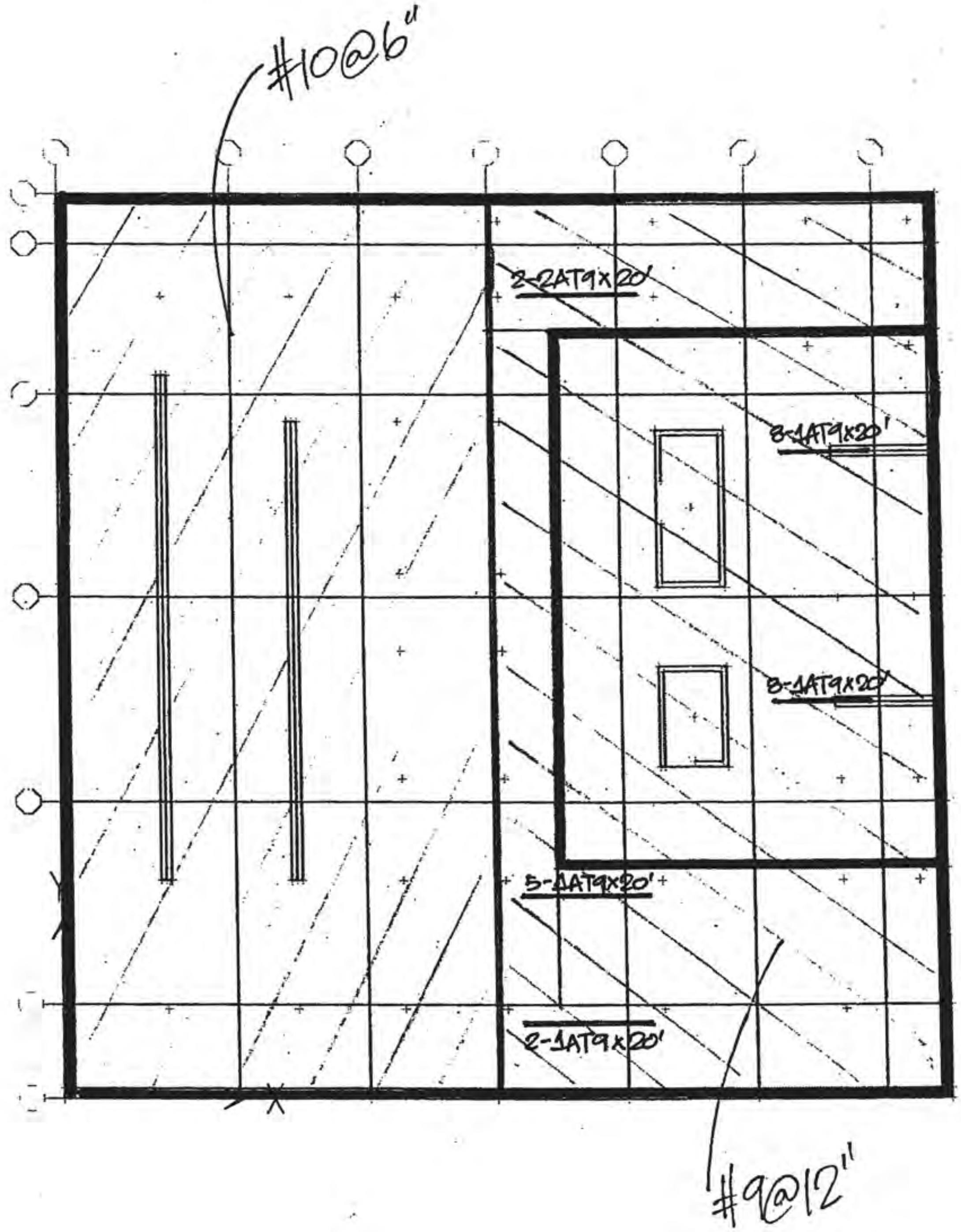
$\phi V_c = 0.85(4)\sqrt{5000}(282.7")(66") \rightarrow \phi V_c = 4486k$

$DCR = \frac{1340}{4486} \rightarrow \underline{\underline{DCR = 0.30}}$ ✓ OK

6.3-14

SAFE FLEXURAL REINFORCEMENT 301 Mission Street - Podium Foundation Mat
Foundation Mat with Tie-Downs @ 60 k/in

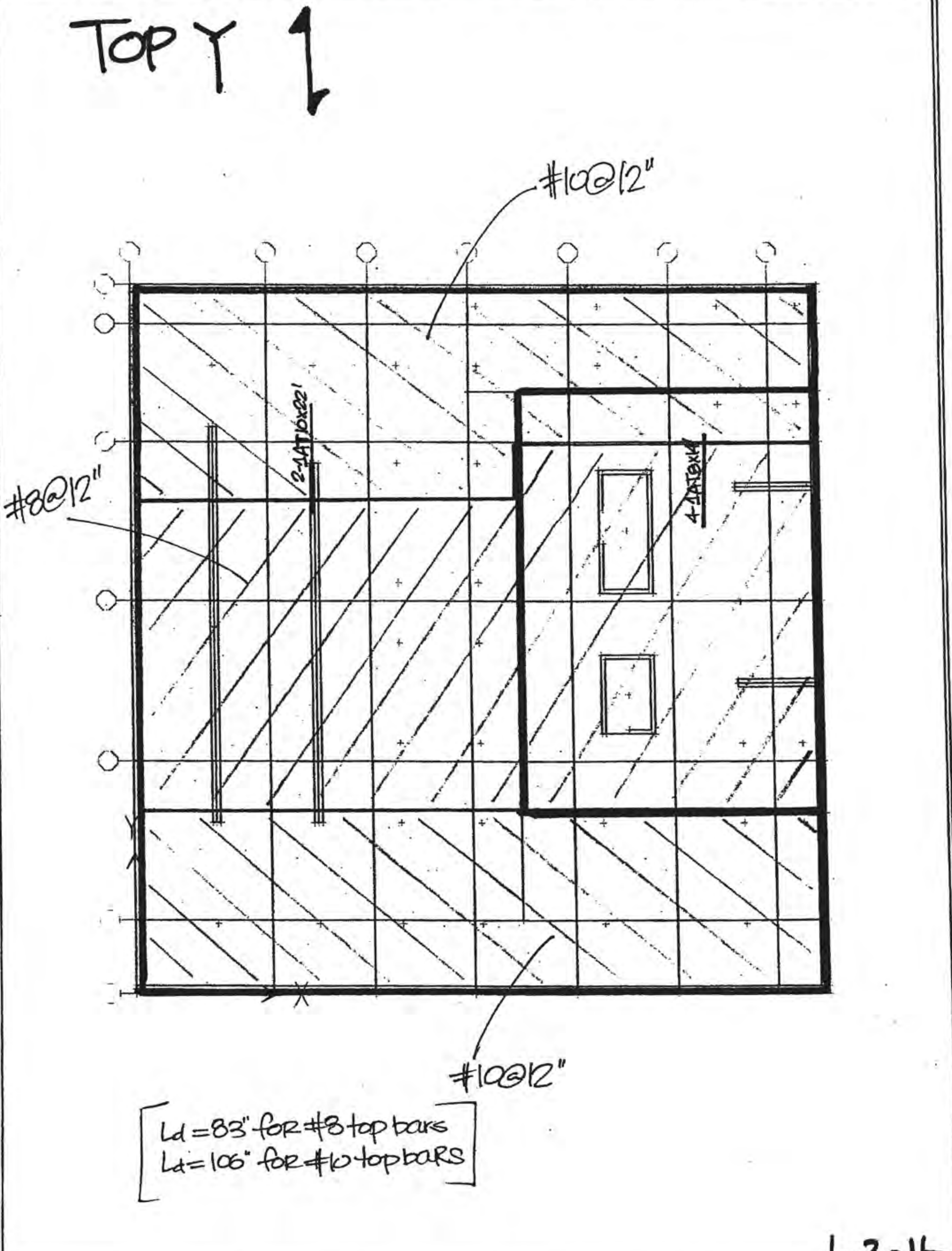
TOP X-DIRECTION



L3-15

SAFE FLEXURAL REINFORCEMENT

301 Mission Street - Podium Foundation Mat
Foundation Mat with Tie-Downs @ 60 k/in

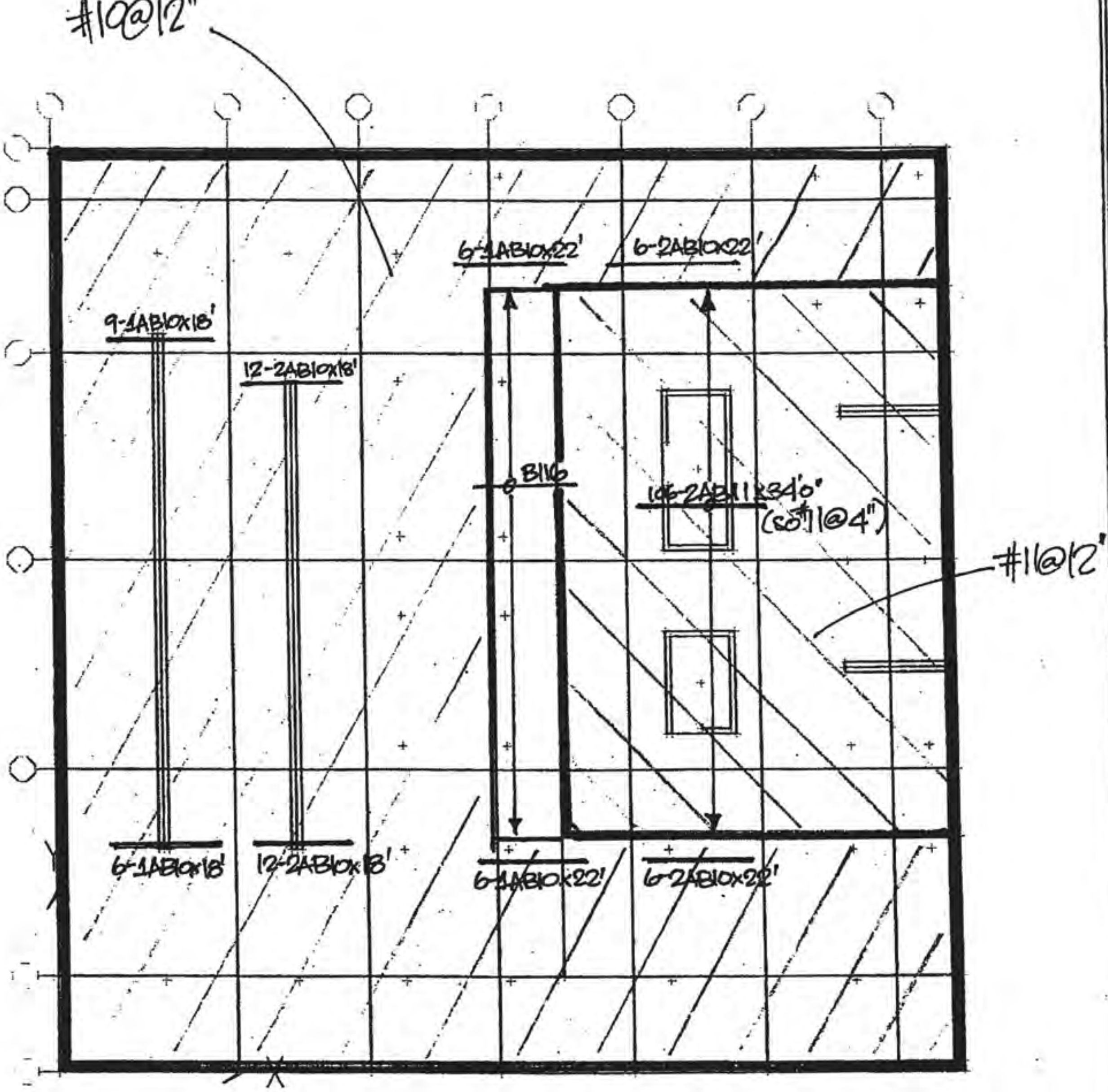


6.3-16

SAFE v8.0.6 - File: 4069-20050509-Podium-Mat-no tie-downs-align - May 18 2005 17:03 - Scale: 1 in = 32 ft

DODSONNOC00000342

BOTTOM X-DIRECTION

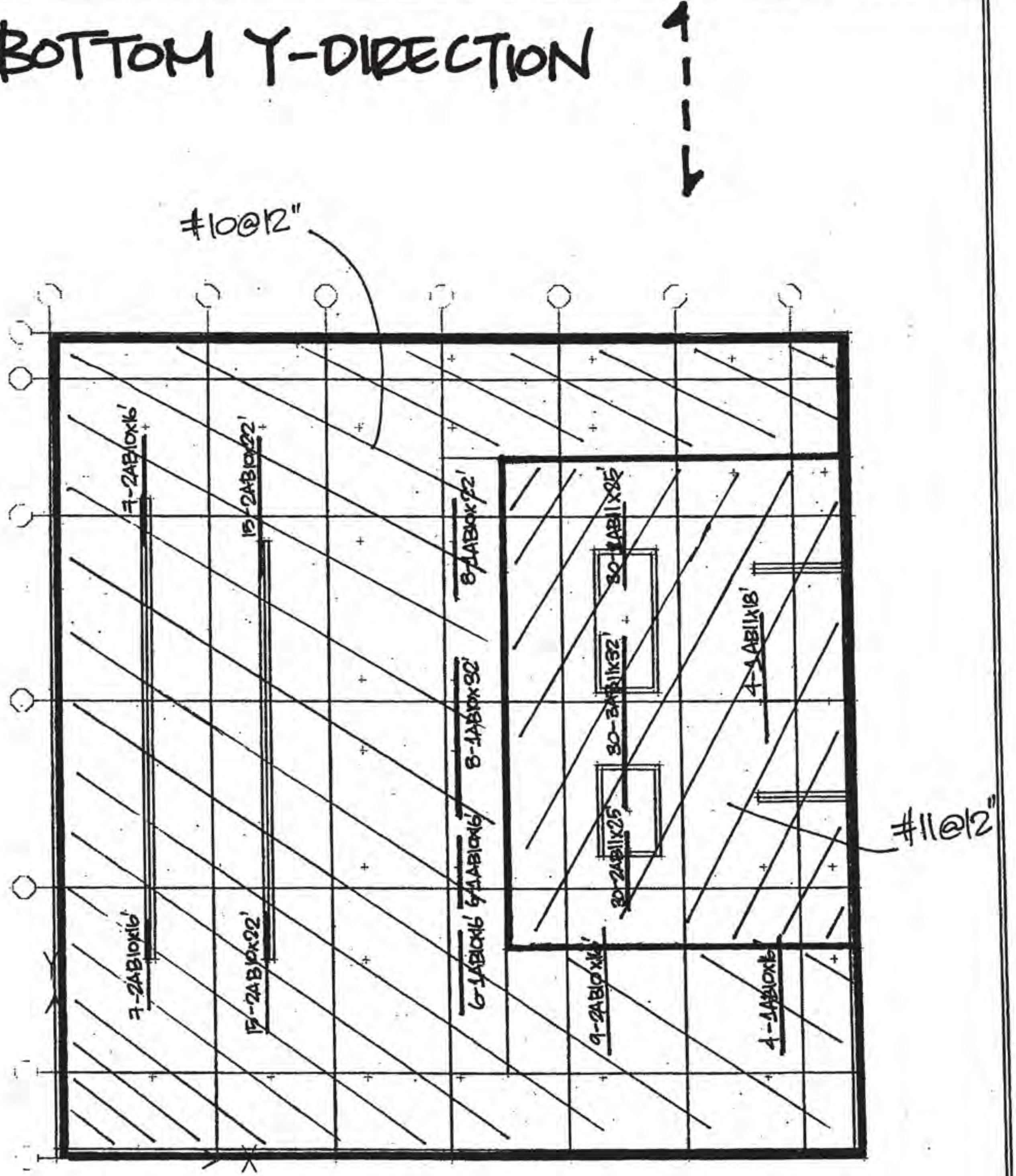


6.3-17

SAFE FLEXURAL REINFORCEMENT

301 Mission Street - Podium Foundation Mat
Foundation Mat with Tie-Downs @ 60 k/in

BOTTOM Y-DIRECTION



Ld = 81" for #10 bottom bars
Ld = 90" for #11 bottom bars

6.3-18

SECTION 7 – MID-RISE PERIMETER BASEMENT WALLS

7.1 North, East, and South Perimeter Wall

7.1 North, East, and South Perimeter Wall

The north, east, and south perimeter walls are the same in geometry and extend from the ground floor down to level B5. The walls are 51'-9" high and braced at each basement level slab every 9'-0", with the top portion between level B2 and the ground floor un-braced 15'-9". The walls are 14" thick from the ground level to B2 and 18" thick from levels B2-B5.

One wall representing the north, east, and south walls is modeled and analyzed using the computational program, RISA. Loads applied to the wall include the permanent and seismic soil pressure along the height of the wall. A traffic surcharge is also applied along the top 10 feet of the wall. The wall is assumed to be fixed at the base (level B5) and pinned at each level and at the top (B4-ground floor).

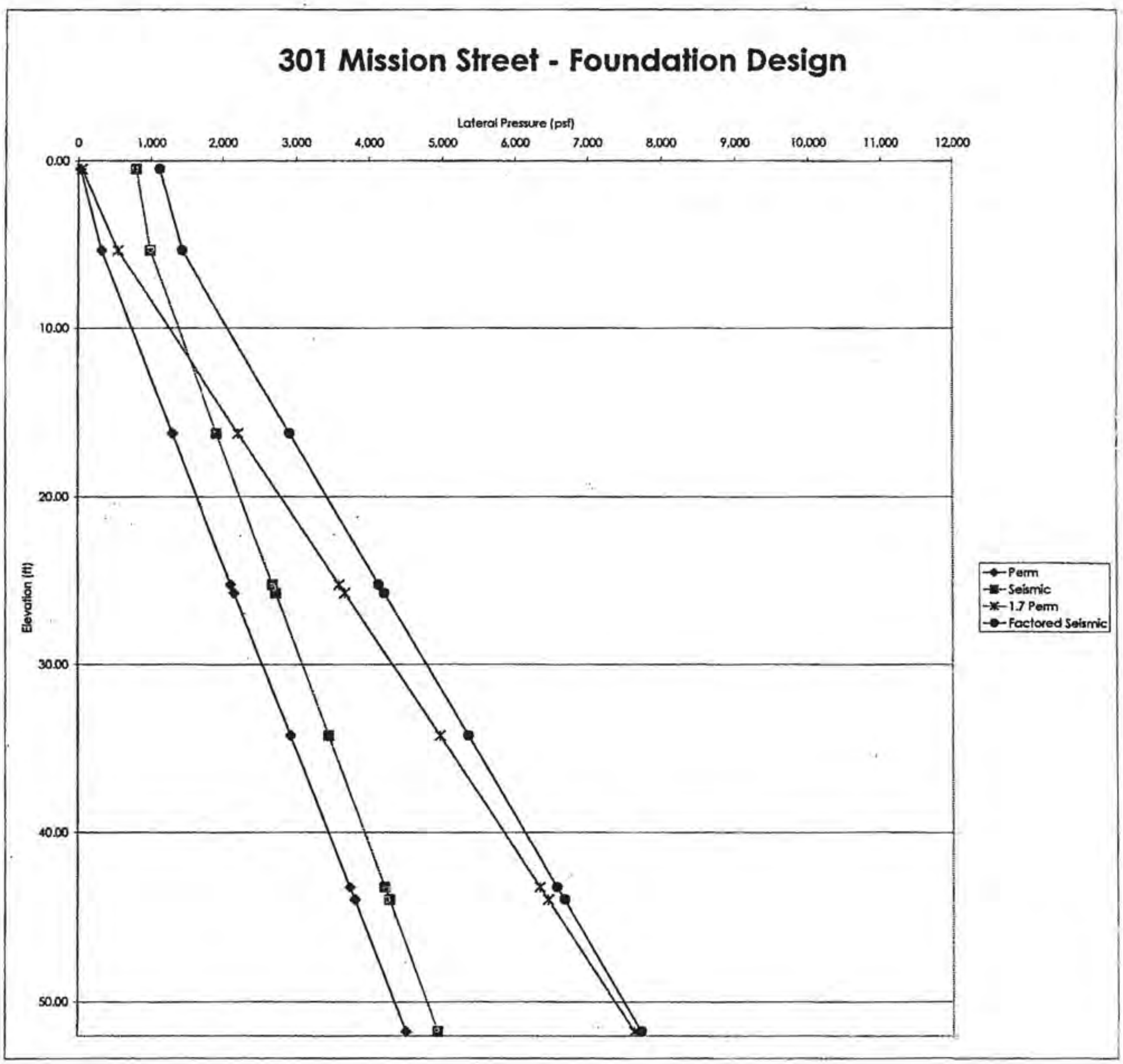
The shear in the wall due to the out-of-plane loads is checked assuming the concrete shear capacity is sufficient to take applied shear. Horizontal shear reinforcement is required for resisting the in-plane seismic loads along the wall. The required vertical flexural reinforcement is designed for both the interior and soil faces based on the maximum moments obtained from the RISA analysis. The wall has also been checked at the four large slab openings at the corners on the mid-rise.

Lateral Earth Pressure Restrained Wall Condition
Ground Elev. = 0'-0", Design Ground Water Elev. = -5.3

	Static	Seismic	
Above -5.36	60	40	15H
Below -5.36	90	85	15H

Negative Elevation (ft)	Perm Pressure (psf)	Force (k)	1.7 Perm Pressure (psf)
0.50	30	854	51
5.36	322	8,839	547
16.25	1,302	15,360	2,213
25.25	2,112	1,067	3,590
25.75	2,157	21,583	3,666
34.25	2,922	29,940	4,967
43.25	3,732	2,824	6,344
44.00	3,799	32,147	6,459
51.75	4,497		7,644
		112,615	

Negative Elevation (ft)	Seismic Soil (psf)	Seismic Incremt (psf)	Seismic Pressure (psf)	1.6 Soil + 1.4 Seismic Force (k)	sure (psf)
0.5	20	776	796	4,342	1,119
5.36	214	776	991	15,828	1,430
16.25	1,140	776	1,916	20,689	2,911
25.25	1,905	776	2,681	1,351	4,135
25.75	1,948	776	2,724	26,223	4,203
34.25	2,670	776	3,446	34,459	5,359
43.25	3,435	776	4,211	3,182	6,583
44.00	3,499	776	4,275	35,684	6,685
51.75	4,158	776	4,934		7,739
				141,760	



7.1-2

Foundation Wall Design Summary

Podium Foundation Walls

Foundation elevation per drawings 11/03/04
Lateral soil pressure per geotech report dated 1/13/2005
RISA model dated 1/27/2005 - Pinned at Top, Fixed at Base

DEMAND

Design Shear (k)

Grd		Perm	Seismic
B1	15'-9"	11.0	17.4
B2	9'-0"	11.1	15.5
B3	9'-0"	15.3	17.4
B4	9'-0"	17.5	18.1
B5	9'-0"	23.9	24.4

Design Moment (k-ft)

M+: Steel on Interior Face

Grd		Perm	Seismic
B1	15'-9"	19.0	35.3
B2	9'-0"	7.0	7.1
B3	9'-0"	17.6	21.7
B4	9'-0"	19.4	19.9
B5	9'-0"	21.1	21.6

M-: Steel on Soil Face

Grd		Perm	Seismic
B1	15'-9"	24.4	39.2
B2	9'-0"	24.6	40.4
B3	9'-0"	26.3	29.0
B4	9'-0"	27.6	28.9
B5	9'-0"	44.8	45.7

DESIGN FORCES

Grd		Shear	M+ Interior	M- Soil
B1	15'-9"	17.4	35.3	39.2
B2	9'-0"	15.5	7.1	40.4
B3	9'-0"	17.4	21.7	29.0
B4	9'-0"	18.1	19.9	28.9
B5	9'-0"	24.2	21.6	45.7

WALL DESIGN

f'c = 5 ksi

Grd			M+ Interior	M- Soil
B1	15'-9"	T = 14"	#7 @9"	#8 @9"
B2	9'-0"	T = 14"	#7 @9"	#8 @9"
B3	9'-0"	T = 18"	#5 @9"	#7 @9"
B4	9'-0"	T = 18"	#5 @9"	#7 @9"
B5	9'-0"	T = 18"	#5 @9"	#7 @9"

CAPACITY

Grd		Shear	M+ Interior	M- Soil
B1	15'-9"	18.4	44.4	46.8
B2	9'-0"	18.4	44.4	46.8
B3	9'-0"	24.2	31.1	50.7
B4	9'-0"	24.2	31.1	50.7
B5	9'-0"	24.2	31.1	50.7

DEMAND-CAPACITY RATIOS

Grd		Shear	M+ Interior	M- Soil
B1	15'-9"	0.95	0.79	0.84
B2	9'-0"	0.84	0.16	0.86
B3	9'-0"	0.72	0.70	0.57
B4	9'-0"	0.75	0.64	0.57
B5	9'-0"	1.00	0.70	0.90

7.1-3

Foundation Wall Design

CONCRETE SHEAR CAPACITY, k per ft

Concrete to take all shear (no shear reinf.)
Assume d = T - 1.25" at inside face for shear

	Concrete Strength			
T (in)	3 ksi	4ksi	5 ksi	6 ksi
6	5.3	6.1	6.9	7.5
8	7.5	8.7	9.7	10.7
10	9.8	11.3	12.6	13.8
12	12.0	13.9	15.5	17.0
14	14.2	16.5	**18.4**	20.1
16	16.5	19.0	21.3	23.3
18	18.7	21.6	**24.2**	26.5
20	21.0	24.2	27.0	29.6
22	23.2	26.8	29.9	32.8
24	25.4	29.4	32.8	35.9

MINIMUM HORIZONTAL STEEL REQUIREMENT
[ACI 14.3.3]

T (in)	Total As,min
6	0.18
8	0.24
10	0.30
12	0.36
14	**0.42**
16	0.48
18	**0.54**
20	0.60
22	0.66
24	0.72

Area of Steel for Each Face

Spg (in)	#4	#5	#6	#7	#8
6	0.40	0.62	0.88	1.20	1.58
8	0.30	0.47	0.66	0.90	1.19
10	0.24	0.37	0.53	0.72	0.95
12	0.20	0.31	0.44	0.60	0.79
14	0.17	0.27	0.38	0.51	0.68
16	0.15	0.23	0.33	0.45	0.59
18	0.13	0.21	0.29	0.40	0.53
20	0.12	0.19	0.26	0.36	0.47
22	0.11	0.17	0.24	0.33	0.43
24	0.10	0.16	0.22	0.30	0.40

WALL FLEXURAL CAPACITY, k-ft per ft

For M+: Assume d = T - 0.75" - dia/2 (verts outside of horiz.)

Wall T = **14 in** f'c = **5 ksi**

Spg (in)	#4	#5	#6	#7	#8	#9	#10	#11
6	22.98	35.08	48.94	65.38	84.04	126.17	153.55	179.90
7	19.75	30.19	42.20	56.50	72.85	110.17	134.88	159.13
8	17.31	26.50	37.09	49.75	64.27	97.73	120.17	142.48
9	15.41	23.61	33.08	**44.43**	57.50	87.79	108.30	128.88
10	13.89	21.29	29.85	40.14	52.01	79.67	98.54	117.60
11	12.64	19.39	27.20	36.60	47.48	72.92	90.37	108.11
12	11.59	17.79	24.98	33.64	43.67	67.22	83.45	100.02
13	10.71	16.44	23.09	31.12	40.43	62.35	77.50	93.04
14	9.95	15.28	21.47	28.95	37.64	58.13	72.34	86.96
15	9.29	14.28	20.07	27.07	35.20	54.44	67.83	81.62
16	8.72	13.39	18.83	25.41	33.07	51.19	63.84	76.90
17	8.25	12.61	17.74	23.94	31.17	48.31	60.29	72.69
18	7.75	11.92	16.77	22.64	29.48	45.73	57.11	68.91

Area of Steel for Each Face

Spg (in)	#4	#5	#6	#7	#8	#9	#10	#11
6	0.40	0.62	0.88	1.20	1.58	2.00	2.54	3.12
7	0.34	0.53	0.75	1.03	1.35	1.71	2.18	2.67
8	0.30	0.47	0.66	0.90	1.19	1.50	1.91	2.34
9	0.27	0.41	0.59	0.80	1.05	1.33	1.69	2.08
10	0.24	0.37	0.53	0.72	0.95	1.20	1.52	1.87
11	0.22	0.34	0.48	0.65	0.86	1.09	1.39	1.70
12	0.20	0.31	0.44	0.60	0.79	1.00	1.27	1.56
13	0.18	0.29	0.41	0.55	0.73	0.92	1.17	1.44
14	0.17	0.27	0.38	0.51	0.68	0.86	1.09	1.34
15	0.16	0.25	0.35	0.48	0.63	0.80	1.02	1.25
16	0.15	0.23	0.33	0.45	0.59	0.75	0.95	1.17
17	0.14	0.22	0.31	0.42	0.56	0.71	0.90	1.10
18	0.13	0.21	0.29	0.40	0.53	0.67	0.85	1.04
dia (in)	0.500	0.625	0.750	0.875	1.000	1.128	1.270	1.410
fy (ksi)	60	60	60	60	60	75	75	75
Total As,min [ACI 10.5.1]	0.53	0.53	0.53	0.52	0.52	0.41	0.41	0.41

For M-: Assume d = T - 3" - dia/2 (verts outside of horiz.)

Wall T = **14 in** f'c = **5 ksi**

Spg (in)	#4	#5	#6	#7	#8	#9	#10	#11
6	18.93	28.80	40.03	53.23	68.05	100.86	121.41	140.42
7	16.27	24.81	34.56	46.09	59.14	88.48	107.33	125.29
8	14.27	21.79	30.40	40.63	52.27	78.75	96.06	112.86
9	12.71	19.43	27.14	36.33	**46.83**	70.92	86.87	102.56
10	11.46	17.52	24.51	32.85	42.41	64.49	79.25	93.91
11	10.43	15.96	22.34	29.98	38.75	59.12	72.84	86.57
12	9.57	14.65	20.53	27.57	35.68	54.57	67.37	80.27
13	8.88	13.55	18.98	25.51	33.05	50.66	62.67	74.81
14	8.22	12.59	17.66	23.74	30.78	47.28	58.57	70.04
15	7.67	11.76	16.50	22.21	28.80	44.31	54.97	65.83
16	7.20	11.04	15.49	20.85	27.07	41.70	51.78	62.09
17	6.78	10.40	14.59	19.66	25.53	39.38	48.94	58.75
18	6.40	9.83	13.80	18.59	24.15	37.30	46.40	55.75

$$V: \quad \frac{V_u}{\phi V_c} = \frac{17.4^k}{18.4^k} = 0.95 \qquad (T = 14")$$

$$M+: \quad \frac{M_u}{\phi M_n} = \frac{35.3}{44.43} = 0.79 \qquad (\#7 @ 9" \text{ o.c.})$$

$$M-: \quad \frac{M_u}{\phi M_n} = \frac{40.4}{46.83} = 0.86 \qquad (\#8 @ 9" \text{ o.c.})$$

7.1-4

Podium Foundation Wall (Grd - B2)

Foundation Wall Design

CONCRETE SHEAR CAPACITY, k per ft

Concrete to take all shear (no shear reinf.)
Assume d = T - 1.25" at inside face for shear

T (in)	Concrete Strength 3 ksi	4ksi	5 ksi	6 ksi
6	5.3	6.1	6.9	7.5
8	7.5	8.7	9.7	10.7
10	9.8	11.3	12.6	13.8
12	12.0	13.9	15.5	17.0
14	14.2	16.5	18.4	20.1
16	16.5	19.0	21.3	23.3
18	18.7	21.6	24.2	26.5
20	21.0	24.2	27.0	29.6
22	23.2	26.8	29.9	32.8
24	25.4	29.4	32.8	35.9

MINIMUM HORIZONTAL STEEL REQUIREMENT
[ACI 14.3.3]

T (in)	Total As,min
6	0.18
8	0.24
10	0.30
12	0.36
14	0.42
16	0.48
18	0.54
20	0.60
22	0.66
24	0.72

Area of Steel for Each Face

Spg (in)	#4	#5	#6	#7	#8
6	0.40	0.62	0.88	1.20	1.58
8	0.30	0.47	0.66	0.90	1.19
10	0.24	0.37	0.53	0.72	0.95
12	0.20	0.31	0.44	0.60	0.79
14	0.17	0.27	0.38	0.51	0.68
16	0.15	0.23	0.33	0.45	0.59
18	0.13	0.21	0.29	0.40	0.53
20	0.12	0.19	0.26	0.36	0.47
22	0.11	0.17	0.24	0.33	0.43
24	0.10	0.16	0.22	0.30	0.40

WALL FLEXURAL CAPACITY, k-ft per ft

For M+: Assume d = T - 0.75" - dia/2 (verts outside of horiz.)

Wall T = 18 in   f'c = 5 ksi

Spg (in)	#4	#5	#6	#7	#8	#9	#10	#11
6	30.18	46.24	64.78	86.98	112.48	171.17	210.70	250.10
7	[illegible]	39.76	55.77	75.02	97.22	148.75	183.87	219.30
8	[illegible]	34.87	48.97	65.95	85.60	131.48	163.03	195.13
9	[illegible]	31.05	43.64	58.83	76.46	117.79	146.40	175.68
10	[illegible]	27.99	39.36	53.10	69.08	106.67	132.83	159.72
11	[illegible]	[illegible]	35.84	48.39	62.99	97.47	121.54	146.40
12	[illegible]	[illegible]	32.90	44.44	57.89	89.72	112.02	135.12
13	[illegible]	[illegible]	30.41	41.09	53.56	83.11	103.88	125.44
14	[illegible]	[illegible]	28.26	38.21	49.83	77.41	96.84	117.05
15	[illegible]	[illegible]	26.40	35.71	46.58	72.44	90.69	109.70
16	[illegible]	[illegible]	[illegible]	33.51	43.73	68.07	85.27	103.22
17	[illegible]	[illegible]	[illegible]	31.57	41.21	64.19	80.46	97.47
18	[illegible]	[illegible]	[illegible]	29.84	38.96	60.73	76.16	92.31

Area of Steel for Each Face

Spg (in)	#4	#5	#6	#7	#8	#9	#10	#11
6	0.40	0.62	0.88	1.20	1.58	2.00	2.54	3.12
7	[illegible]	0.53	0.75	1.03	1.35	1.71	2.18	2.67
8	[illegible]	0.47	0.66	0.90	1.19	1.50	1.91	2.34
9	[illegible]	0.41	0.59	0.80	1.05	1.33	1.69	2.08
10	[illegible]	0.37	0.53	0.72	0.95	1.20	1.52	1.87
11	[illegible]	[illegible]	0.48	0.65	0.86	1.09	1.39	1.70
12	[illegible]	[illegible]	0.44	0.60	0.79	1.00	1.27	1.56
13	[illegible]	[illegible]	0.41	0.55	0.73	0.92	1.17	1.44
14	[illegible]	[illegible]	0.38	0.51	0.68	0.86	1.09	1.34
15	[illegible]	[illegible]	0.35	0.48	0.63	0.80	1.02	1.25
16	[illegible]	[illegible]	[illegible]	0.45	0.59	0.75	0.95	1.17
17	[illegible]	[illegible]	[illegible]	0.42	0.56	0.71	0.90	1.10
18	[illegible]	[illegible]	[illegible]	0.40	0.53	0.67	0.85	1.04
dia (in)	0.500	0.625	0.750	0.875	1.000	1.128	1.270	1.410
fy (ksi)	60	60	60	60	60	75	75	75
Total As,min [ACI 10.5.1]	0.70	0.70	0.69	0.69	0.69	0.55	0.55	0.54

For M-: Assume d = T - 3" - dia/2 (verts outside of horiz.)

Wall T = 18 in   f'c = 5 ksi

Spg (in)	#4	#5	#6	#7	#8	#9	#10	#11
6	26.13	39.96	55.87	74.83	96.49	145.86	178.56	210.62
7	[illegible]	34.38	48.14	64.60	83.51	127.05	156.32	185.46
8	[illegible]	30.16	42.28	56.83	73.60	112.50	138.92	165.51
9	[illegible]	26.87	37.70	50.73	65.79	100.92	124.97	149.36
10	[illegible]	24.22	34.01	45.81	59.48	91.49	113.54	136.03
11	[illegible]	[illegible]	30.98	41.76	54.27	83.66	104.01	124.86
12	[illegible]	[illegible]	28.45	38.37	49.90	77.07	95.95	115.37
13	[illegible]	[illegible]	26.29	35.48	46.17	71.43	89.04	107.21
14	[illegible]	[illegible]	24.44	33.00	42.97	66.56	83.06	100.12
15	[illegible]	[illegible]	22.84	30.85	40.18	62.31	77.83	93.91
16	[illegible]	[illegible]	[illegible]	28.95	37.73	58.58	73.21	88.42
17	[illegible]	[illegible]	[illegible]	27.28	35.56	55.26	69.11	83.53
18	[illegible]	[illegible]	[illegible]	25.79	33.63	52.30	65.45	79.15

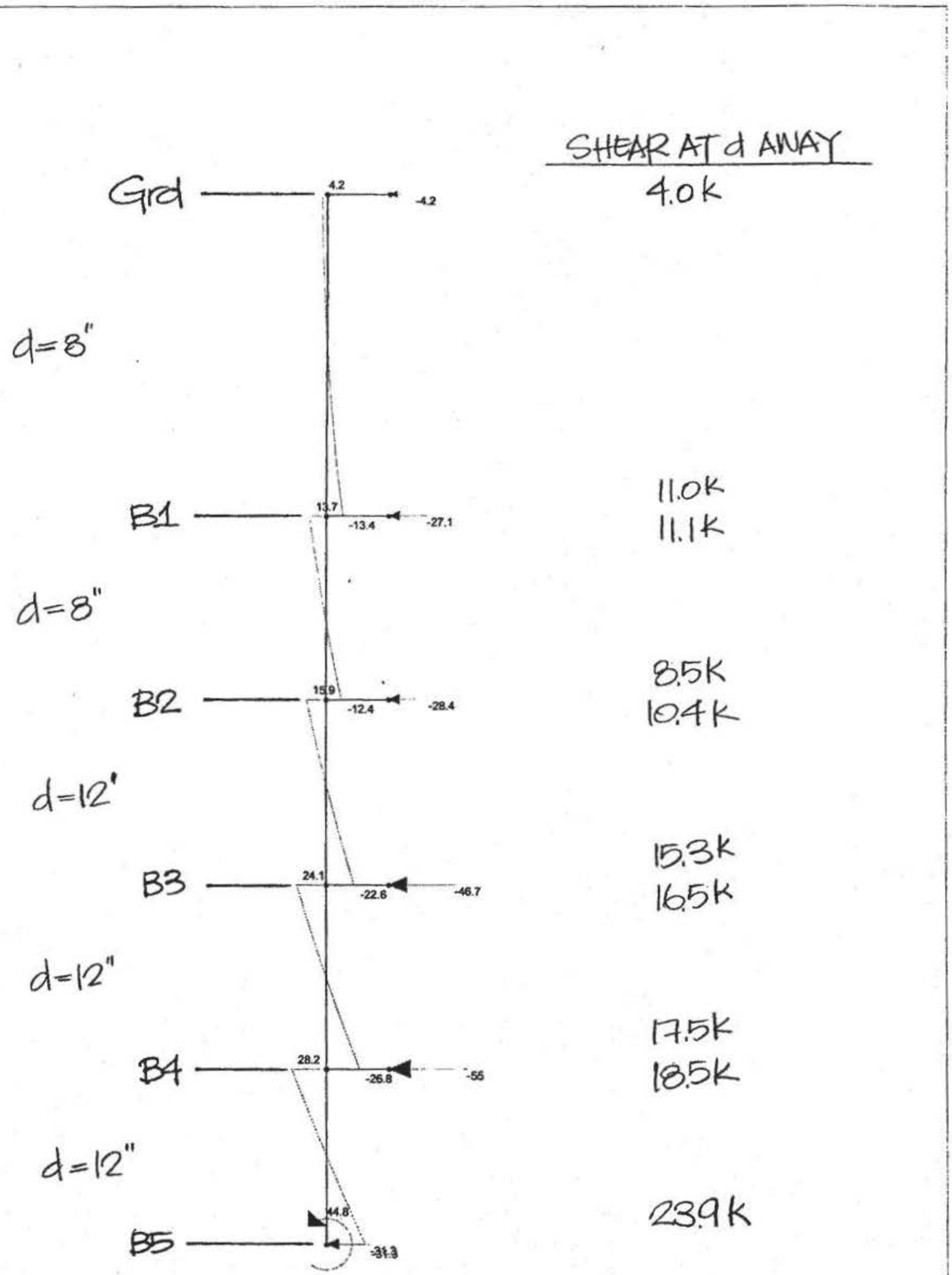
$V: \frac{V_u}{\phi V_c} = \frac{24.19^k}{24.2^k} = 0.99^+$ (T = 18")

$M+: \frac{M_u}{\phi M_n} = \frac{21.7^{k\text{-}'}}{31.05^{k\text{-}'}} = 0.70$ (#5 @ 9" o.c.)

$M-: \frac{M_u}{\phi M_n} = \frac{45.7^{k\text{-}'}}{50.73^{k\text{-}'}} = 0.90$ (#7 @ 9" o.c.)

S-1.2

Podium Foundation Wall (B2 - B5)



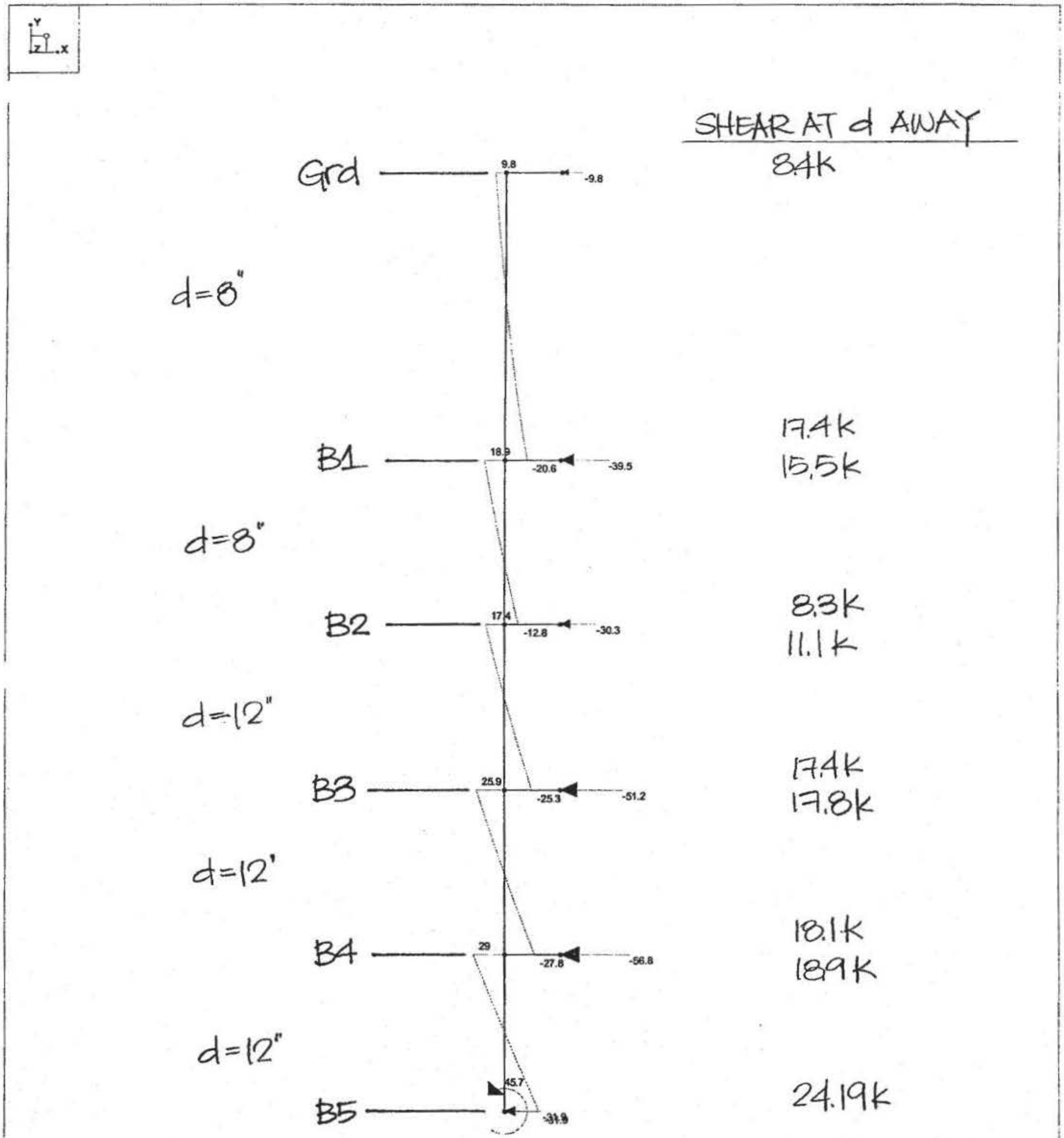
Results for LC 5, 1.7 Perm
Member y Shear Forces (k)
Reaction units are k and k-ft

1.7 Perm Soil + 1.7 Traffic Surcharge

DeSimone Consulting Eng...	301 Mission Street Podium Foundation Walls	
ML		Mar 8, 2005 at 2:21 PM
4069		4069-20050127-MKL-B5-Fdn-Wall....

7.1-6

DODSONNOC00000351



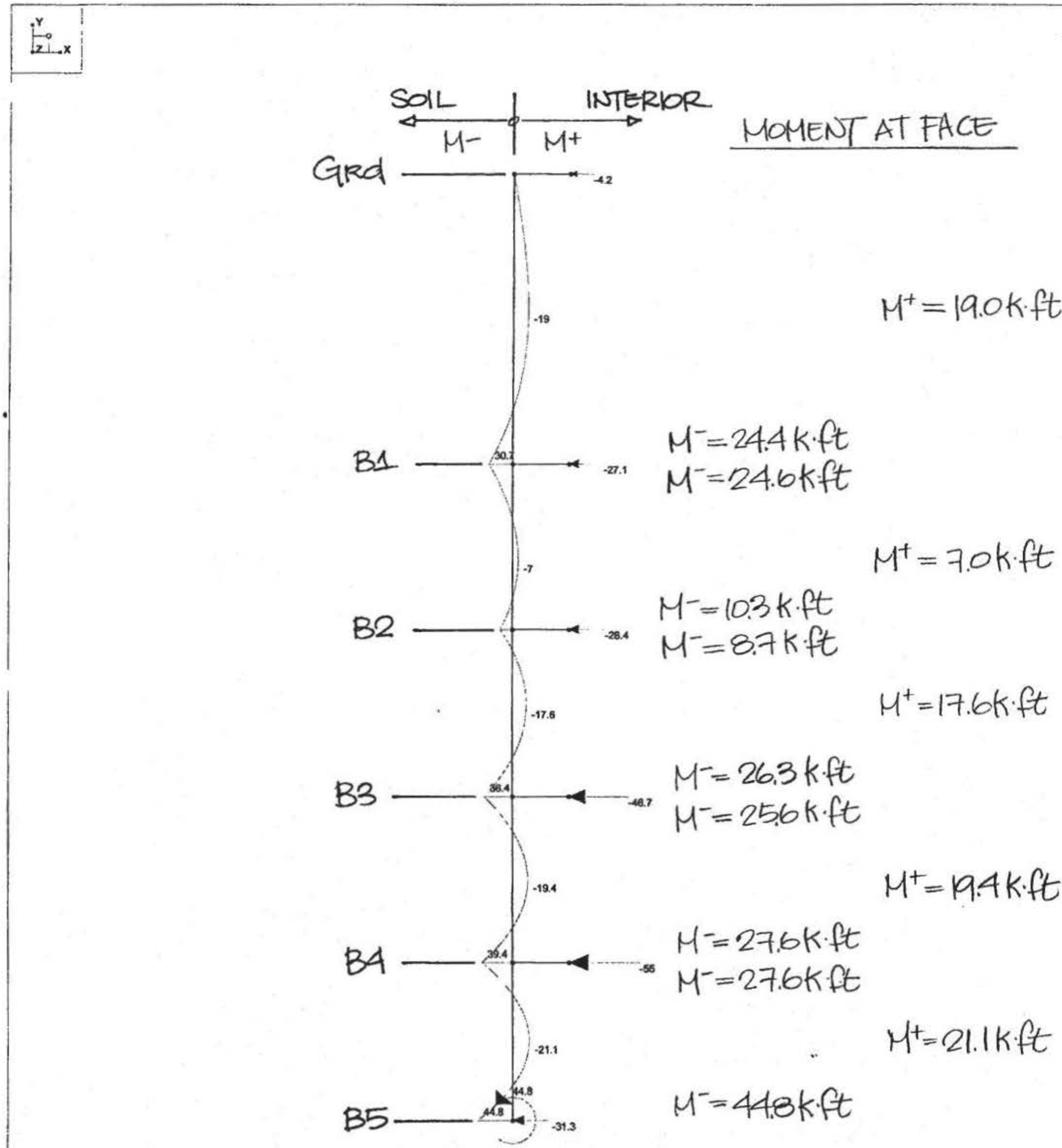
Results for LC 6, Seismic Combo
Member y Shear Forces (k)
Reaction units are k and k-ft

1.6Seismic Soil + 1.4Seismic Increment + 1.0Traffic Surcharge

DeSimone Consulting Eng...	301 Mission Street Podium Foundation Walls	
ML		Mar 8, 2005 at 2:30 PM
4069		4069-20050127-MKL-B5-Fdn-Wall....

7.1-7

DODSONNOC00000352



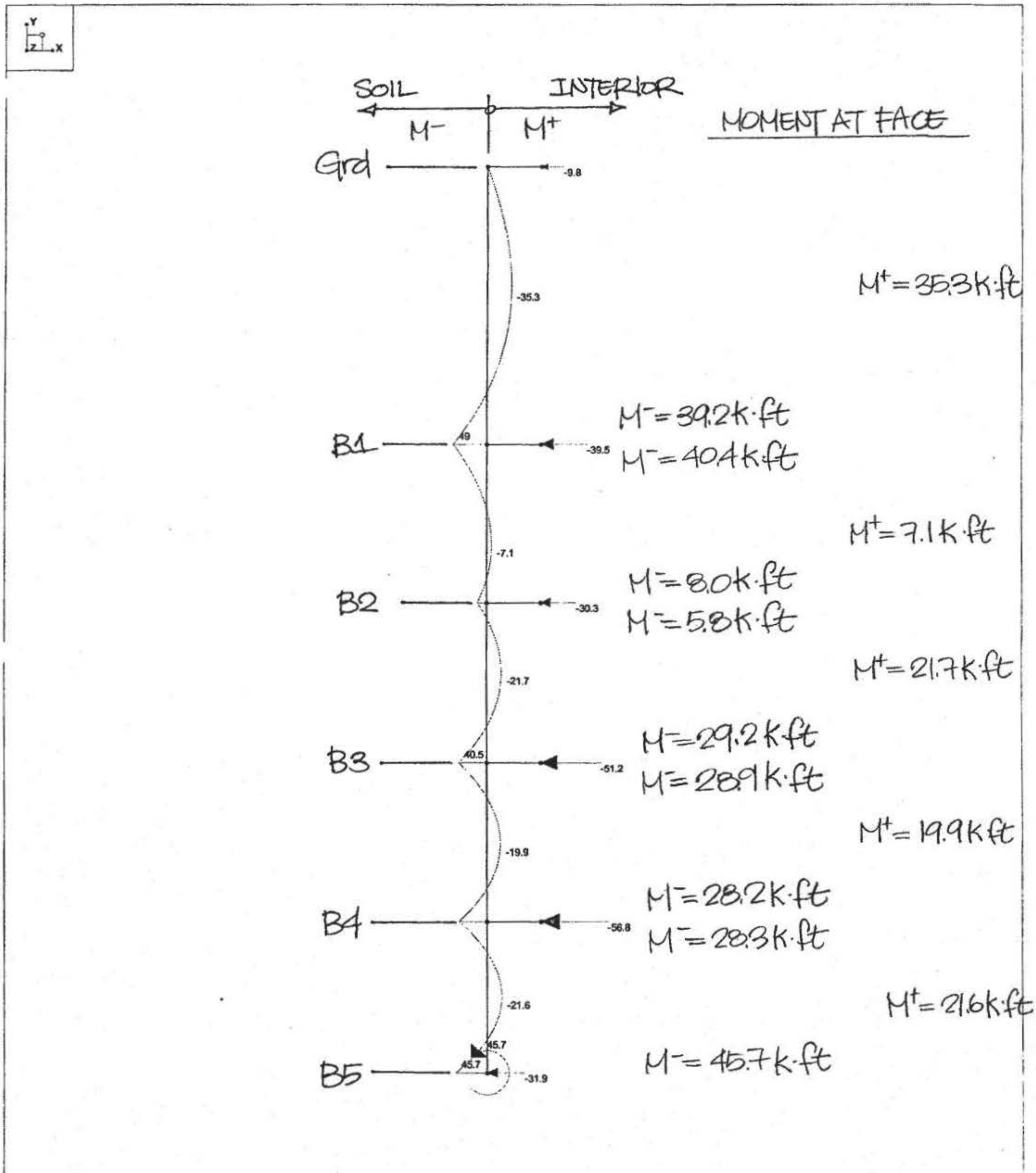
Results for LC 5, 1.7 Perm
Member z Bending Moments (k-ft)
Reaction units are k and k-ft

1.7 Perm Soil + 1.7 Traffic Surcharge

DeSimone Consulting Eng...	301 Mission Street Podium Foundation Walls	
ML		Mar 8, 2005 at 2:29 PM
4069		4069-20050127-MKL-B5-Fdn-Wall....

701-8

DODSONNOC00000353



Results for LC 6, Seismic Combo
Member z Bending Moments (k-ft)
Reaction units are k and k-ft

1.6 Seismic Soil + 1.4 Seismic Increment + 1.0 Traffic Surcharge

DeSimone Consulting Eng...	301 Mission Street Podium Foundation Walls	
ML		Mar 8, 2005 at 2:30 PM
4069		4069-20050127-MKL-B5-Fdn-Wall....

701-9

DODSONNOC00000354

DESIMONE

Project 301 MISSION STREET

Project No. 4069

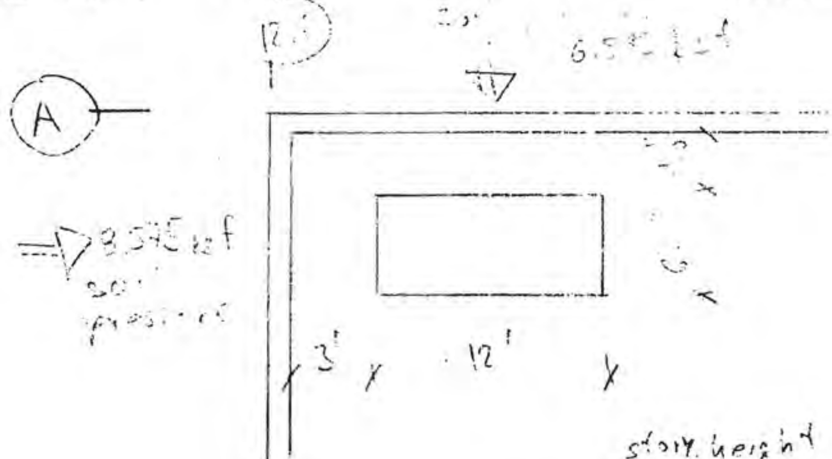
Item

Page ____ Of ____

Date / /2005

By J.P. Ch'kd ____

Check slab around opening @ B4



story height

6583 ksf × 9' = 59.25 k/f

12'-0" 356^k M = 311 k-ft (demand)

77.2 k/f

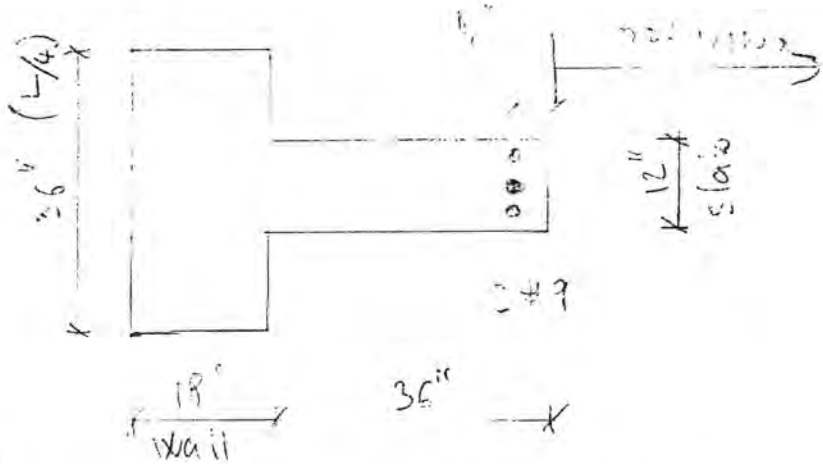
R = 257^k

7.1-10

DESIMONE

Project	301 MISSION STREET	**Page** ____ **Of** ____	
Project No.	4069	**Date**	/ /2005
Item		**By** J.P.	**Ch'kd**

Section [illegible] north [illegible]



M_u = [illegible]

V_u = [illegible]

Moment

3#9 : A_s = 3.0 in² ; f_y = 75 ksi , T = 225[k] , a = [illegible]" , γ = 50.[illegible]"

[illegible] = 0.9 × 225[k] × 50.75"/12" = [illegible] k-ft > [illegible] k-ft (O.K.)

Shear

V_c = $2\sqrt{f_c'} \times b_w \times d$ = $2\sqrt{[illegible]}$ × [illegible]" × 50 = [illegible]k

$V_{s,req}$ = [illegible]/0.85 − 34.9[k] = [illegible]k

V_s : #5 A_v = 0.31 in² , $A_{v,4legs}$ = 1.24 in²

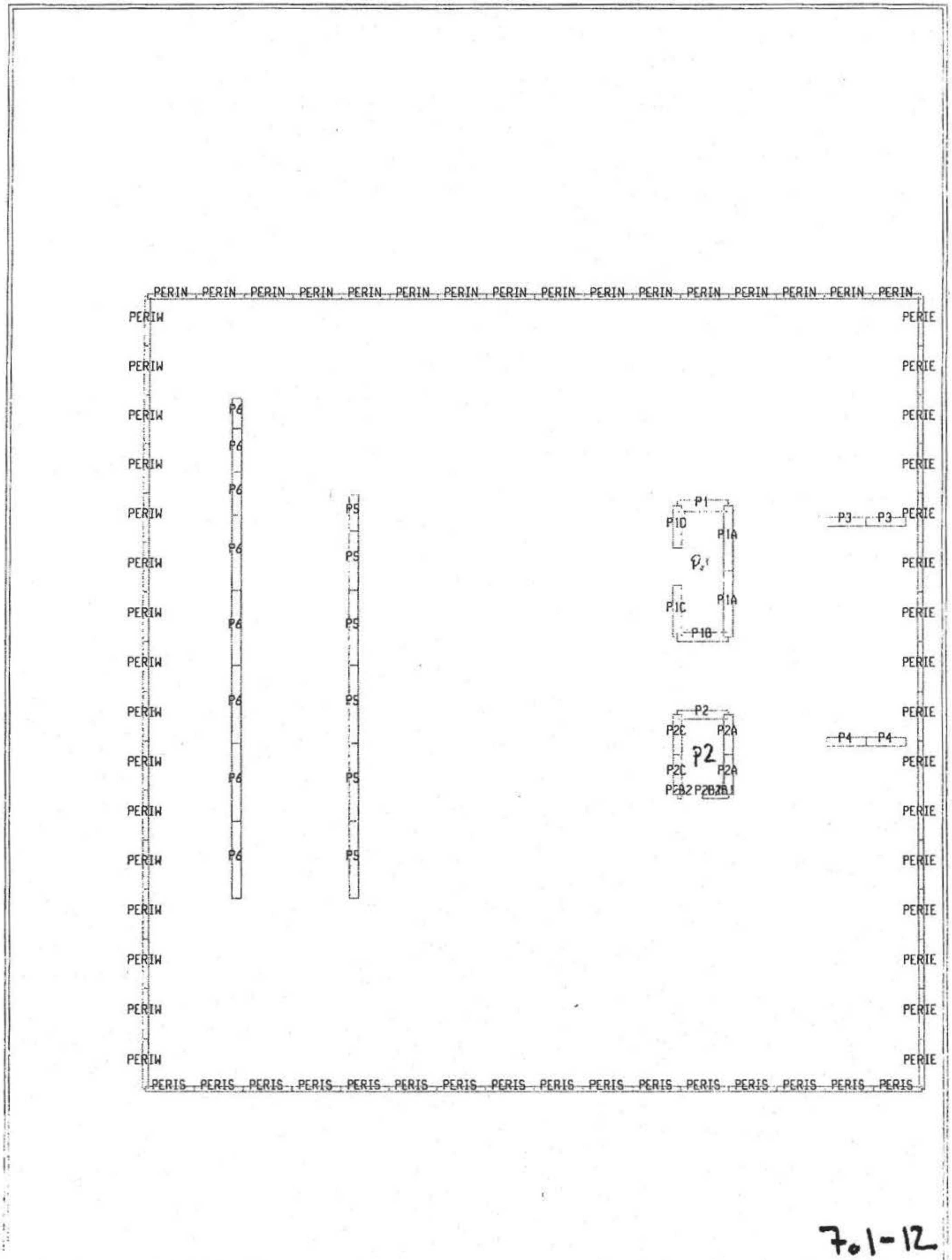
$s = \frac{A_v f_y d}{V_{s,req}}$ = 1.24 in × 60 ksi × 50"/33[illegible] = [illegible] in

Use 6" spacing for #5 w/4 legs

ϕV_n = ([illegible] + 1.24 in² × 60 ksi × $\frac{50}{6"}$) × 0.85 = 599[k] > 356[k] (O.K.)

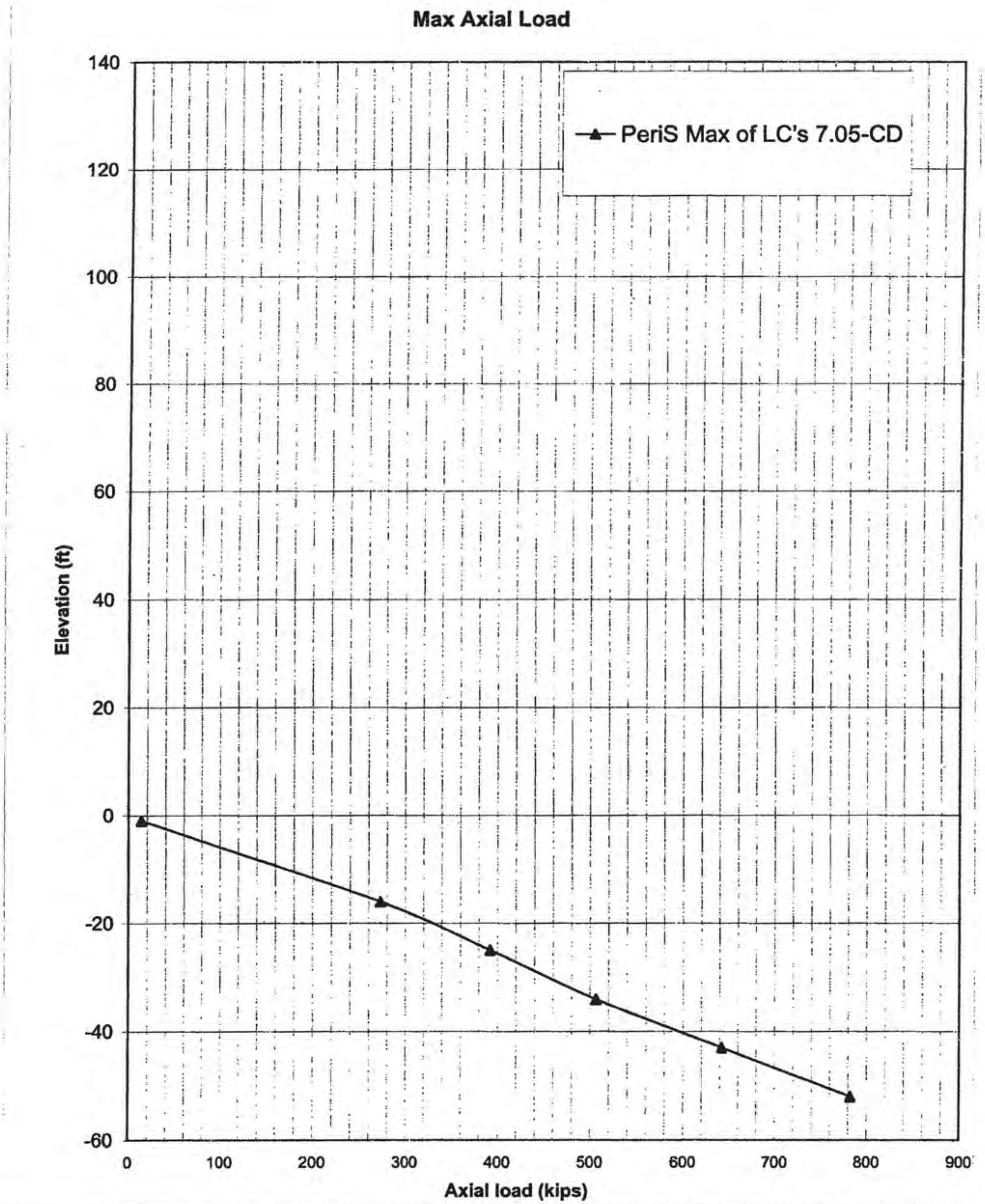
7.1-11

ETABS

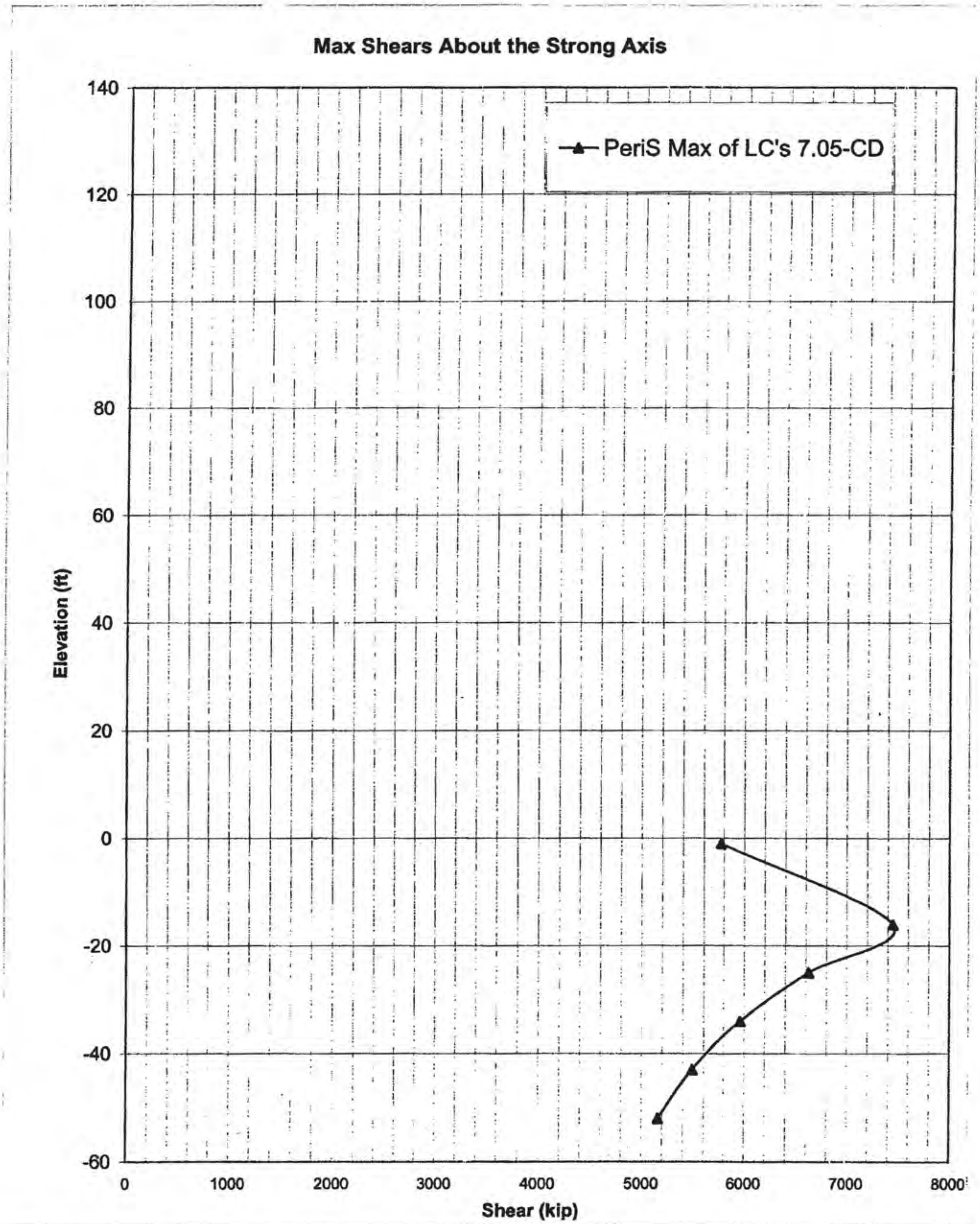


ETABS v8.5.0 - File: 4069-20050413-7.05a-CD - April 14,2005 17:11
Plan View - B1 - Elevation -192 - Kip-in Units

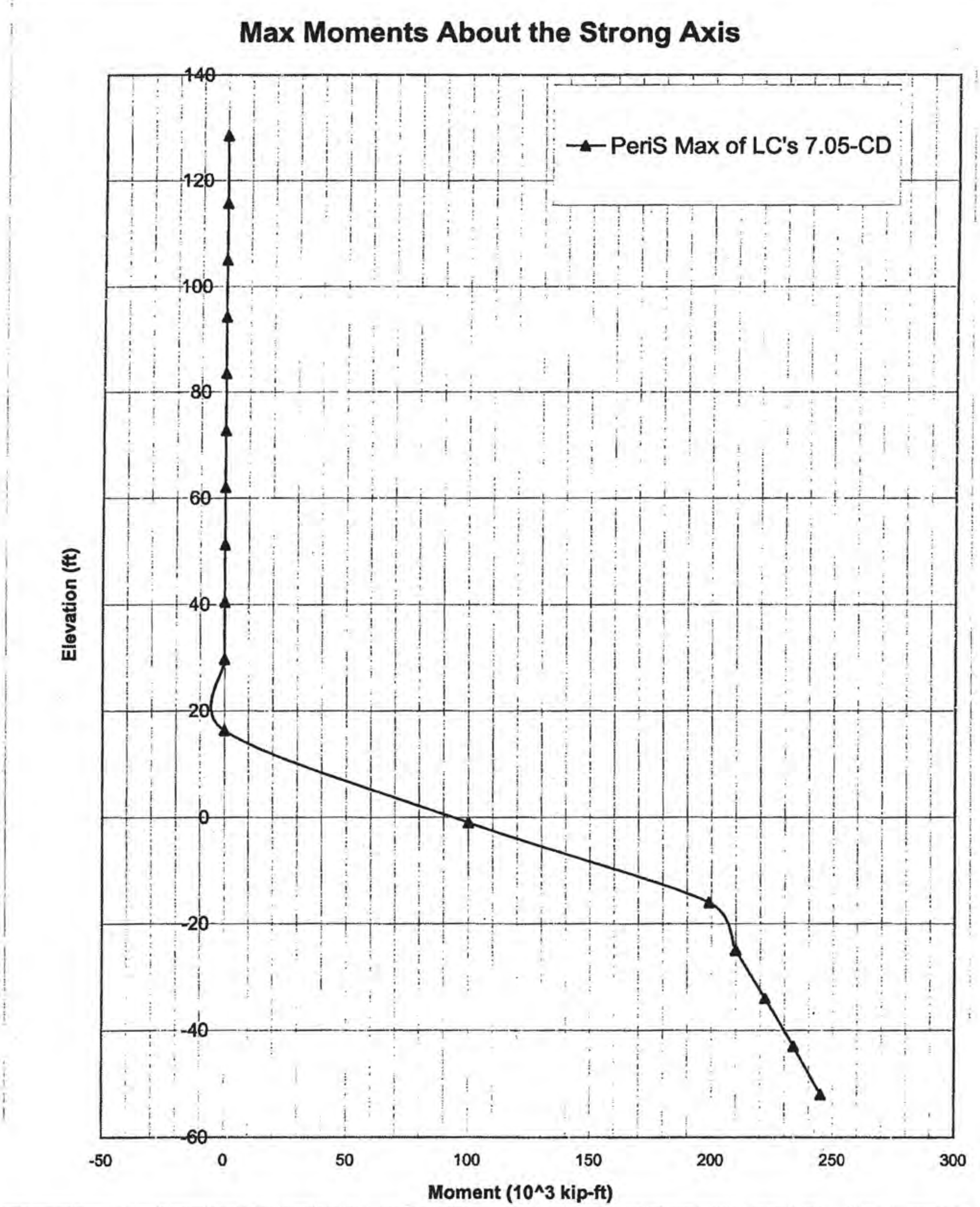
Print Date 4/22/2005



7.1-13


Max Shears About the Strong Axis
PeriS Max of LC's 7.05-CD
Elevation (ft)
140
120
100
80
60
40
20
0
-20
-40
-60
0
1000
2000
3000
4000
5000
6000
7000
8000
Shear (kip)

7.1-14


Max Moments About the Strong Axis
PeriS Max of LC's 7.05-CD
Elevation (ft)
140
120
100
80
60
40
20
0
-20
-40
-60
-50
0
50
100
150
200
250
300
Moment (10^3 kip-ft)

7.1-15

Print Date 4/22/2005

dlf	dlf	dlf
1.0	1.0	1.0

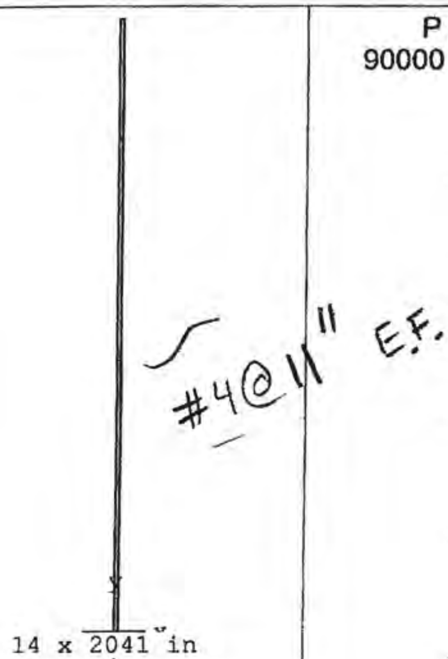
Unit Wt.	0.150	kcf
Min trib area from group	Varies	ft^2 (For Tension)
Max trib area from group	Varies	ft^2 (For Compression)

PeriS

Floor	Usage	Flr. Ht. ft.	Elevation ft.	Width in	Length in	Min Trib A. sq. ft	Min Cum Trib A. sq. ft	Max Trib A. sq. ft	Max Cum Trib A. sq. ft	Floor DL psf	Red. LL psf	Gravity Beams Total Trib Length ft	Gravity Beams Wt. klf
13	Cap	12.75	141.3	0	0	0	0	0	0	0	0	0	0.000
12	Roof	12.83	128.6	0	0	0	0	0	0	0	0	0	0.000
11	Typ	10.75	115.8	0	0	0	0	0	0	0	0	0	0.000
10	Typ	10.75	105.0	0	0	0	0	0	0	0	0	0	0.000
9	Typ	10.75	94.3	0	0	0	0	0	0	0	0	0	0.000
8	Typ	10.75	83.5	0	0	0	0	0	0	0	0	0	0.000
7	Typ	10.75	72.8	0	0	0	0	0	0	0	0	0	0.000
6	Typ	10.75	62.0	0	0	0	0	0	0	0	0	0	0.000
5	Typ	10.75	51.3	0	0	0	0	0	0	0	0	0	0.000
4	Typ	10.75	40.5	0	0	0	0	0	0	0	0	0	0.000
3	Public	13.42	29.8	0	0	0	0	0	0	0	0	0	0.000
2	Public	17.33	16.3	14	2041	3672	3672	4079	4079	350	40	192	1.925
1	Public	15.00	-1.0	14	2041	1619	5291	1810	5889	200	100	66	1.400
B1	Parking	9.00	-16.0	18	2041	1158	6449	1307	7197	155	50	0	0.000
B2	Parking	9.00	-25.0	18	2041	1158	7608	1233	8430	155	50	0	0.000
B3	Parking	9.00	-34.0	18	2041	1158	8766	1233	9663	155	50	0	0.000
B4	Parking	9.00	-43.0	18	2041	1158	9925	1233	10895	155	50	0	0.000
Base		0.00	-52.0										

Floor	Torsion Beam Trib Length ft	Torsion Beam Wt. klf	Beam DL kips	Self Wt kips	Min Total DL kips	Min Cum DL kips	Max Total DL kips	Max Cum DL kips	Cum Reducible LL kips	LL % multiplier	Cum Red. LL kips	Min Cum Service kips	Max Cum Service kips	1.42(dlf)D+0.5L Design kips	0.9*D Design kips	1.4(dlf)D+1.7L Design kips
13			0	0	0	0	0	0	0	1.00	0	0	0	0	0	0
12	0	0	0	0	0	0	0	0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0	#DIV/0!
11	0	0	0	0	0	0	0	0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0	#DIV/0!
10	0	0	0	0	0	0	0	0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0	#DIV/0!
9	0	0	0	0	0	0	0	0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0	#DIV/0!
8	0	0	0	0	0	0	0	0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0	#DIV/0!
7	0	0	0	0	0	0	0	0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0	#DIV/0!
6	0	0	0	0	0	0	0	0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0	#DIV/0!
5	0	0	0	0	0	0	0	0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0	#DIV/0!
4	0	0	0	0	0	0	0	0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0	#DIV/0!
3	0	0	0	0	0	0	0	0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0	#DIV/0!
2	170	1.88	688	516	2489	2489	2632	2632	147	1.00	147	2636	2779	3811	2240	3935
1	170	1.88	411	446	1181	3671	1220	3852	309	1.00	309	3980	4160	5624	3304	5917
B1	0	0	0	344	524	4195	547	4399	367	0.44	160	4355	4559	6326	3775	6430
B2	0	0	0	344	524	4719	536	4934	425	0.42	179	4898	5113	7096	4247	7213
B3	0	0	0	344	524	5243	536	5470	483	0.41	198	5441	5668	7866	4718	7994
B4	0	0	0	344	524	5767	536	6005	540	0.40	216	5983	6222	8636	5190	8775
Base																
			1100	2340												

7.1-16



Code: ACI 318-95

Units: English

Run axis: About X-axis

Run option: Investigation

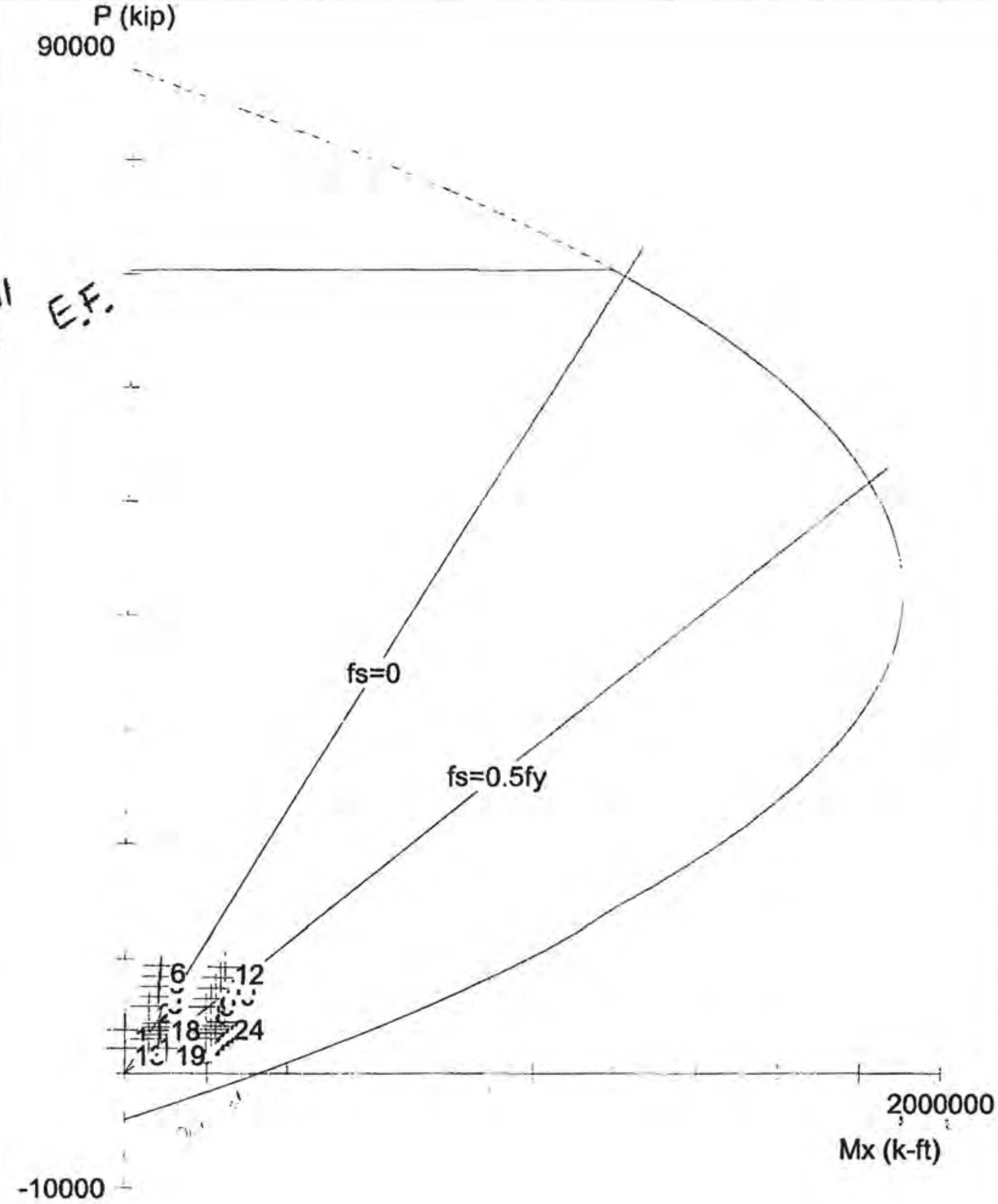
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 04/22/05

Time: 18:10:05



PCACOL V3.00 (PCA 1999) - Licensed to: Licensee name not yet specified.

File: F:\PROJECTS\4069\PCA\PODIUM\14-406~1\PERISL1.COL

Project:

Column:

Engineer:

f'c = 5 ksi	fy = 60 ksi	Ag = 28574 in^2	374 #4 bars
Ec = 4031 ksi	Es = 29000 ksi	As = 74.80 in^2	Rho = 0.26% ≤ min
fc = 4.25 ksi	e_rup = Infinity	Xo = 7.00 in	Ix = 9.91918e+009 in^4
e_u = 0.003 in/in		Yo = 1020.50 in	Iy = 466709 in^4
Beta1 = 0.8		Clear spacing = 10.42 in	Clear cover = N/A

7.1-17

Confinement: Tied phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.7

DODSONNOC00000362

General Information:
====================
File Name: F:\PROJECTS\4069\PCA\PODIUM\14-406~1\PERISL1.COL
Project:
Column: Engineer:
Code: ACI 318-95 Units: English

Run Option: Investigation Slenderness: Not considered
Run Axis: X-axis Column Type: Structural

Material Properties:
====================
f'c = 5 ksi fy = 60 ksi
Ec = 4030.51 ksi Es = 29000 ksi
fc = 4.25 ksi Rupture strain = Infinity
Ultimate strain = 0.003 in/in
Betal = 0.8

Section:
========
Exterior Points

No.	X (in)	Y (in)	No.	X (in)	Y (in)	No.	X (in)	Y (in)
1	0.0	0.0	2	14.0	0.0	3	14.0	2041.0

Gross section area, Ag = 28574 in^2
Ix = 9.91918e+009 in^4 Iy = 466709 in^4
Xo = 7 in Yo = 1020.5 in

Reinforcement:
==============
Rebar Database: ASTM A615

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.7

Pattern: Irregular

Total steel area, As = 74.80 in^2 at 0.26%

Area in^2	X (in)	Y (in)	Area in^2	X (in)	Y (in)	Area in^2	X (in)	Y (in)
0.20	1.5	1.5	0.20	12.5	1.5	0.20	1.5	12.5
0.20	12.5	12.5	0.20	1.5	23.5	0.20	12.5	23.5
0.20	1.5	34.5	0.20	12.5	34.5	0.20	1.5	45.5
0.20	12.5	45.5	0.20	1.5	56.5	0.20	12.5	56.5
0.20	1.5	67.5	0.20	12.5	67.5	0.20	1.5	78.5
0.20	12.5	78.5	0.20	1.5	89.5	0.20	12.5	89.5
0.20	1.5	100.5	0.20	12.5	100.5	0.20	1.5	111.5
0.20	12.5	111.5	0.20	1.5	122.5	0.20	12.5	122.5
0.20	1.5	133.5	0.20	12.5	133.5	0.20	1.5	144.5
0.20	12.5	144.5	0.20	1.5	155.5	0.20	12.5	155.5
0.20	1.5	166.5	0.20	12.5	166.5	0.20	1.5	177.5
0.20	12.5	177.5	0.20	1.5	188.5	0.20	12.5	188.5
0.20	1.5	199.5	0.20	12.5	199.5	0.20	1.5	210.5
0.20	12.5	210.5	0.20	1.5	221.5	0.20	12.5	221.5
0.20	1.5	232.5	0.20	12.5	232.5	0.20	1.5	243.5
0.20	12.5	243.5	0.20	1.5	254.5	0.20	12.5	254.5
0.20	1.5	265.5	0.20	12.5	265.5	0.20	1.5	276.5

7.1-18

0.20	12.5	276.5	0.20	1.5	287.5	0.20	12.5	287.5
0.20	1.5	298.5	0.20	12.5	298.5	0.20	1.5	309.5
0.20	12.5	309.5	0.20	1.5	320.5	0.20	12.5	320.5
0.20	1.5	331.5	0.20	12.5	331.5	0.20	1.5	342.5
0.20	12.5	342.5	0.20	1.5	353.5	0.20	12.5	353.5
0.20	1.5	364.5	0.20	12.5	364.5	0.20	1.5	375.5
0.20	12.5	375.5	0.20	1.5	386.5	0.20	12.5	386.5
0.20	1.5	397.5	0.20	12.5	397.5	0.20	1.5	408.5
0.20	12.5	408.5	0.20	1.5	419.5	0.20	12.5	419.5
0.20	1.5	2039.5	0.20	12.5	2039.5	0.20	1.5	2028.5
0.20	12.5	2028.5	0.20	1.5	2017.5	0.20	12.5	2017.5
0.20	1.5	2006.5	0.20	12.5	2006.5	0.20	1.5	1995.5
0.20	12.5	1995.5	0.20	1.5	1984.5	0.20	12.5	1984.5
0.20	1.5	1973.5	0.20	12.5	1973.5	0.20	1.5	1962.5
0.20	12.5	1962.5	0.20	1.5	1951.5	0.20	12.5	1951.5
0.20	1.5	1940.5	0.20	12.5	1940.5	0.20	1.5	1929.5
0.20	12.5	1929.5	0.20	1.5	1918.5	0.20	12.5	1918.5
0.20	1.5	1907.5	0.20	12.5	1907.5	0.20	1.5	1896.5
0.20	12.5	1896.5	0.20	1.5	1885.5	0.20	12.5	1885.5
0.20	1.5	1874.5	0.20	12.5	1874.5	0.20	1.5	1863.5
0.20	12.5	1863.5	0.20	1.5	1852.5	0.20	12.5	1852.5
0.20	1.5	1841.5	0.20	12.5	1841.5	0.20	1.5	1830.5
0.20	12.5	1830.5	0.20	1.5	1819.5	0.20	12.5	1819.5
0.20	1.5	1808.5	0.20	12.5	1808.5	0.20	1.5	1797.5
0.20	12.5	1797.5	0.20	1.5	1786.5	0.20	12.5	1786.5
0.20	1.5	1775.5	0.20	12.5	1775.5	0.20	1.5	1764.5
0.20	12.5	1764.5	0.20	1.5	1753.5	0.20	12.5	1753.5
0.20	1.5	1742.5	0.20	12.5	1742.5	0.20	1.5	1731.5
0.20	12.5	1731.5	0.20	1.5	1720.5	0.20	12.5	1720.5
0.20	1.5	1709.5	0.20	12.5	1709.5	0.20	1.5	1698.5
0.20	12.5	1698.5	0.20	1.5	1687.5	0.20	12.5	1687.5
0.20	1.5	1676.5	0.20	12.5	1676.5	0.20	1.5	1665.5
0.20	12.5	1665.5	0.20	1.5	1654.5	0.20	12.5	1654.5
0.20	1.5	1643.5	0.20	12.5	1643.5	0.20	1.5	1632.5
0.20	12.5	1632.5	0.20	1.5	1621.5	0.20	12.5	1621.5
0.20	1.5	430.4	0.20	12.5	430.4	0.20	1.5	441.4
0.20	12.5	441.4	0.20	1.5	452.3	0.20	12.5	452.3
0.20	1.5	463.2	0.20	12.5	463.2	0.20	1.5	474.1
0.20	12.5	474.1	0.20	1.5	485.1	0.20	12.5	485.1
0.20	1.5	496.0	0.20	12.5	496.0	0.20	1.5	506.9
0.20	12.5	506.9	0.20	1.5	517.8	0.20	12.5	517.8
0.20	1.5	528.8	0.20	12.5	528.8	0.20	1.5	539.7
0.20	12.5	539.7	0.20	1.5	550.6	0.20	12.5	550.6
0.20	1.5	561.6	0.20	12.5	561.6	0.20	1.5	572.5
0.20	12.5	572.5	0.20	1.5	583.4	0.20	12.5	583.4
0.20	1.5	594.3	0.20	12.5	594.3	0.20	1.5	605.3
0.20	12.5	605.3	0.20	1.5	616.2	0.20	12.5	616.2
0.20	1.5	627.1	0.20	12.5	627.1	0.20	1.5	638.0
0.20	12.5	638.0	0.20	1.5	649.0	0.20	12.5	649.0
0.20	1.5	659.9	0.20	12.5	659.9	0.20	1.5	670.8
0.20	12.5	670.8	0.20	1.5	681.8	0.20	12.5	681.8
0.20	1.5	692.7	0.20	12.5	692.7	0.20	1.5	703.6
0.20	12.5	703.6	0.20	1.5	714.5	0.20	12.5	714.5
0.20	1.5	725.5	0.20	12.5	725.5	0.20	1.5	736.4
0.20	12.5	736.4	0.20	1.5	747.3	0.20	12.5	747.3
0.20	1.5	758.2	0.20	12.5	758.2	0.20	1.5	769.2
0.20	12.5	769.2	0.20	1.5	780.1	0.20	12.5	780.1
0.20	1.5	791.0	0.20	12.5	791.0	0.20	1.5	802.0
0.20	12.5	802.0	0.20	1.5	812.9	0.20	12.5	812.9
0.20	1.5	823.8	0.20	12.5	823.8	0.20	1.5	834.7
0.20	12.5	834.7	0.20	1.5	845.7	0.20	12.5	845.7
0.20	1.5	856.6	0.20	12.5	856.6	0.20	1.5	867.5
0.20	12.5	867.5	0.20	1.5	878.4	0.20	12.5	878.4
0.20	1.5	889.4	0.20	12.5	889.4	0.20	1.5	900.3
0.20	12.5	900.3	0.20	1.5	911.2	0.20	12.5	911.2
0.20	1.5	922.2	0.20	12.5	922.2	0.20	1.5	933.1

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0.20	12.5	933.1	0.20	1.5	944.0	0.20	12.5	944.0
0.20	1.5	954.9	0.20	12.5	954.9	0.20	1.5	965.9
0.20	12.5	965.9	0.20	1.5	976.8	0.20	12.5	976.8
0.20	1.5	987.7	0.20	12.5	987.7	0.20	1.5	998.6
0.20	12.5	998.6	0.20	1.5	1009.6	0.20	12.5	1009.6
0.20	1.5	1020.5	0.20	12.5	1020.5	0.20	1.5	1031.4
0.20	12.5	1031.4	0.20	1.5	1042.4	0.20	12.5	1042.4
0.20	1.5	1053.3	0.20	12.5	1053.3	0.20	1.5	1064.2
0.20	12.5	1064.2	0.20	1.5	1075.1	0.20	12.5	1075.1
0.20	1.5	1086.1	0.20	12.5	1086.1	0.20	1.5	1097.0
0.20	12.5	1097.0	0.20	1.5	1107.9	0.20	12.5	1107.9
0.20	1.5	1118.8	0.20	12.5	1118.8	0.20	1.5	1129.8
0.20	12.5	1129.8	0.20	1.5	1140.7	0.20	12.5	1140.7
0.20	1.5	1151.6	0.20	12.5	1151.6	0.20	1.5	1162.6
0.20	12.5	1162.6	0.20	1.5	1173.5	0.20	12.5	1173.5
0.20	1.5	1184.4	0.20	12.5	1184.4	0.20	1.5	1195.3
0.20	12.5	1195.3	0.20	1.5	1206.3	0.20	12.5	1206.3
0.20	1.5	1217.2	0.20	12.5	1217.2	0.20	1.5	1228.1
0.20	12.5	1228.1	0.20	1.5	1239.0	0.20	12.5	1239.0
0.20	1.5	1250.0	0.20	12.5	1250.0	0.20	1.5	1260.9
0.20	12.5	1260.9	0.20	1.5	1271.8	0.20	12.5	1271.8
0.20	1.5	1282.8	0.20	12.5	1282.8	0.20	1.5	1293.7
0.20	12.5	1293.7	0.20	1.5	1304.6	0.20	12.5	1304.6
0.20	1.5	1315.5	0.20	12.5	1315.5	0.20	1.5	1326.5
0.20	12.5	1326.5	0.20	1.5	1337.4	0.20	12.5	1337.4
0.20	1.5	1348.3	0.20	12.5	1348.3	0.20	1.5	1359.2
0.20	12.5	1359.2	0.20	1.5	1370.2	0.20	12.5	1370.2
0.20	1.5	1381.1	0.20	12.5	1381.1	0.20	1.5	1392.0
0.20	12.5	1392.0	0.20	1.5	1403.0	0.20	12.5	1403.0
0.20	1.5	1413.9	0.20	12.5	1413.9	0.20	1.5	1424.8
0.20	12.5	1424.8	0.20	1.5	1435.7	0.20	12.5	1435.7
0.20	1.5	1446.7	0.20	12.5	1446.7	0.20	1.5	1457.6
0.20	12.5	1457.6	0.20	1.5	1468.5	0.20	12.5	1468.5
0.20	1.5	1479.4	0.20	12.5	1479.4	0.20	1.5	1490.4
0.20	12.5	1490.4	0.20	1.5	1501.3	0.20	12.5	1501.3
0.20	1.5	1512.2	0.20	12.5	1512.2	0.20	1.5	1523.2
0.20	12.5	1523.2	0.20	1.5	1534.1	0.20	12.5	1534.1
0.20	1.5	1545.0	0.20	12.5	1545.0	0.20	1.5	1555.9
0.20	12.5	1555.9	0.20	1.5	1566.9	0.20	12.5	1566.9
0.20	1.5	1577.8	0.20	12.5	1577.8	0.20	1.5	1588.7
0.20	12.5	1588.7	0.20	1.5	1599.6	0.20	12.5	1599.6
0.20	1.5	1610.6	0.20	12.5	1610.6			

Factored Loads and Moments with Corresponding Capacities: (see user's manual for notation)

No.	Pu kip	Mux k-ft	fMnx k-ft	fMn/Mu
1	3825.0	586.2	606441.0	1034.462
2	5896.0	58244.8	744174.8	12.777
3	6719.0	81037.8	796478.2	9.828
4	7603.0	82933.4	850991.9	10.261
5	8508.0	85206.4	904914.1	10.620
6	9418.0	87554.2	957077.7	10.931
7	3824.2	100076.1	606385.3	6.059
8	5842.1	198988.0	740698.8	3.722
9	6550.8	209984.0	785907.9	3.743
10	7468.5	222084.8	842807.7	3.795
11	8398.2	233805.6	898480.8	3.843
12	9315.8	244931.8	951323.3	3.884
13	2227.0	586.2	494631.7	843.739
14	3031.0	58244.8	551457.9	9.468
15	3383.0	81037.8	575976.5	7.108
16	3740.0	82933.4	600612.4	7.242
17	4076.0	85206.4	623574.9	7.318

7.1-20

18	4408.0	87554.2	646056.8	7.379
19	2227.2	100076.1	494645.4	4.943
20	3085.4	198988.0	555268.3	2.790
21	3550.7	209984.0	587574.9	2.798
22	3874.6	222084.8	609829.9	2.746
23	4186.3	233805.6	631065.6	2.699
24	4510.1	244931.8	652929.8	2.666

*** Program completed as requested! ***

SHEAR WALL SHEAR CHECK

Etabs model: 7.05-CD-straight
Date: 4/22/2005
By: NJR

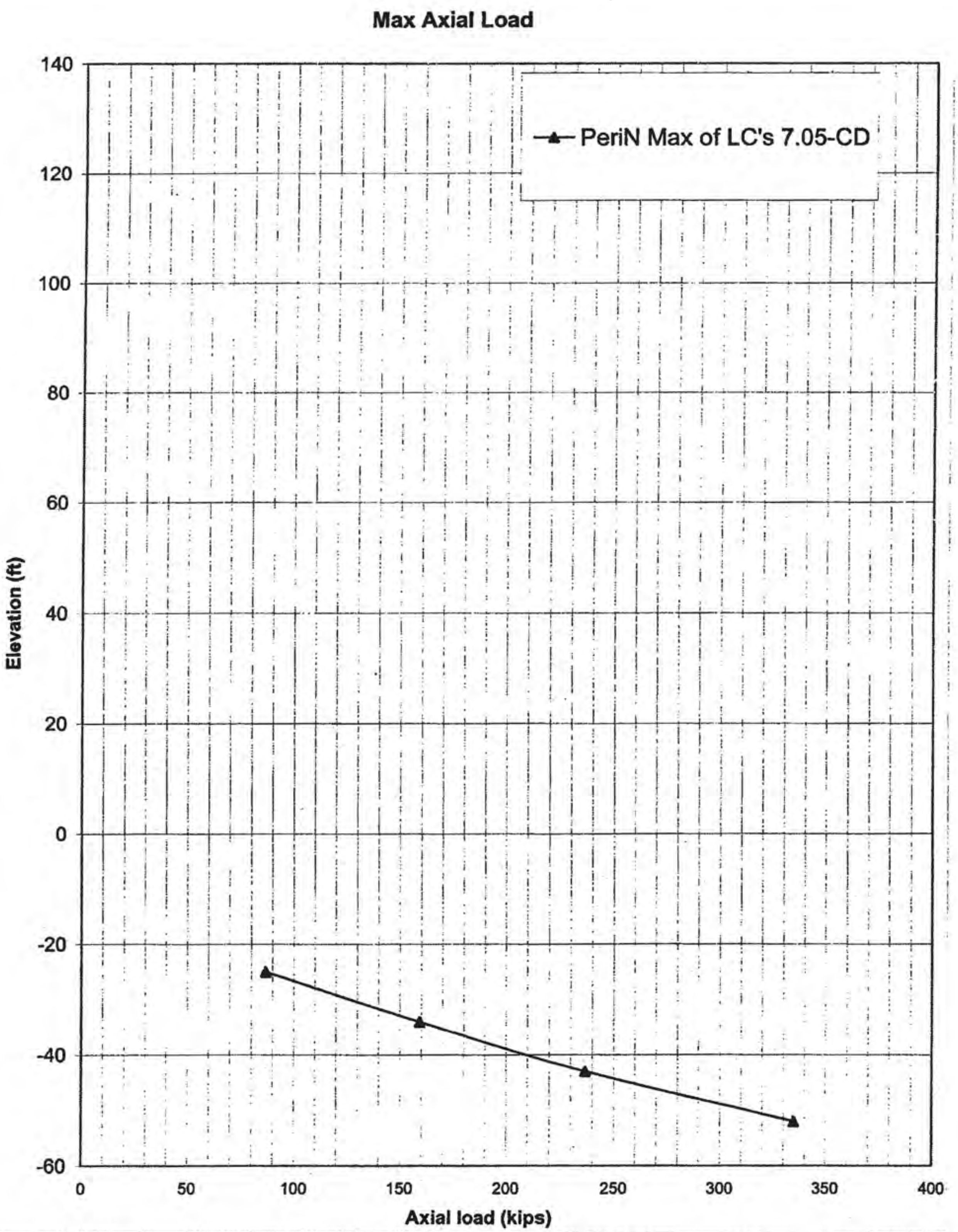
MAY BE VERTICALLY SLOTTING THIS WALL DETAIL IN CD PHASE

phi = 0.6

PERIs								Shear Reinforcement of Wall							Check design					
Wall ID	Story	Width in	Length in	f'_c psi	f_{yv} ksi	ϕ	V_u kips	A_{cp} in²	$V_{n\,max}$ = 10Acp*sqrt(f'c) kips	Check size of section $V_{n\,max}<(V_u/\phi)$	ϕV_c kips	$\rho_{req'd}$	Area of steel within spacing in²	Spacing required in	Spacing provided in	$\rho_{provided}$	V_c+V_s kips	V_n = min of V_c+V_s or 10Acp*sqrt(f'c) kips	$V_u/\phi V_n$	Overstrength Provided $(V_c+V_s)/V_u$
PeriS	L1-L2	14	2041	5000	60	0.60	5773	28574	20205	OK	2425	0.0033	**0.62**	13.6	**9.0**	0.0049	12477	12477	0.77	2.16
	B1-L1	14	2041	5000	60	0.60	7448	28574	20205	OK	2425	0.0049	**0.62**	9.1	**9.0**	0.0049	12477	12477	0.99	1.68
	B5-B1	18	2041	5000	60	0.60	6624	36738	25978	OK	3117	0.0027	**0.62**	13.0	**9.0**	0.0038	13632	13632	0.81	2.06

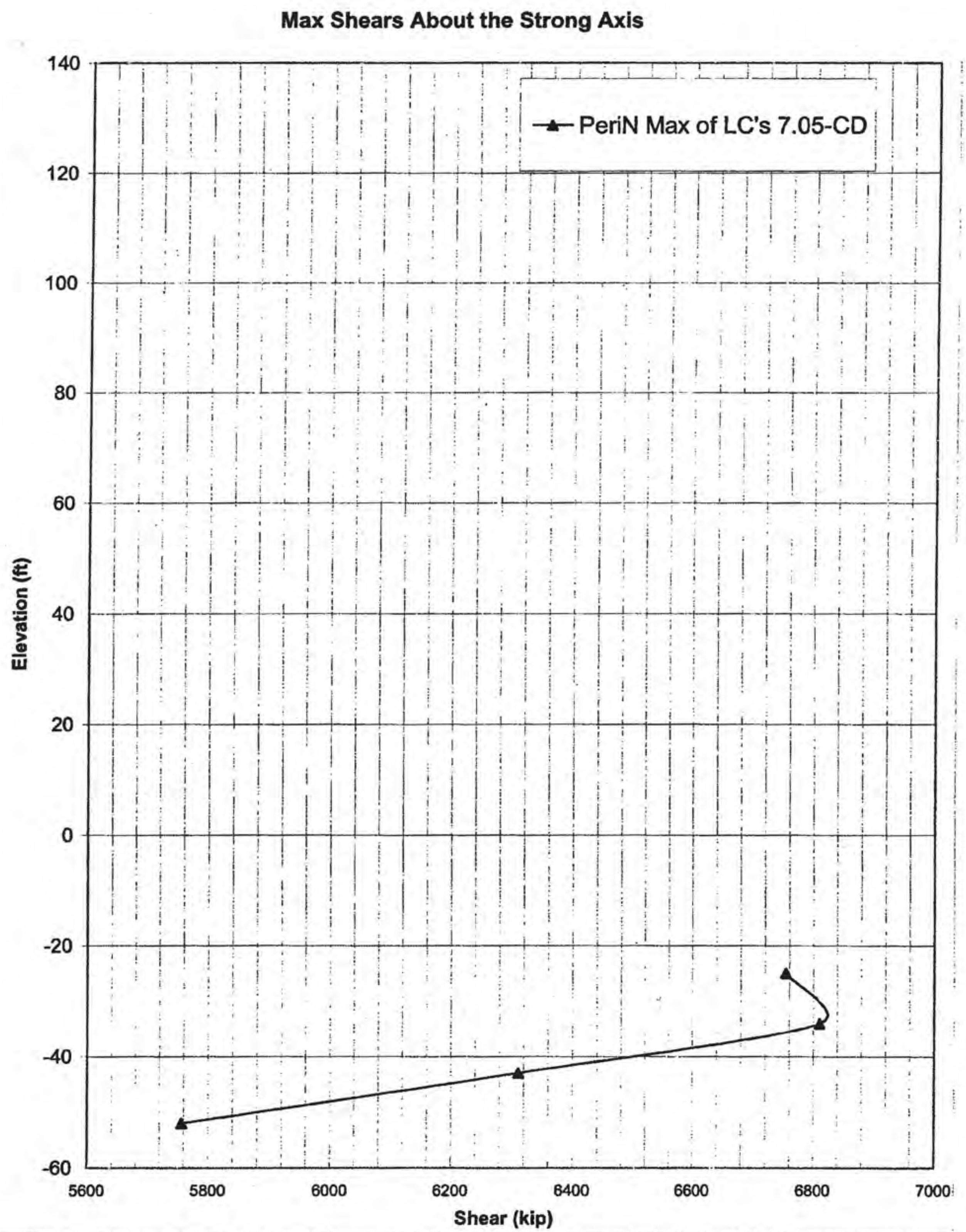
#5 @ 9" EF

7.1-22

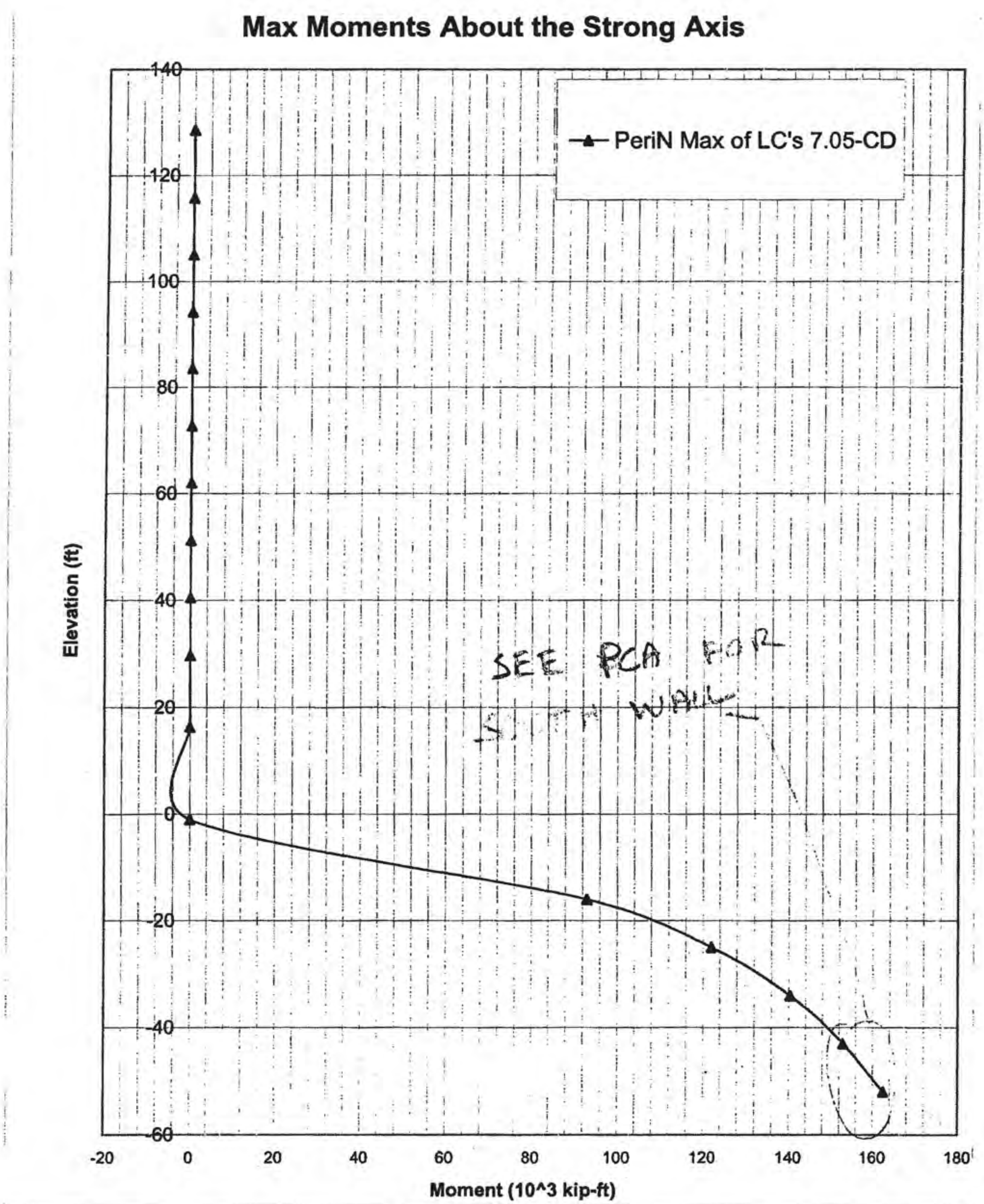

Max Axial Load
PeriN Max of LC's 7.05-CD
Elevation (ft)
140
120
100
80
60
40
20
0
-20
-40
-60
0
50
100
150
200
250
300
350
400
Axial load (kips)

7.1-23

Print Date 4/22/2005



7o1-24



Max Moments About the Strong Axis
PeriN Max of LC's 7.05-CD
Elevation (ft)
Moment (10^3 kip-ft)
140
120
100
80
60
40
20
0
-20
-40
-60
-20
0
20
40
60
80
100
120
140
160
180
SEE PCA FOR

7.1-25

Unit Wt.	0.150	kcf
Min trib area from group	902.5	ft^2 (For Tension)
Max trib area from group	997.5	ft^2 (For Compression)

dlf	dlf	dlf
1.0	1.0	1.0

PeriN

Floor	Usage	Flr. Ht. ft.	Elevation ft.	Width in	Length in	Min Trib A. sq. ft	Min Cum Trib A. sq. ft	Max Trib A. sq. ft	Max Cum Trib A. sq. ft	Floor DL psf	Red. LL psf	Self Wt kips	Min Total DL kips	Min Cum DL kips	Max Total DL kips	Max Cum DL kips	Cum Reducible LL kips	LL % multiplier	Cum Red. LL kips	Min Cum Service kips	Max Cum Service kips	1.42(dlf)D+0.5L Design kips	0.9*D Design kips	1.4(dlf)D+1.7L Design kips
13	Cap	12.75	141.3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1.00	0	0	0	0	0	0
12	Roof	12.83	128.6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0	#DIV/0!
11	Typ	10.75	115.8	0	0	0	0	0	0	0	0	0	0	0	0	0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0	#DIV/0!
10	Typ	10.75	105.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0	#DIV/0!
9	Typ	10.75	94.3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0	#DIV/0!
8	Typ	10.75	83.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0	#DIV/0!
7	Typ	10.75	72.8	0	0	0	0	0	0	0	0	0	0	0	0	0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0	#DIV/0!
6	Typ	10.75	62.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0	#DIV/0!
5	Typ	10.75	51.3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0	#DIV/0!
4	Typ	10.75	40.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0	#DIV/0!
3	Public	13.42	29.8	0	0	0	0	0	0	0	0	0	0	0	0	0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0	#DIV/0!
2	Public	17.33	16.3	0	0	0	0	0	0	0	0	0	0	0	0	0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	0	#DIV/0!
1	Public	15.00	-1.0	14	2041	704	704	799	799	200	100	446	587	587	606	606	70	1.00	70	658	677	896	529	968
B1	Parking	9.00	-16.0	18	2041	647	1351	742	1541	155	50	344	445	1032	459	1066	103	0.66	68	1100	1133	1547	929	1607
B2	Parking	9.00	-25.0	18	2041	647	1999	742	2284	155	50	344	445	1477	459	1525	135	0.59	79	1556	1604	2205	1329	2270
B3	Parking	9.00	-34.0	18	2041	647	2646	742	3026	155	50	344	445	1922	459	1985	168	0.54	91	2012	2075	2864	1729	2933
B4	Parking	9.00	-43.0	18	2041	647	3294	742	3769	155	50	344	445	2366	459	2444	200	0.51	102	2469	2546	3522	2130	3596
Base		0.00	-52.0																					
												1824												

7.1-26

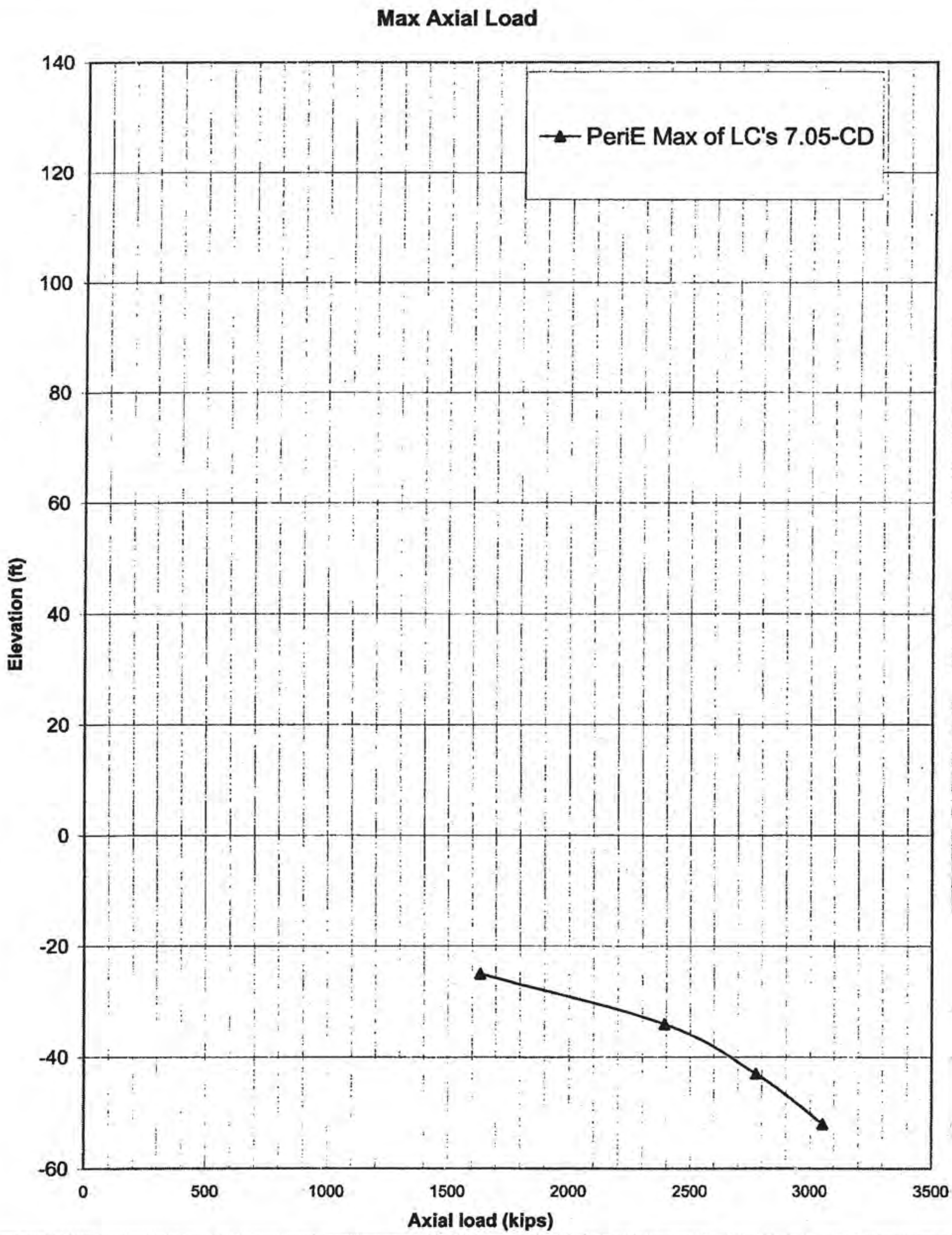
SHEAR WALL SHEAR CHECK

Etabs model: 7.05-CD-straight
Date: 4/22/2005
By: NJR

phi = 0.6

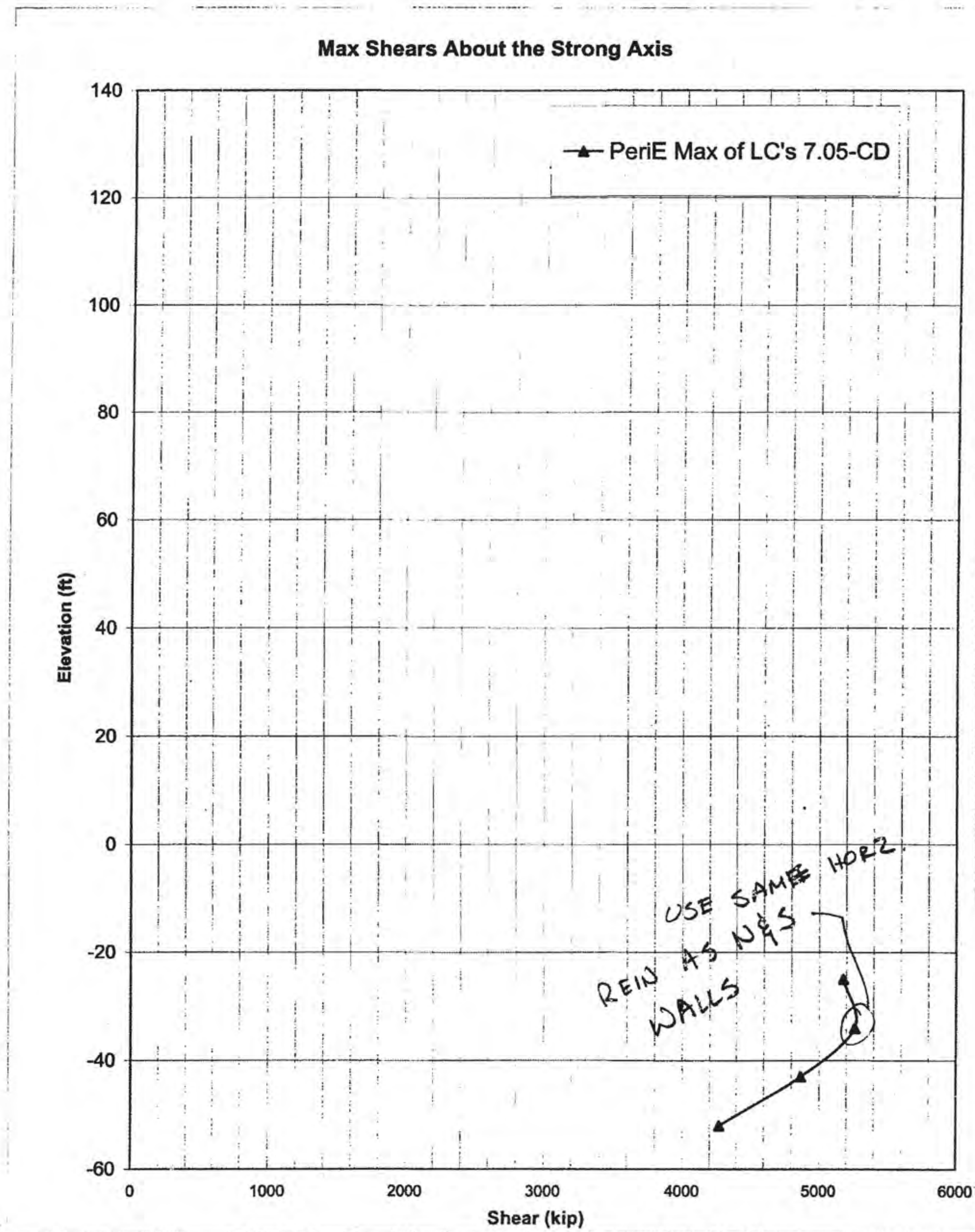
PERIN								Shear Reinforcement of Wall							Check design					
Wall ID	Story	Width in	Length in	f'_c psi	f_{yv} ksi	ϕ	V_u kips	A_{cp} in^2	$V_{n\,max}$ = 10Acp*sqrt(f'c) kips	Check size of section $V_{n\,max}<(V_u/\phi)$	ϕV_c kips	$\rho_{req'd}$	Area of steel within spacing in^2	Spacing required in	Spacing provided in	$\rho_{provided}$	V_c+V_s kips	V_n = min of V_c+V_s or 10Acp*sqrt(f'c) kips	$V_u/\phi V_n$	Overstrength Provided $(V_c+V_s)/V_u$
PeriS	B1-L1	14	2041	5000	60	0.60	6754	28574	20205	OK	2425	0.0042	0.62	10.5	9.0	0.0049	12477	12477	0.90	1.85
	B5-B1	18	2041	5000	60	0.60	6811	36738	25978	OK	3117	0.0028	0.62	12.3	9.0	0.0038	13632	13632	0.83	2.00

7.1-27


Max Axial Load
PeriE Max of LC's 7.05-CD
Elevation (ft)
140
120
100
80
60
40
20
0
-20
-40
-60
Axial load (kips)
0
500
1000
1500
2000
2500
3000
3500

7.1-28

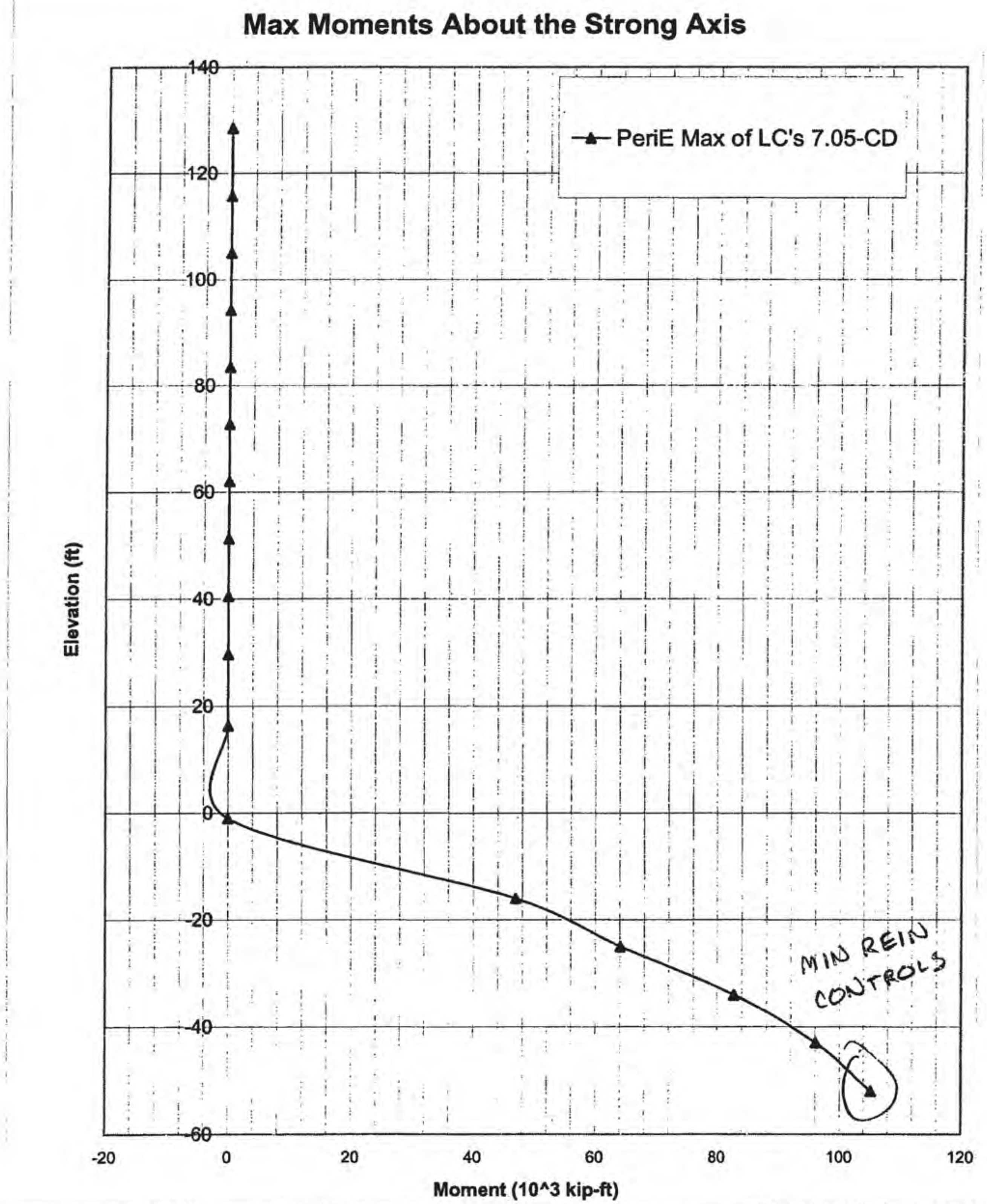
Print Date 4/22/2005



70l-29

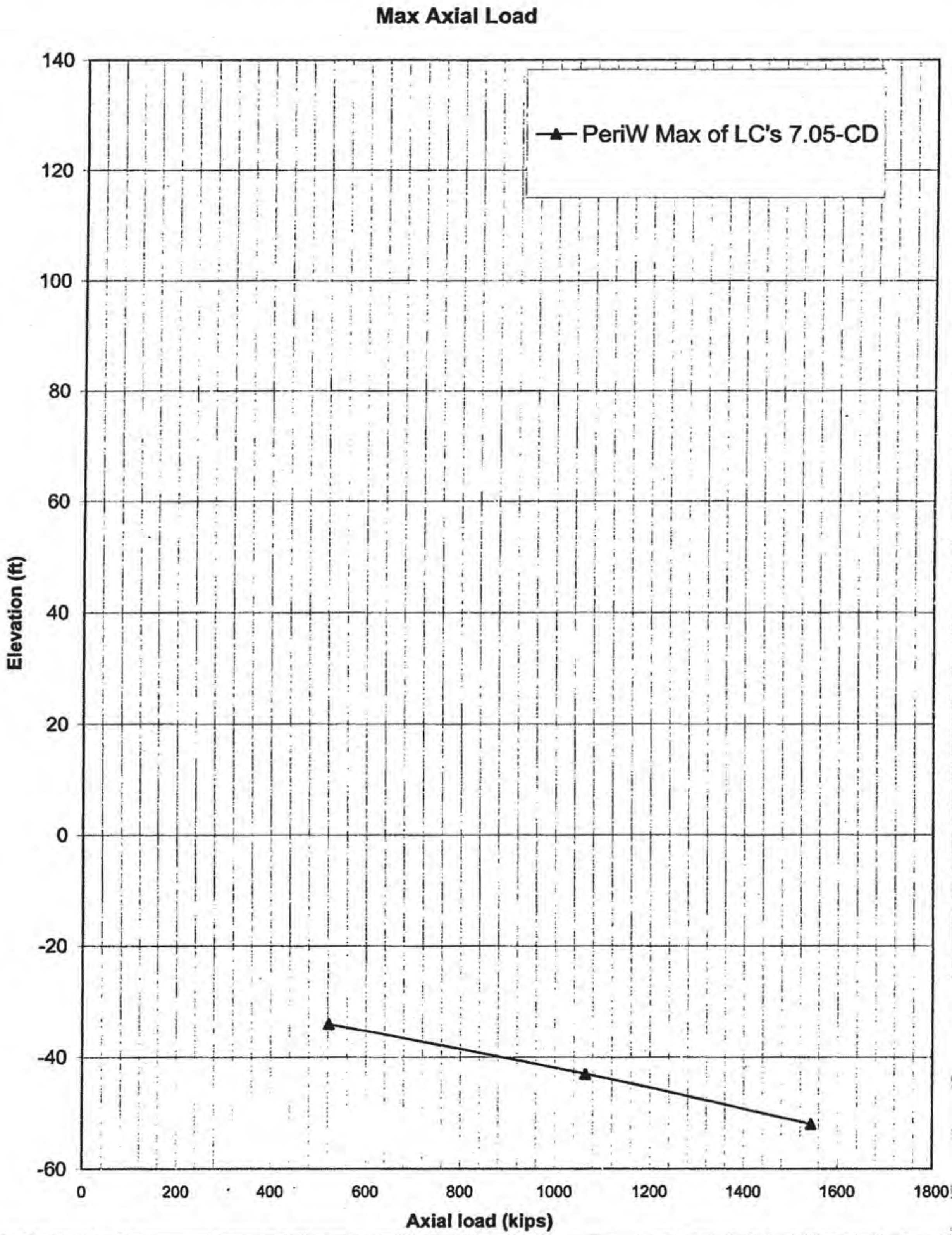
DODSONNOC00000374

Print Date 4/22/2005



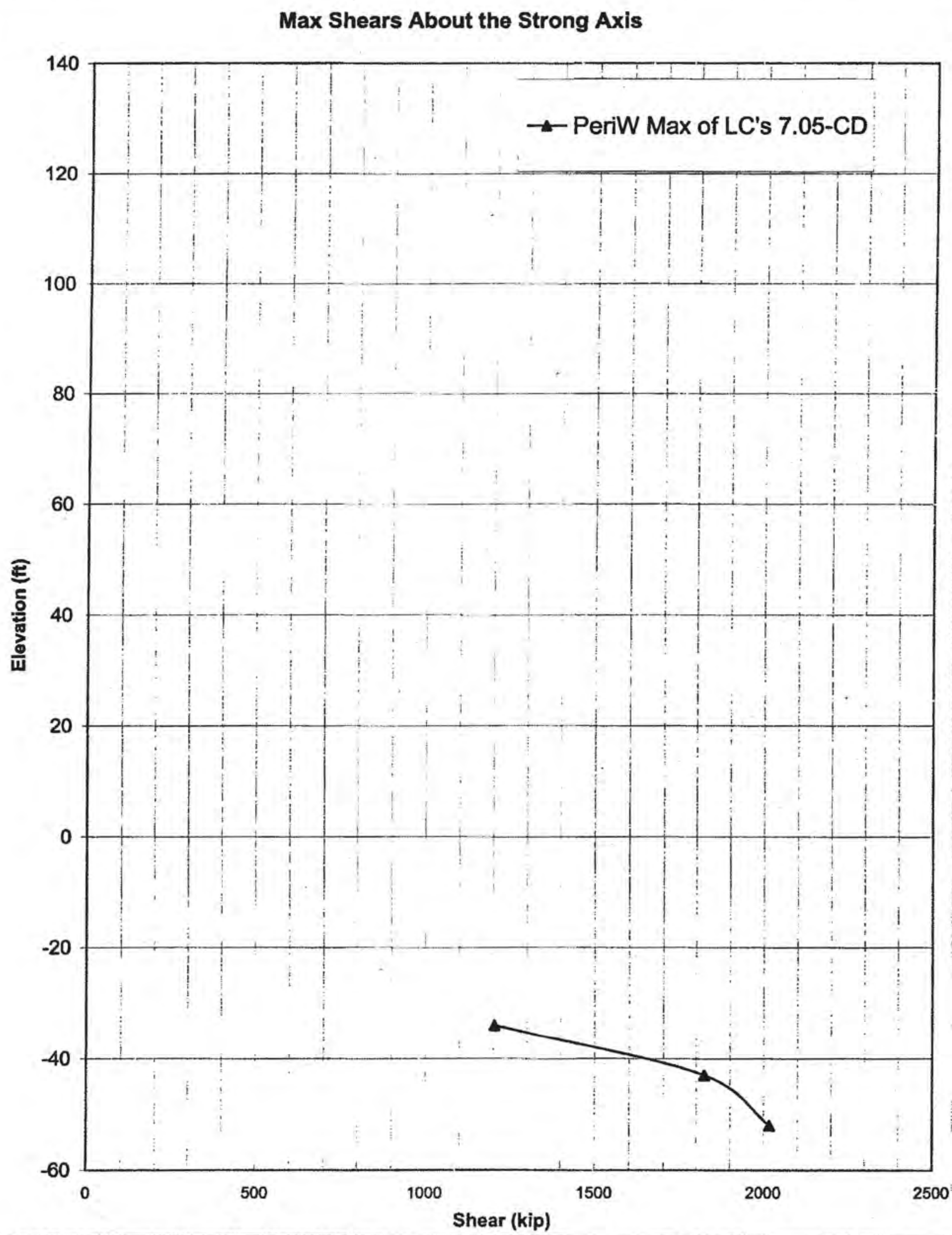
7.1-30

DODSONNOC00000375

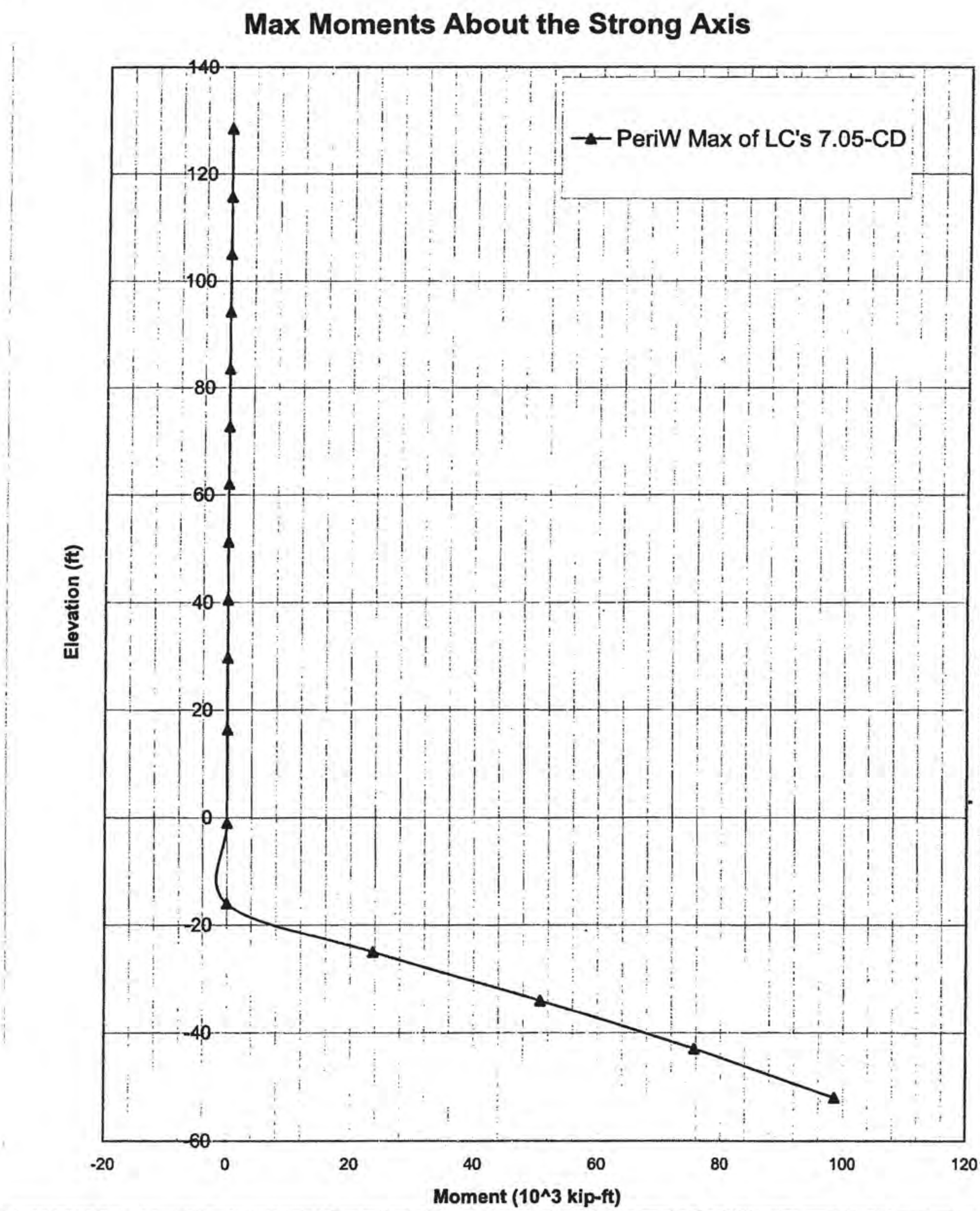

Max Axial Load
PeriW Max of LC's 7.05-CD
Elevation (ft)
Axial load (kips)
140
120
100
80
60
40
20
0
-20
-40
-60
0
200
400
600
800
1000
1200
1400
1600
1800

7.1-31

Print Date 4/22/2005



7.1-32


Max Moments About the Strong Axis
PeriW Max of LC's 7.05-CD
Elevation (ft)
140
120
100
80
60
40
20
0
-20
-40
-60
-20
0
20
40
60
80
100
120
Moment (10^3 kip-ft)

7-1-33

7.2 West Perimeter Wall

7.2 West Perimeter Wall

The west perimeter wall is similar in geometry to the other walls but only extends from level B2 down to level B5. The wall is 27'-0" high and is braced at each basement level slab every 9'-0". The west wall is 30" thick for the entire height.

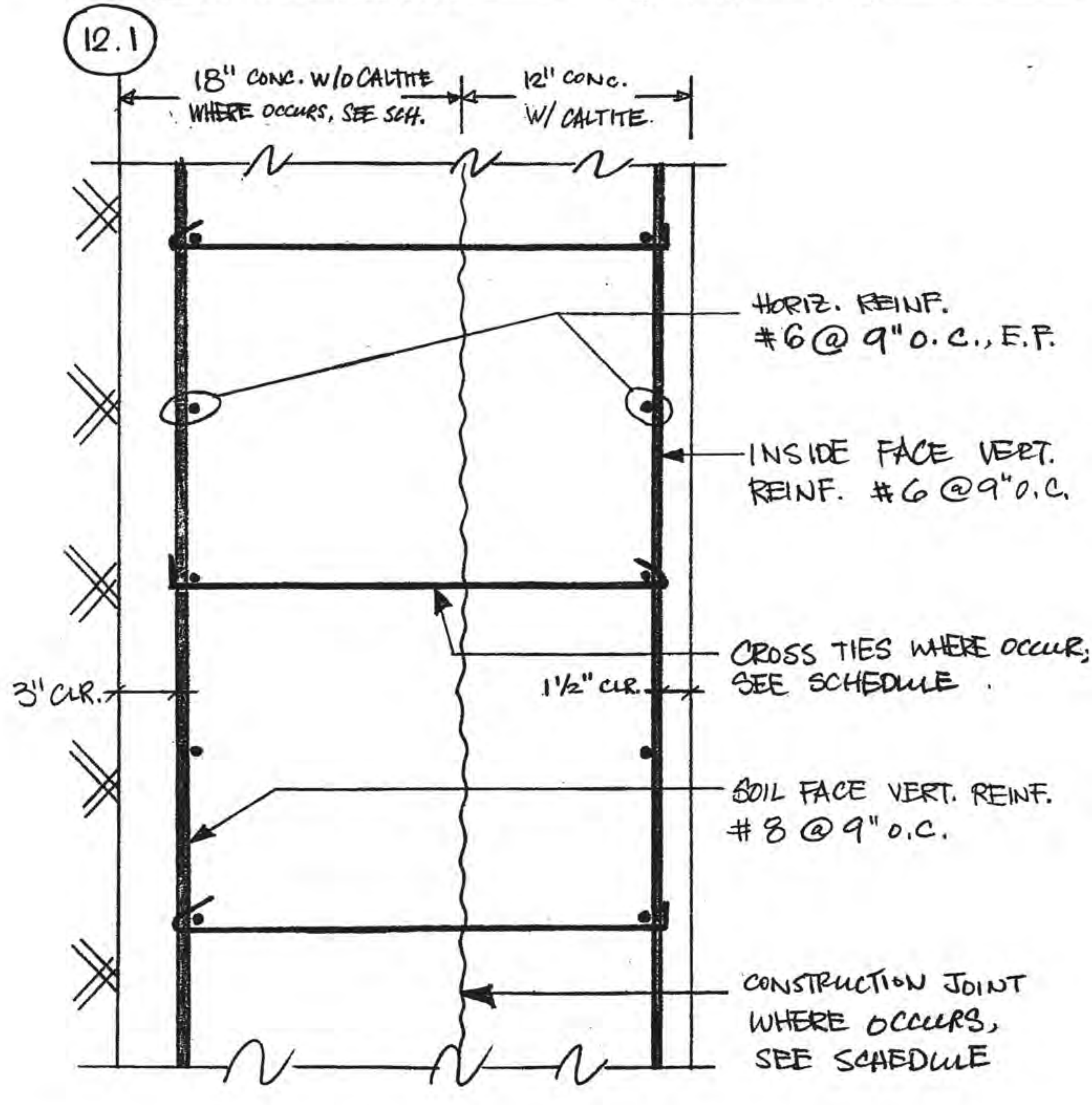
Three options exist for the contractor in constructing this wall. The 30" wall can be cast monolithically or cast in two sections - 18" thick concrete with Caltite admixture and 12" thick concrete. If cast in two sections, the surface between the two areas can either be intentionally roughened or left smooth. The required amount of cross ties varies depending on the contractor's choice.

The west wall is modeled and analyzed using the computational program, RISA. Loads applied to the wall include the permanent and seismic soil pressure along the height of the wall. A traffic surcharge is applied along the top 10 feet of the wall. Since the west wall is in between the tower and the podium, a surcharge from the tower piles is also applied to the wall. The wall is assumed to be fixed at the base (level B5) and pinned at each level and at the top (B4-B2).

The shear in the wall due to the out-of-plane loads is checked assuming the concrete shear capacity is sufficient to take applied shear. Horizontal shear reinforcement is required for resisting the in-plane loads along the wall. The required vertical flexural reinforcement is designed for both the interior and soil faces based on the maximum moments obtained from the RISA analysis.

DESIMONE

Project: 301 MISSION STREET
Page ______ Of ______
Project No.: 4069
Date: 2/4/05
Item: FOUNDATION SECTION
By: ML
Ch'kd ______



SECTION OF FOUNDATION WALL BETWEEN TOWER & PODIUM

7.2-2

DESIMONE

Project 301 MISSION STREET

Project No. 4069

Item FOUNDATION WALL

Page ________ Of ________

Date 2/4/05

By ML Ch'kd ________

Case No.	Description	Construction Joint	Cross Ties
1	30" wall cast monolithically	Not applicable	Not required
2	18" concrete (w/o Caltite) cast prior to casting of 12" concrete (w/ Caltite)	Intentionally roughened to ¼" full amplitude	#5 @ 18" o.c., e.w. vertically & horizontally
3	18" concrete (w/o Caltite) cast prior to casting of 12" concrete (w/ Caltite)	Not intentionally roughened	#5 @ 9" o.c., e. w. vertically & horizontally

FOUNDATION WALL BETWEEN TOWER & PODIUM

SCHEDULE FOR DIFFERENT CONSTRUCTION CASES

7.2-3

DESIMONE

Project: 301 Mission
Project No.: 4069
Item: FOUNDATION WALL
Page ____ Of ____
Date: 2/4/05
By: ML Ch'kd ____

CASE 1. 30" WALL CAST MONOLITHICALLY

$\phi V_c = 0.85 \times 2\sqrt{5000} \times 12 \times 28 / 1000 = 40.4^k$

$V_u = 31.3^k$ DCR = $31.3/40.4 = \underline{0.77}$ O.K.

CASE 2 C.J. ROUGHENED, MIN. TIES

$A_v = \frac{50\, b_w s}{f_y} = \frac{50 \times 12 \times 12}{60,000} = 0.12\ in^2/ft^2$

#5 @ 18" O.C., E.W. $A_v = \frac{0.31\ in^2}{1.5 \times 1.5\ ft^2} = 0.138\ \frac{in^2}{ft^2}$

$\phi V_{nh} = 0.85(260 + 0.6 \rho_v f_y)\lambda b_v d$

$= 0.85(260 + 0.6 \times \frac{0.138}{144} \times 60,000) \times 1.0 \times 12 \times 28$

$= 84.1^k$

DCR = $31.3^k / 84.1^k = \underline{0.37}$ O.K.

CASE 3 C.J. SMOOTH, TIES TAKE ALL SHEAR

#5 @9" O.C., E.W. $A_v = \frac{0.31\ in^2}{0.75 \times 0.75\ ft^2} = 0.55\ \frac{in^2}{ft^2}$

$\phi V_{nh} = 0.85 \times 0.6 \times \frac{0.55}{144} \times 60,000 \times 1.0 \times 12 \times 28 / 1000$

$= 39.3^k$

DCR = $31.3^k / 39.3^k = \underline{0.80}$ O.K. 7.2-4

Lateral Earth Pressure Restrained Wall Condition
Ground Elev. = 0'-0", Design Ground Water Elev. = -5.36'

	Static	Seismic	
Above -5.36'	60	40	15H
Below -5.36'	90	85	15H

Negative Elevation (ft)	Perm Pressure (psf)	Force (k)	1.7 Perm Pressure (psf)
0.50	30	854	51
5.36	322	8,839	547
16.25	1,302	15,360	2,213
25.25	2,112	1,067	3,590
25.75	2,157	21,583	3,666
34.25	2,922	29,940	4,967
43.25	3,732	2,824	6,344
44.00	3,799	32,147	6,459
51.75	4,497		7,644
		112,615	

Negative Elevation (ft)	Pile Pressure (psf)	Force (k)
0.50	0	0
5.36	0	0
16.25	0	0
25.25	0	0
25.75	0	2,709
34.25	638	8,775
43.25	1,313	1,005
44.00	1,369	15,113
51.75	2,531	
		27,602

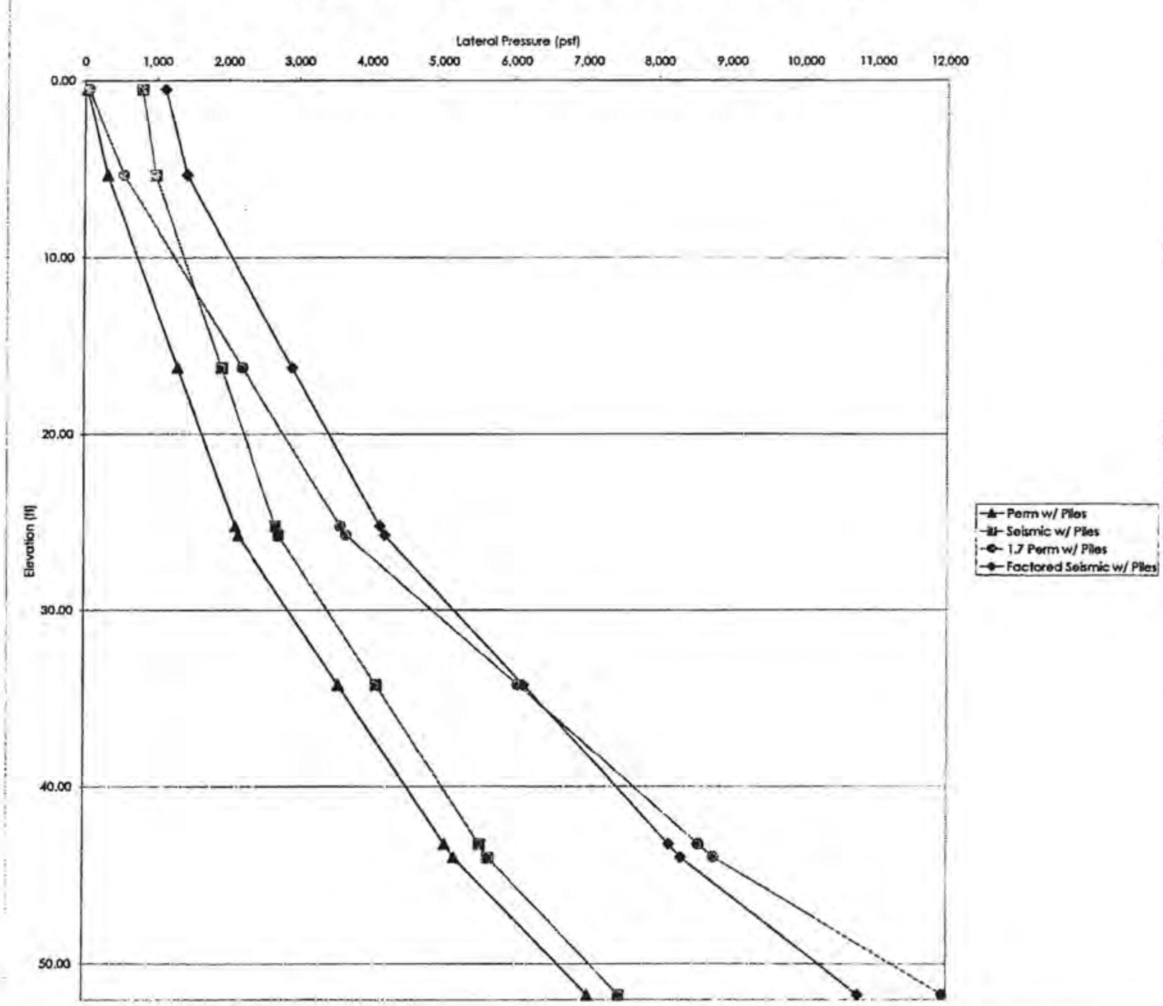
Negative Elevation (ft)	Middle Total Pressure (psf)	Middle Total Force (k)	1.7 Perm + 1.7 Pile Pressure (psf)
0.50	30	854	51
5.36	322	8,839	547
16.25	1,302	15,360	2,213
25.25	2,112	1,067	3,590
25.75	2,157	24,293	3,666
34.25	3,559	38,715	6,051
43.25	5,044	3,830	8,575
44.00	5,168	47,259	8,786
51.75	7,028		11,948
		114,097	

Negative Elevation (ft)	Seismic Soil (psf)	Seismic Incremt (psf)	Seismic Pressure (psf)	Force (k)	1.6 Soil + 1.4 Seismic Pressure (psf)
0.5	20	776	796	4,342	1,119
5.36	214	776	991	15,828	1,430
16.25	1,140	776	1,916	20,689	2,911
25.25	1,905	776	2,681	1,351	4,135
25.75	1,948	776	2,724	26,223	4,203
34.25	2,670	776	3,446	34,459	5,359
43.25	3,435	776	4,211	3,182	6,583
44.00	3,499	776	4,275	35,684	6,685
51.75	4,158	776	4,934		7,739
				141,760	

Negative Elevation (ft)	Pile Pressure (psf)	Force (k)
0.50	0	0
5.36	0	0
16.25	0	0
25.25	0	0
25.75	0	2,709
34.25	638	8,775
43.25	1,313	1,005
44.00	1,369	15,113
51.75	2,531	
		27,602

Negative Elevation (ft)	Middle Total Pressure (psf)	Middle Total Force (k)	1.6 Soil + 1.4 Seismic + 1.2 Pile Pressure (psf)
0.50	796	4,342	1,119
5.36	991	15,828	1,430
16.25	1,916	20,689	2,911
25.25	2,681	1,351	4,135
25.75	2,724	28,932	4,203
34.25	4,084	43,234	6,124
43.25	5,524	4,188	8,158
44.00	5,644	50,797	8,327
51.75	7,465		10,776
		169,362	

301 Mission Street - Foundation Design



7.2-5

Foundation Wall Design Summary

Middle Foundation Wall between Tower & Podium

Foundation elevation per drawings 11/03/04
Lateral soil pressure per geotech report dated 1/13/2005
RISA model dated 1/27/2005 - Pinned at Top, Fixed at Base

DEMAND

Design Shear (k)

Level	Height	Perm	Seismic
B2–B3	9'-0"	13.5	14.3
B3–B4	9'-0"	14.3	14.2
B4–B5	9'-0"	25.8	23.7

Design Moment (k-ft)

M+: Steel on Interior Face

Level	Height	Perm	Seismic
B2–B3	9'-0"	27.0	30.0
B3–B4	9'-0"	29.2	28.1
B4–B5	9'-0"	29.1	26.4

M-: Steel on Soil Face

Level	Height	Perm	Seismic
B2–B3	9'-0"	34.2	34.5
B3–B4	9'-0"	33.0	33.8
B4–B5	9'-0"	84.4	77.6

DESIGN FORCES

Level	Height	Shear	M+ Interior	M- Soil
B2–B3	9'-0"	14.3	30.0	34.5
B3–B4	9'-0"	14.3	29.2	33.8
B4–B5	9'-0"	25.8	29.1	84.4

WALL DESIGN

f'c = 5 ksi

Level	Height		M+ Interior	M- Soil
B2–B3	9'-0"	T = 30"	#6 @9"	#8 @9"
B3–B4	9'-0"	T = 30"	#6 @9"	#8 @9"
B4–B5	9'-0"	T = 30"	#6 @9"	#8 @9"

CAPACITY

Level	Height	Shear	M+ Interior	M- Soil
B2–B3	9'-0"	41.5	75.3	122.7
B3–B4	9'-0"	41.5	75.3	122.7
B4–B5	9'-0"	41.5	75.3	122.7

DEMAND-CAPACITY RATIOS

Level	Height	Shear	M+ Interior	M- Soil
B2–B3	9'-0"	0.34	0.40	0.28
B3–B4	9'-0"	0.34	0.39	0.28
B4–B5	9'-0"	0.62	0.39	0.69

Foundation Wall Design

CONCRETE SHEAR CAPACITY, k per ft

Concrete to take all shear (no shear reinf.)
Assume d = T - 1.25" at inside face for shear

	Concrete Strength			
T (in)	3 ksi	4ksi	5 ksi	6 ksi
6	5.3	6.1	6.9	7.5
8	7.5	8.7	9.7	10.7
10	9.8	11.3	12.6	13.8
12	12.0	13.9	15.5	17.0
14	14.2	16.5	18.4	20.1
16	16.5	19.0	21.3	23.3
18	18.7	21.6	24.2	26.5
20	21.0	24.2	27.0	29.6
22	23.2	26.8	29.9	32.8
24	25.4	29.4	32.8	35.9
30	32.1	37.1	41.8	45.4

MINIMUM HORIZONTAL STEEL REQUIREMENT
[ACI 14.3.3]

	Total
T (in)	As,min
6	0.18
8	0.24
10	0.30
12	0.36
14	0.42
16	0.48
18	0.54
20	0.60
22	0.66
24	0.72

Area of Steel for Each Face

Spg (in)	#4	#5	#6	#7	#8
6	0.40	0.62	0.88	1.20	1.58
8	0.30	0.47	0.66	0.90	1.19
10	0.24	0.37	0.53	0.72	0.95
12	0.20	0.31	0.44	0.60	0.79
14	0.17	0.27	0.38	0.51	0.68
16	0.15	0.23	0.33	0.45	0.59
18	0.13	0.21	0.29	0.40	0.53
20	0.12	0.19	0.26	0.36	0.47
22	0.11	0.17	0.24	0.33	0.43
24	0.10	0.16	0.22	0.30	0.40

WALL FLEXURAL CAPACITY, k-ft per ft

For M+: Assume d = T - 0.75" - dia/2 (verts outside of horiz.)

Wall T = 30 in   f'c = 5 ksi

Spg (in)	#4	#5	#6	#7	#8	#9	#10	#11
6	51.78	79.72	112.30	151.78	197.80	306.17	382.15	460.70
7	44.43	68.45	96.50	130.56	170.36	264.46	330.83	399.82
8	38.91	59.98	84.61	114.55	149.59	232.73	291.62	353.08
9	34.61	53.37	75.32	102.03	133.34	207.79	260.70	316.08
10	31.17	48.06	67.87	91.98	120.27	187.67	235.70	286.08
11	28.35	43.73	[illegible]	83.73	109.53	171.11	215.06	261.27
12	25.99	[illegible]	[illegible]	[illegible]	100.55	157.22	197.75	240.42
13	24.00	37.03	[illegible]	[illegible]	92.94	145.42	183.01	222.64
14	22.29	[illegible]	[illegible]	[illegible]	86.39	135.27	170.31	207.30
15	20.81	[illegible]	[illegible]	[illegible]	80.71	126.44	159.27	193.94
16	19.52	[illegible]	[illegible]	[illegible]	75.73	118.69	149.56	182.20
17	18.37	[illegible]	[illegible]	[illegible]	[illegible]	111.84	140.97	171.80
18	17.35	26.83	[illegible]	[illegible]	[illegible]	105.73	133.31	162.51

Area of Steel for Each Face

Spg (in)	#4	#5	#6	#7	#8	#9	#10	#11
6	0.40	0.62	0.88	1.20	1.58	2.00	2.54	3.12
7	0.34	0.53	0.75	1.03	1.35	1.71	2.18	2.67
8	0.30	0.47	0.66	0.90	1.19	1.50	1.91	2.34
9	0.27	0.41	0.59	0.80	1.05	1.33	1.69	2.08
10	0.24	0.37	0.53	0.72	0.95	1.20	1.52	1.87
11	0.22	0.34	0.48	0.65	0.86	1.09	1.39	1.70
12	0.20	0.31	0.44	0.60	0.79	1.00	1.27	1.56
13	0.18	0.29	0.41	0.55	0.73	0.92	1.17	1.44
14	0.17	0.27	0.38	0.51	0.68	0.86	1.09	1.34
15	0.16	0.25	0.35	0.48	0.63	0.80	1.02	1.25
16	0.15	0.23	0.33	0.45	0.59	0.75	0.95	1.17
17	0.14	0.22	0.31	0.42	0.56	0.71	0.90	1.10
18	0.13	0.21	0.29	0.40	0.53	0.67	0.85	1.04
dia (in)	0.500	0.625	0.750	0.875	1.000	1.128	1.270	1.410
fy (ksi)	60	60	60	60	60	75	75	75
Total As,min [ACI 10.5.1]	1.21	1.21	1.20	1.20	1.20	0.96	0.95	0.95

For M-: Assume d = T - 3" - dia/2 (verts outside of horiz.)

Wall T = 30 in   f'c = 5 ksi

Spg (in)	#4	#5	#6	#7	#8	#9	#10	#11
6	47.73	73.44	103.39	139.63	181.81	280.86	350.01	421.22
7	[illegible]	[illegible]	88.87	120.15	156.64	242.76	303.27	365.97
8	35.87	55.27	77.92	105.43	137.59	213.75	267.51	323.46
9	31.91	49.19	69.38	93.93	122.67	190.92	239.27	289.76
10	[illegible]	44.31	[illegible]	84.69	110.67	172.49	216.41	262.39
11	26.14	[illegible]	[illegible]	77.10	100.81	157.30	197.53	239.74
12	[illegible]	[illegible]	[illegible]	[illegible]	92.56	144.57	181.67	220.67
13	[illegible]	[illegible]	[illegible]	[illegible]	85.55	133.74	168.17	204.41
14	[illegible]	[illegible]	[illegible]	[illegible]	79.54	124.42	156.54	190.38
15	19.19	29.62	[illegible]	[illegible]	74.31	116.31	146.41	178.15
16	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	109.20	137.51	167.39
17	16.94	26.18	[illegible]	[illegible]	65.68	102.91	129.63	157.86
18	[illegible]	24.71	34.92	47.39	62.07	97.30	122.60	149.35

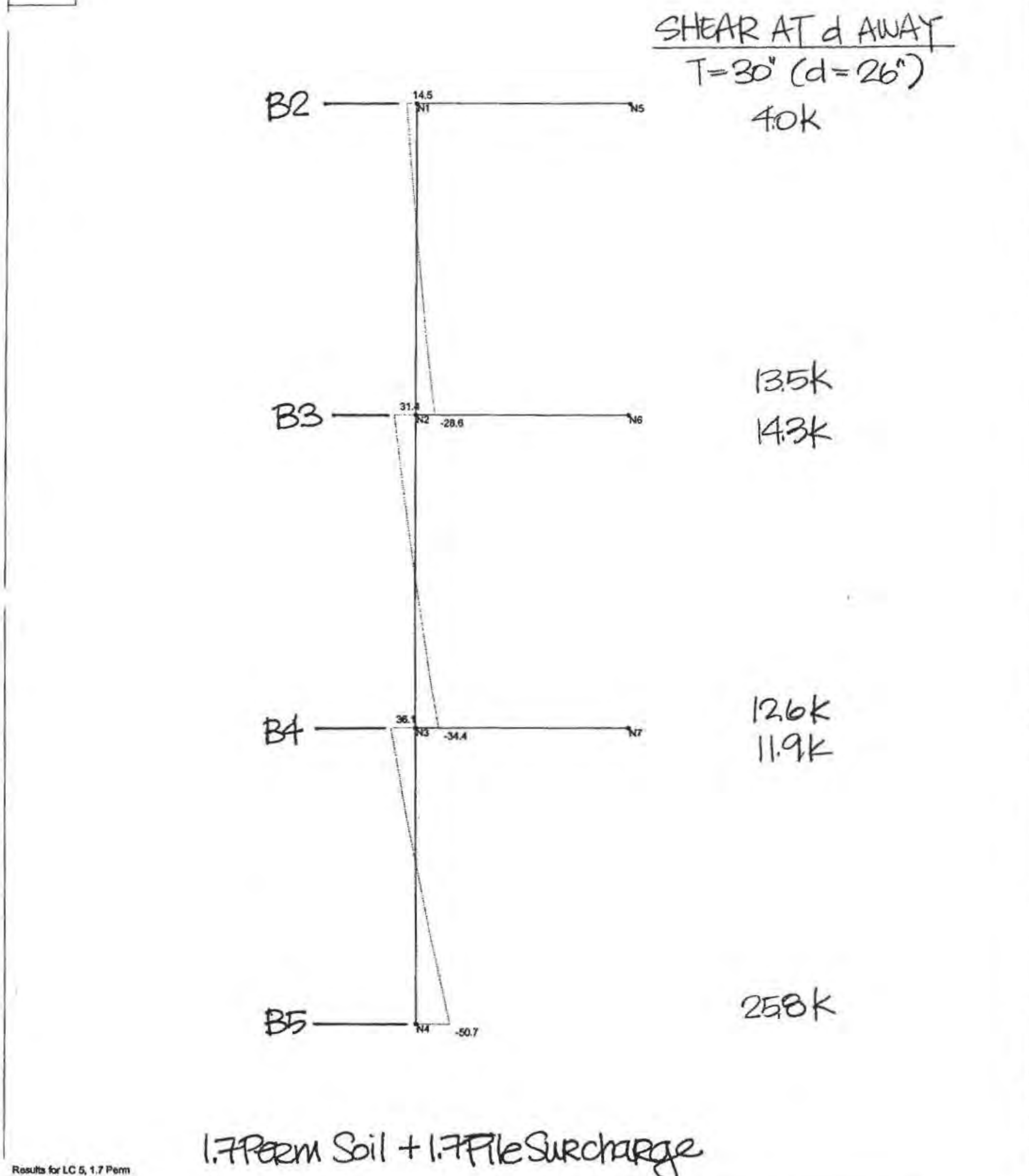
$$V: \frac{V_u}{\phi V_c} = \frac{25.8}{41.5} = 0.62 \quad (T = 30")$$

$$M+: \frac{M_u}{\phi M_n} = \frac{30.0}{75.32} = 0.39 \quad (\#6 @ 9" \text{ o.c.})$$

$$M-: \frac{M_u}{\phi M_n} = \frac{84.4}{122.67} = 0.69 \quad (\#8 @ 9" \text{ o.c.})$$

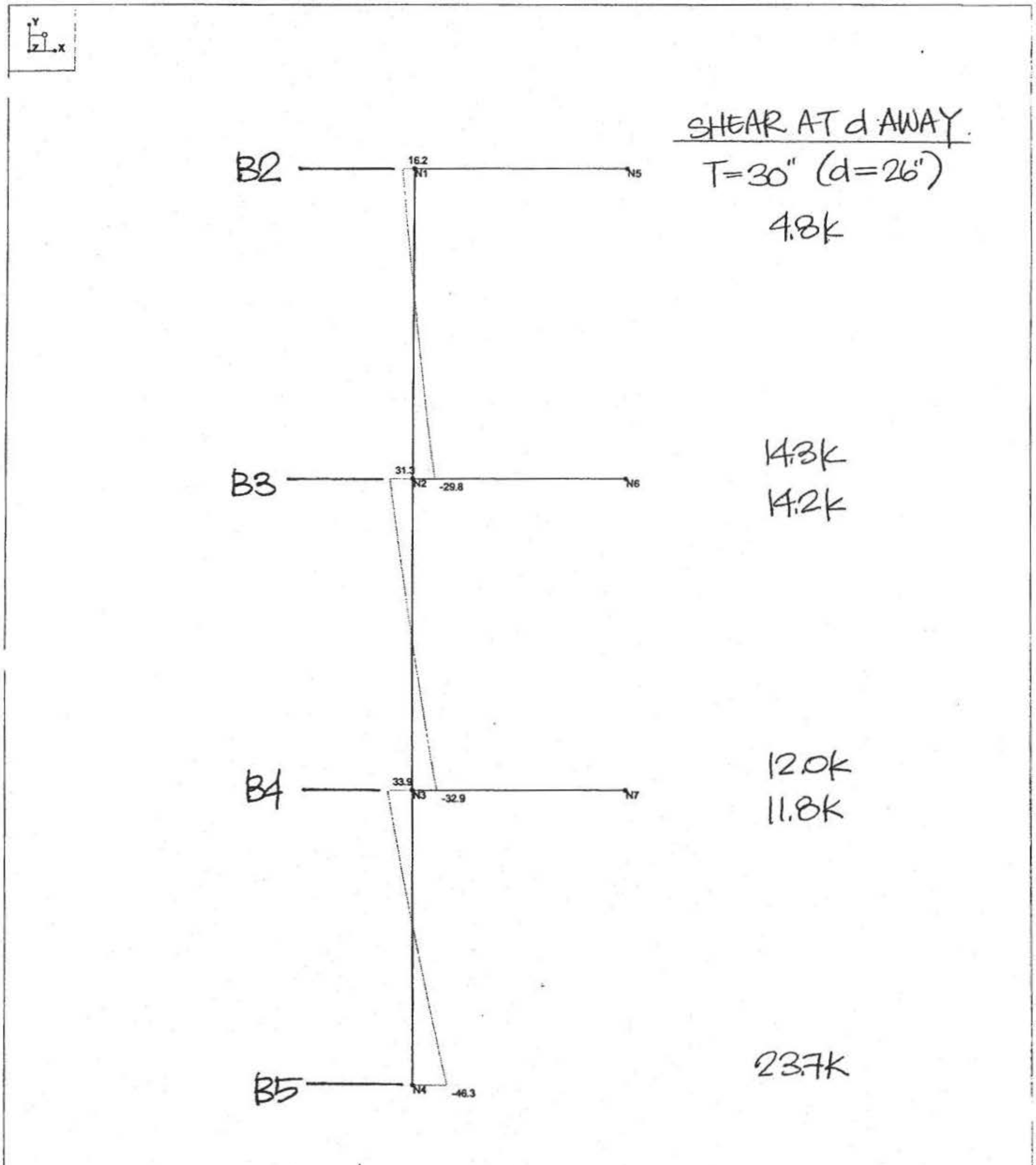
7.2-7

4069-20040827-MKL-Fdn-Wall.xls\Fdn Wall Design Middle Foundation Wall (Between Tower & Podium)



DeSimone Consulting Eng...	301 Mission Street Middle Foundation Wall	
ML		Mar 10, 2005 at 1:32 PM
4069		4069-20050127-MKL-Middle-Fdn-...

7.2-8

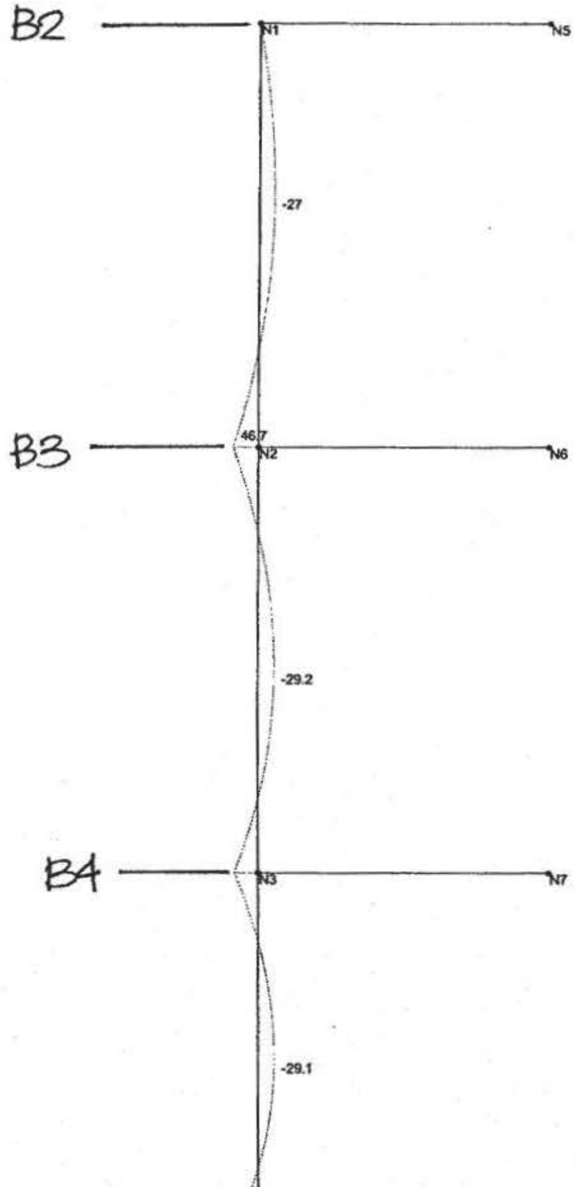


1.6 Seismic Soil + 1.4 Seismic Increment + 1.2 Pile Surcharge

Results for LC 6, Seismic Combo
Member y Shear Forces (k)

DeSimone Consulting Eng...	301 Mission Street Middle Foundation Wall	
ML		Mar 10, 2005 at 1:32 PM
4069		4069-20050127-MKL-Middle-Fdn-...

MOMENT AT FACE

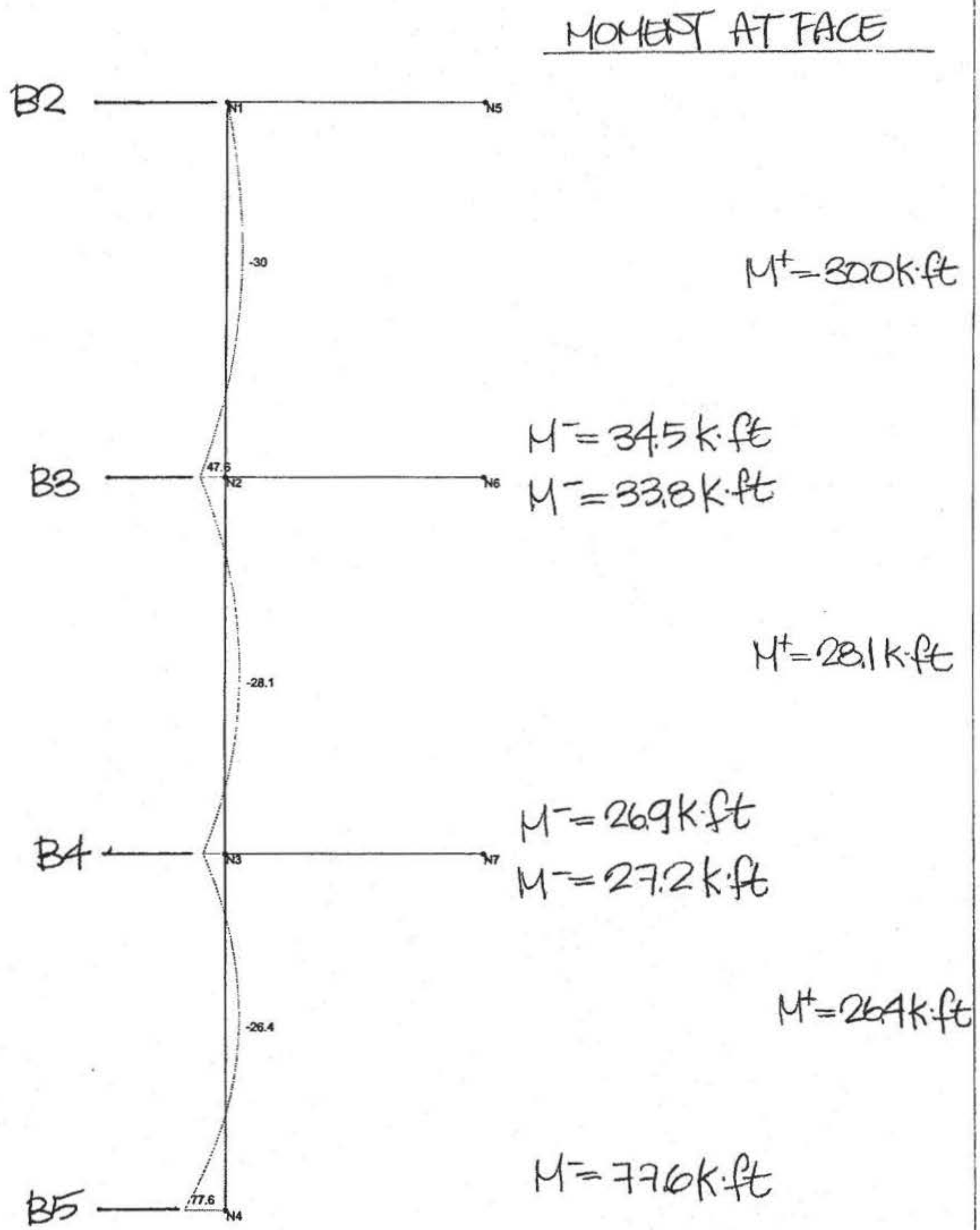


1.7 Perm Soil + 1.7 Pile Surcharge

Results for LC 5, 1.7 Perm
Member z Bending Moments (k-ft)

DeSimone Consulting Eng...	301 Mission Street Middle Foundation Wall	
ML		Mar 10, 2005 at 1:32 PM
4069		4069-20050127-MKL-Middle-Fdn-...

7.2-10



1.6Seismic Soil +1.4Seismic Increment +1.2Pile Surcharge

Results for LC 6, Seismic Combo
Member z Bending Moments (k-ft)

DeSimone Consulting Eng...	301 Mission Street Middle Foundation Wall	
ML		Mar 10, 2005 at 1:33 PM
4069		4069-20050127-MKL-Middle-Fdn-...