THE Borgata
ATLANTIC CITY

TECHNICAL ASSIGNMENT ONE

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Structural Option
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Executive Summary

Presented in this report is a preliminary description and analysis of the existing structural system of the Borgata Hotel Tower in Atlantic City, NJ. The building primarily serves as a hotel for the adjoined low rise casino. The tower's first two floors are dedicated for casino use. Floors 3 through 43 are used solely for guest rooms. The roof supports the majority of the mechanical equipment as well as a catwalk system used to access the equipment.

A grid of concrete columns and shear walls support the gravity load. The lateral load is assumed solely by the shear walls. Gravity and seismic loads are then dissipated to the earth through a deep foundation utilizing deep piles. Post-tensioning was used in the concrete floor system to minimize the depth of slabs.

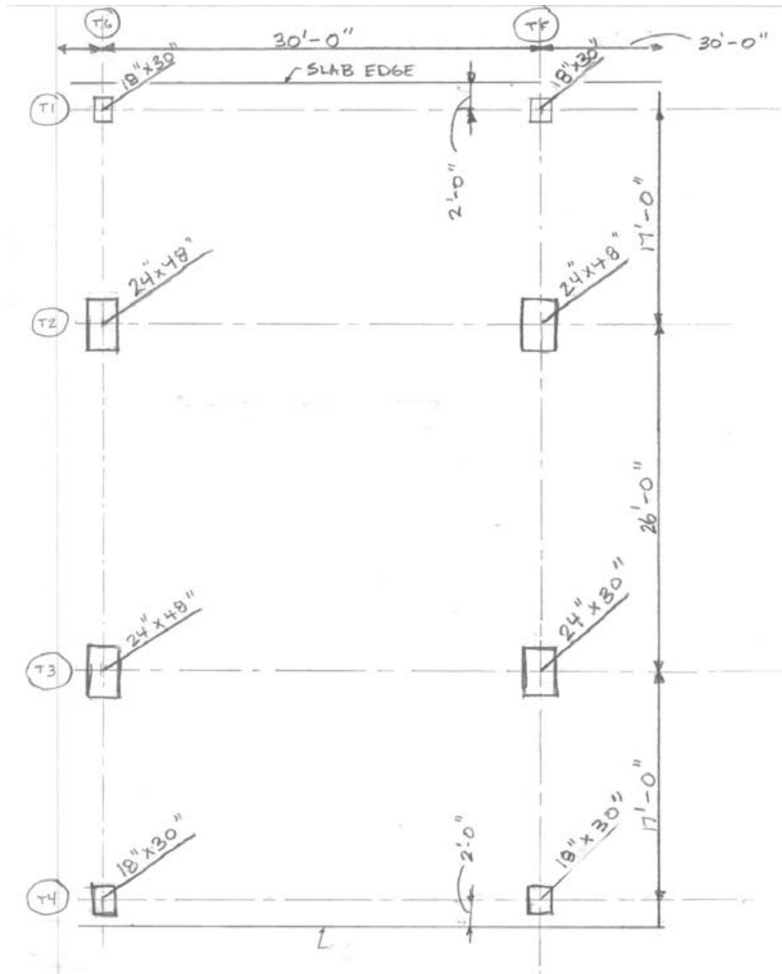
Seismic loads were calculated traditionally using The Equivalent Lateral Force Procedure outlined in Chapter 12 - ASCE 7-05. Wind loads were calculated using Method 2 – Analytical Procedure outlined in Chapter 6 – ASCE 7-05.

The Borgata Hotel was designed using the New Jersey Uniform Construction Code, 1997 and the 1996 BOCA Building Code. The preliminary analysis was done using ASCE 7-05. Spot check calculations were made to verify the existing conditions. Spot checks of a typical column, shear wall and two-way slab panel were calculated in this report.

Structural System

Floor System:

The typical floor is supported by a post-tensioned concrete slab system. The concrete is normal weight (145 pcf dry unit weight) and has a minimum strength of 5000 psi. The slab is 7" thick at the center of the building, and 8 ½" thick at each end where the floor plan is circular in shape. The typical bay sizes are 30'-0" X 26'-0" and 30'-0" X 17'-0". There is variation in span sizes at the ends of the building. Post tensioned cables are to conform to ASTM A-416 and shall be Grade A or Grade B and are loaded with varying forces from 50 to 900 kips. The non typical floors are a mix of post-tensioned systems with a thicker slab, and two way flat slabs with drop panels. The figure to the right shows the typical bay sizes along the building. A full typical floor plan can be found in the appendix.



Roof System:

The flat roof slab is similar to the typical floor slab. It is a post-tensioned system, but the slab is 8 ½" thick for the entire slab. The roof slab supports most of the buildings mechanical equipment as well as catwalks used to access the mechanical equipment.

Lateral System:

The structure is laterally supported by reinforced high strength concrete shear walls in both the North-South and East-West directions. The shear walls also assume gravity load from the floors. The concrete is normal weight and has a minimum strength of 9000psi. Most of the shear walls extend the full height of the building, but a few stop at certain stories because of smaller shears towards the top of the building. The layout of the shear walls can be seen on the typical floor plan in the appendix.

Foundation:

The Borgata Hotel is located on the site of a former landfill. The dump was not excavated and the soil below the dump is a combination of marine tidal marsh and clay/sand seams. A deep foundation system was chosen for the building. The transfers gravity and lateral loads to the earth through concrete filled steel tube piles. The piles are 16" in diameter and contain reinforced concrete. Piles are driven to various depths until reaching very dense sand. Columns bear directly on pile caps which vary in size. In some cases at shear walls, the walls and columns bear on 9'-0" concrete pile mats. The slab on grade is a 1'-6" thick structural two-way slab. This slab spans between piles caps since the soil below (landfill) has no bearing capacity.

Columns:

Columns are cast-in-place concrete with strengths that vary depending on stories. Below, table one contains the column concrete strengths for the various stories. The figure to the right shows the typical column sizes and common reinforcing arrangements.

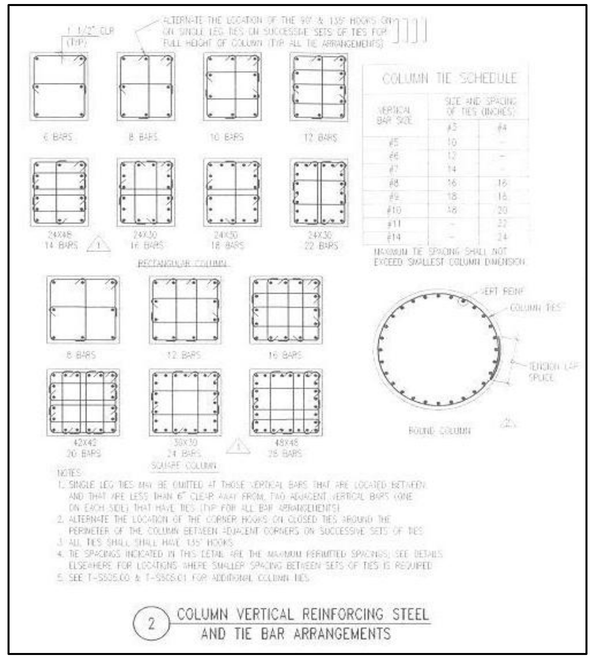


Table 1

Stories	f'c	Time
Level B -12	9000 psi	@56 days
Level 12 – 23	7000 psi	@56 days
Level 23 and up	5000 psi	@28 days

Codes

Codes Used for Original Design

- BOCA National Building Code – 1996, Building Officials and Code Administrators, Inc.
- New Jersey Uniform Construction Code – 1997
- Minimum Design Loads For Buildings and Other Structures (ANSI/ASCE 7-95 – 1996), American Society Of Civil Engineers
- Building Code Requirements For Reinforced Concrete, ACI 318-99, American Concrete Institute
- ACI Manual Of Concrete Practice – Parts 1 Through 5 – 1999
- Manual Of Standard Practice, Concrete Reinforcing Institute
- PCI Design Handbook – Precast and Prestressed Concrete, Third Edition, Prestressed Concrete Institute
- Post Tensioning Manual, Fifth Edition, Post Tensioning Institute
- Manual Of Steel Construction – Load and Resistance Factor Design, Second Edition, 1994, America Institute Of Steel Construction
- Manual Of Steel Construction, Volume II Connections, ASD 9th Edition/LRFD 1st Edition, American Institute Of Steel Construction
- Detailing For Steel Construction, American Institute Of Steel Construction
- Structural Welding Code ANSI/AWS D1.1-94, American Welding Society
- Specification For The Design Of Cold-Formed Steel Structural Members, American Iron And Steel Institute

Codes Used For Thesis

- ASCE 7-05, Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers
- ACI 318-05, Building Code Requirements For Reinforced Concrete, American Concrete institute

Gravity Loads

Live Loads:

Public Floors (Casino)

- Floor – 100 psf
- Corridors – 100 psf
- Exits/Stairs – 100 psf

Guest Floors (Hotel)

- Rooms – 40 psf * (Use of live load reduction factor for certain floors allowed)
- Corridors – 40 psf

Mechanical Rooms

- Basement – 150 psf

Roof

- Mechanical Allowance – 150 psf
- Snow/Live – 30 psf

Dead Loads

Level 1

- Slab – 194psf
- Superimposed
    - Finishes – 10 psf
    - MEP – 10 psf
    - Interior Partition Walls – 15 psf

Level 2

- Slab – 145psf
- Superimposed – Same as level 1

Level 3

- Slab – 104 psf
- Superimposed – Same as level 1

Levels 4 – 43

- Slab – 85, 103 psf   *slab thickness varies
- Superimposed – Same as level 1

Roof

- Slab – 103psf
- Mechanical Equipment – Allowance in Live Load

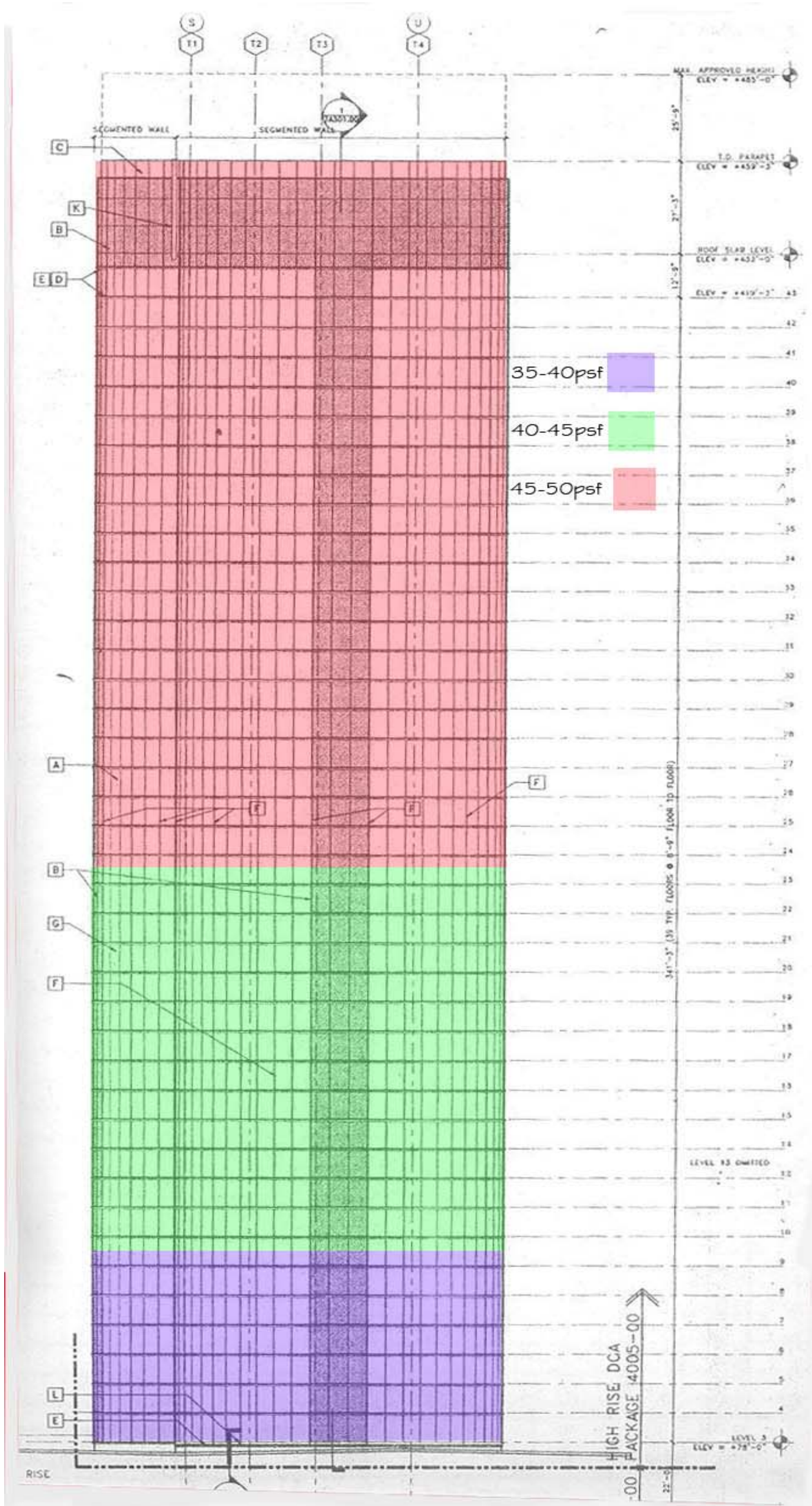
Analysis

Wind Loading

Wind load analysis was performed using Method 2 – Analytical Procedure, outlined in Chapter 6 – ASCE 7-05.The table to the right lists the design criteria summary.

Design Criteria Summary	
V =	120mph
Kd =	0.85
I =	1.0
Occupancy	2
Exposure	B
Kzt =	1.0
Gf =	0.82
GCpi	±0.18
Cp, windward	0.8
Cp, Leeward	-0.47

The building was assumed to act as a cantilever and the base shear was the summation of the forces at each story. For ease of calculation, the building was assumed to be a rectangle with dimensions 510'-0" x 107'-0". The building is classified as a "dynamically sensitive structure" because calculation of the approximate period, using equation 12.8-7 of ASCE 7-05, shows the period is larger than 1 second. The period is approximately 1.94 seconds. Since the building is classified as "dynamically sensitive", the Gust Effect Factor was calculated according to section 6.5.8.2. During calculation of the Resonant Response Factor (ASCE 7-05 Equation 6-10), the critical damping ratio, β, was assumed as 0.05, or 5 percent of critical. Calculation of wind loads yielded base shears of 9858 kips transverse to the long side of the building, and 2068 kips transverse to the short side of the building. The figure below illustrates the different wind pressure zones of the building. All hand calculations and spreadsheets for the wind loads can be found in the appendix.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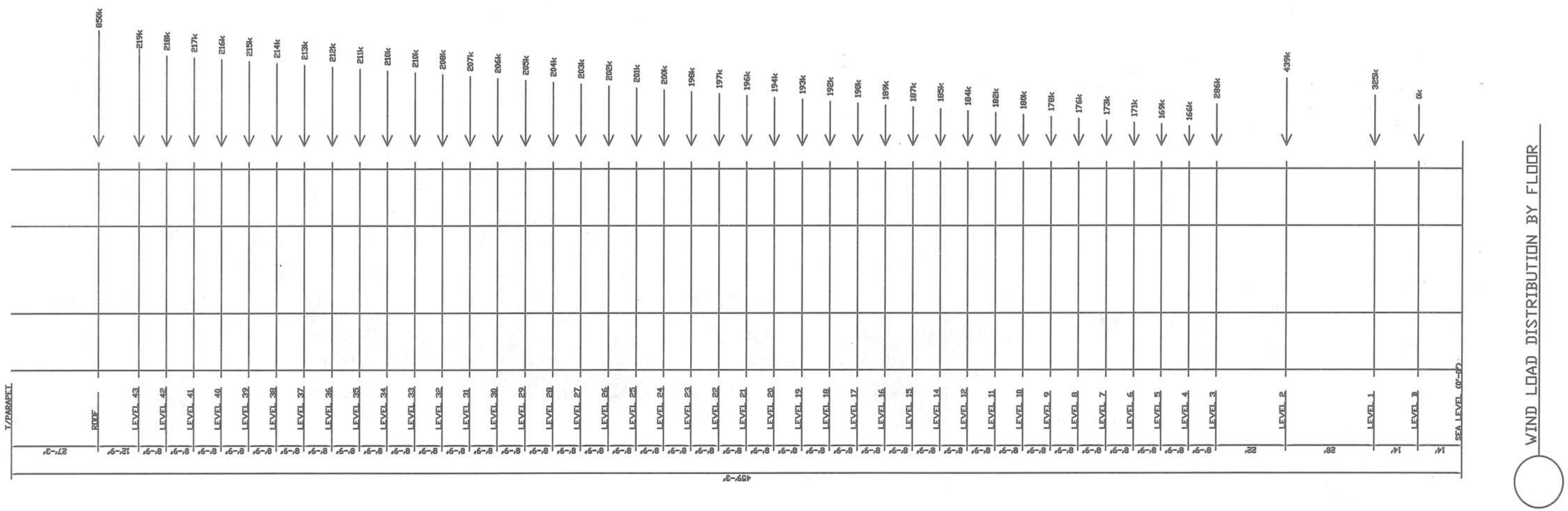
HIGH RISE DCA
PACKAGE 4005-00

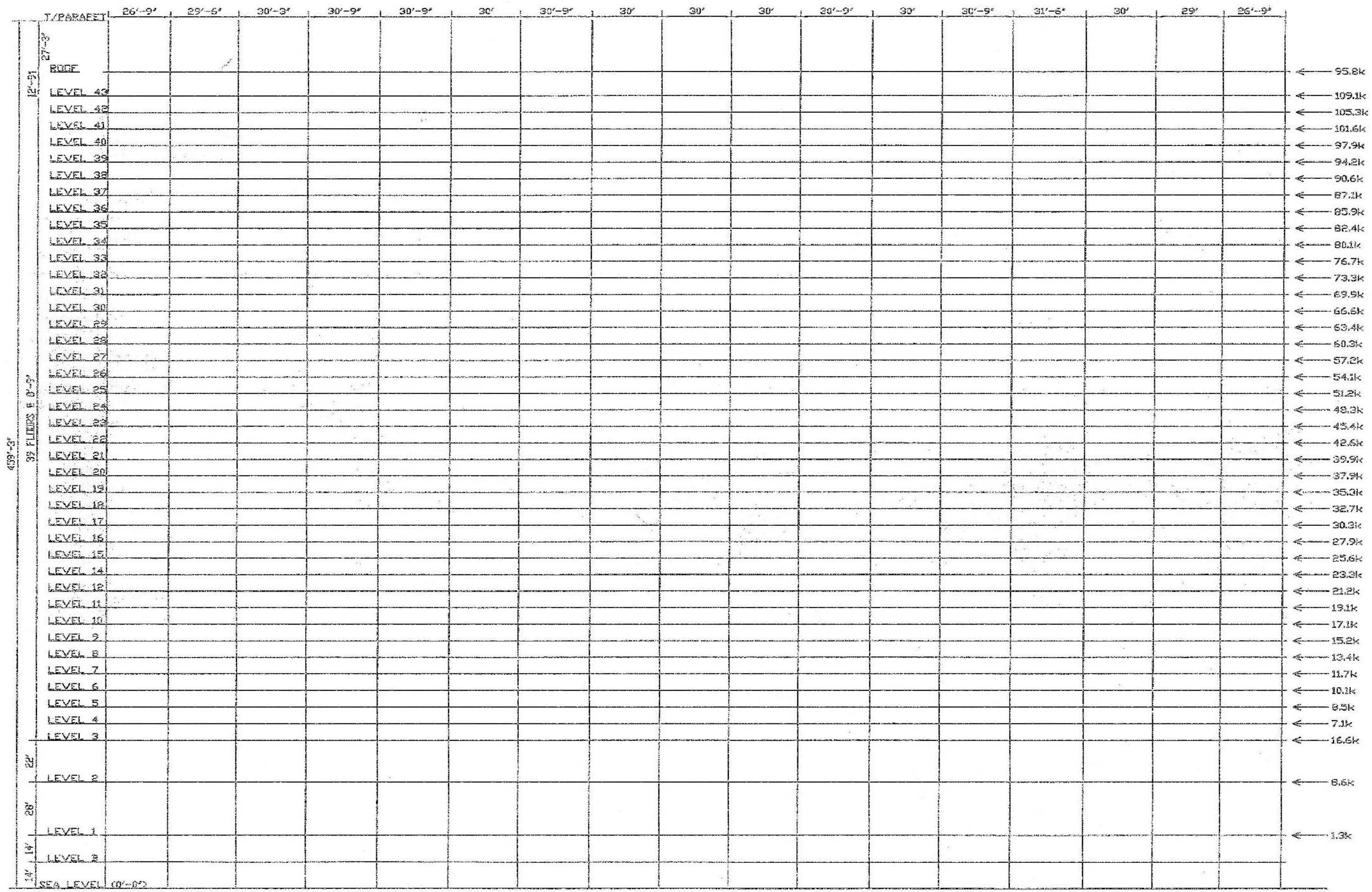

WIND LOAD DISTRIBUTION BY FLOOR

Seismic Loading

Seismic loads were calculated using the Equivalent Lateral Force Procedure outlined in Chapter 12 – ASCE 7-05. The Table to the right lists the design criteria used in the calculations.

The effective seismic weight of the building was calculated according to Section 12.7.2 ASCE 7-05. Only dead load of the building was included in the calculation because no live load satisfied the criteria specified in the code. The dead loads used were the summation of the slabs, columns, shear walls, the exterior cladding and the estimated superimposed dead loads. The design base shear was distributed vertically through the building using Equations 12.8-11 and 12.8-12 from ASCE 7-05. On the next page is an illustration of the distribution of base shear by floor. All hand calculations and spreadsheets for the calculation of seismic loads can be found in the appendix.

Seismic Design Summary	
S_s	0.166
S_1	0.048
Site Class	D
F_a	1.600
F_v	2.400
T_o	0.460s
T_s	2.300s
S_a	0.166
I	1.000
SDC	III
R	4.000
R/I	4.000
$T=C_t h_n^x$	1.939*
C_t	0.020
x	0.750
Cs	0.010
Vs	2142k



SEISMIC LOAD DISTRIBUTION BY FLOOR

Lateral Analysis Conclusion

Calculations of the wind and seismic loads have shown that the lateral system is controlled by wind loads in the North-South direction and seismic loads in the East-West direction. The wind load will control in both directions once a 1.6 LRFD Load Factor is applied, but good judgment rules that serviceability will be more critical than strength. The base shear in the North-South direction is 9592 kips. The base shear in the East-West direction is 2142. This difference in magnitude for the base shears is due to the long and narrow design of the building. This can also be seen from the layout of shear walls, with 9 walls in the North-South direction, and only 2 walls in the East-West direction.

Lateral Load Distribution – Horizontal

All shear walls are composed of 9000 psi concrete and most are the same height. For simplification, lateral loads are assumed to be distributed to shear walls by the area of concrete in shear per wall versus the total area of concrete in shear. More precise calculation of relative stiffness will completed in later technical assignments for the distribution of the loads. The distribution of lateral loads to shear walls can be found on the typical floor plan in the appendix.

Spot Checks

Spot checks were done to verify a few typical structural elements. All of the spot checks show that the members are adequate for the loading.

In the check of the column though, it was found using the estimated loads, that the column only needed the minimum amount of reinforcing required by code whereas the original design calls for about two times the amount of reinforcing steel. This could be due to inaccuracies in the estimation of superimposed dead loads, or the fact that no construction loads were used.

The two-way slab flexural reinforcement calculation showed the required reinforcement was about half of what the original design called to be used. This could be due to inaccuracy in estimating superimposed dead loads. The calculations were for the casino level 2. A live load of 100 psf was used, and a superimposed dead load of 35 psf was used.

The shear wall spot check showed that the high strength concrete alone was enough to resist the lateral load. The longitudinal and transverse shear reinforcing was then calculated as the code required minimum. It was also found that the presence of a Boundary Element was necessary to resist the compressive force created by the moment in the shear wall. This boundary element contained nearly the same amount of reinforcing as calculated by the design engineer. For simplicity of the preliminary design, no gravity load was used in this calculation.

Column Tk-T1

GRAVITY LOAD ANALYSIS 1/3

COLUMN 24-TK, 5-T1

$A_T = 30' \times \frac{1}{2} 17' = 255$ SF/FLOOR

CHECK @ FLOOR 25

@ FLOOR 3

@FLOOR 25

- SUPPORTS - 18 GUEST FLOORS
  - ROOF

LIVE LOAD: 40psf (GUEST FLOORS) ASSUME 40PSF FOR STEEL CATWALKS ON ROOF

DEAD LOAD:

FLOORS:
- SLAB: $\frac{7}{12} \cdot 145 \cdot 255/1000 = 21.6^K$
- SUPERIMPOSED: 30PSF · 255SF = 7.7 KIPS ← * SUPER IMPOSED GIVEN IN DEAD LOAD CALCULATIONS FOR EFFECTIVE SEISMIC WEIGHT
- WALL: $[30' \cdot 8.75' \cdot 18 \text{ Flrs} + 30' \cdot 27']\, 15\text{PSF} = 83^{Kips}$

MECH EQUIPTMENT:

ROOF SLAB: $\frac{8.5}{12} \cdot 145 \cdot 255/1000 = 26.2^K$

CATWALKS: 20PSF FOR MISC. STL FRAMING & MTL GRATING

LIVE LOAD REDUCTION

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) = L_o\left(0.25 + \frac{15}{\sqrt{2(255)(18 \text{ flrs})}}\right) = 0.41\, L_o = 16.4 \text{ psf}$$

LOAD ON COLUMN

LIVE: $18 \text{ flrs} \times 16.4 \text{ psf/flr} \times 255 \text{ sf} + 255 \text{ sf} \times 40 \text{ psf} = 85.5^K$

DEAD: $18 \text{ flrs} \times (21.6^K + 7.7^K) + 26.2^K + 20(255) + 83 = 641.1^K$

$P_U =$
- $1.2(641.1) + 1.6(85.5) = 906.1^K$ *
- most: $1.4(641.1) = 897.5^K$

$P_U = 906.1$ Kips

@floor 3 - SAME LOADING

$$L = 40\left(0.25 + \frac{15}{\sqrt{2(255)(40)}}\right) = 0.4(40) = 16 \text{PSF}$$

LIVE: $40 \cdot 16 \cdot 255 + 255 \cdot 40 = 173.4^K$

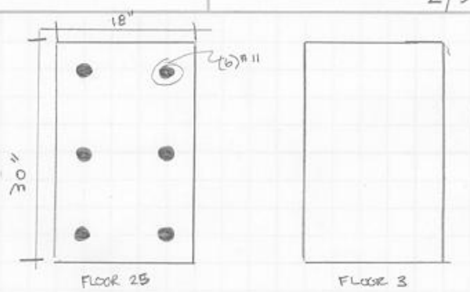
DEAD: $40 \cdot (21.6 + 7.7) + 26.2 + \frac{[(30)(8.75)(40) + 30(27)]}{1000} \cdot 15 + 20(255) = 1373^{KIPS}$

$P_U = 1.2(1373) + 1.6(173.4) = 1925^K$ *

$= 1.4(1373) = 1922.2$

GRAVITY LOAD ANALYSIS 2/3

COLUMN TK-T1
18" x 30"
$A_G = 540 in^2$
@ FLR 25 $f'_c = 5000 psi$
$P_u = 906.1 kips$
@ FLR 3 $f'_c = 9000 psi$
$P_u = 1925 kips$



UNBRACED LENGTH: 8'-9" FOR GUEST LEVELS

ASSUME COLUMN ACTS AS SHORT COLUMN W/ NO SLENDERNESS EFFECTS

@ FLOOR 25 $P_u \leq \phi P_n = 906 kips$

$906 kips = \phi P_{n_{max}} = 0.80\phi [0.85 f'_c (A_g - A_{st}) + f_y A_{st}]$

$906000 = 0.80(0.65)[0.85(5000)(18 \cdot 30 - A_{st}) + 60000 A_{st}]$
$-287400 = 30940 A_{st}$ $A_{st} = -9.3 in^2$

* WHAT IS MIN REQ'D AMOUNT OF STEEL BY CODE

$0.01 < \rho_{st} < 0.08$

$0.015 \cdot A_g = 0.015(18)(30) = 8.1 in^2$
TRY (6) #11 BARS $= 6(\frac{11}{16})^2 \pi = 8.91 in^2$
$\rho = \frac{8.91}{18 \times 30} = 0.0165 > 0.01$ OK

$\phi P_n = 0.80(0.65)[0.85(5000)(18 \times 30 - 8.91) + 60000(8.91)] = 1454$ Kips >> 906 OK

TIES: USE #4 TIES FOR ≥ #11

SPACING (least of):
- $16 D_b = 16 \cdot 11/8 = 22"$
- $48 \cdot 4/8 = 24"$
- LEAST DIM. COL = (18")

18x30 $f'_c = 5000$ (6) #11 BARS w/ #4 TIES @ 18"

GRAVITY LOAD ANALYSIS 3/3

@ FLOOR 3

$$P_u \leq \phi P_n = 0.8\phi[0.85f'_c(A_g - A_{st}) + A_{st}f_y]$$ ← SOLVE A_{st}

$$1925K = 0.8(0.65)[0.85(9\,ksi)(18\cdot30 - A_{st}) + 60A_{st}]$$

$-223K = 27.22A_{st}$ $\quad A_{st} = -8.2\,in^2$

USE MIN REQ'D STL

USE (6) #11 BARS w/ #4 TIES @ 18" O.C.

AS BUILT

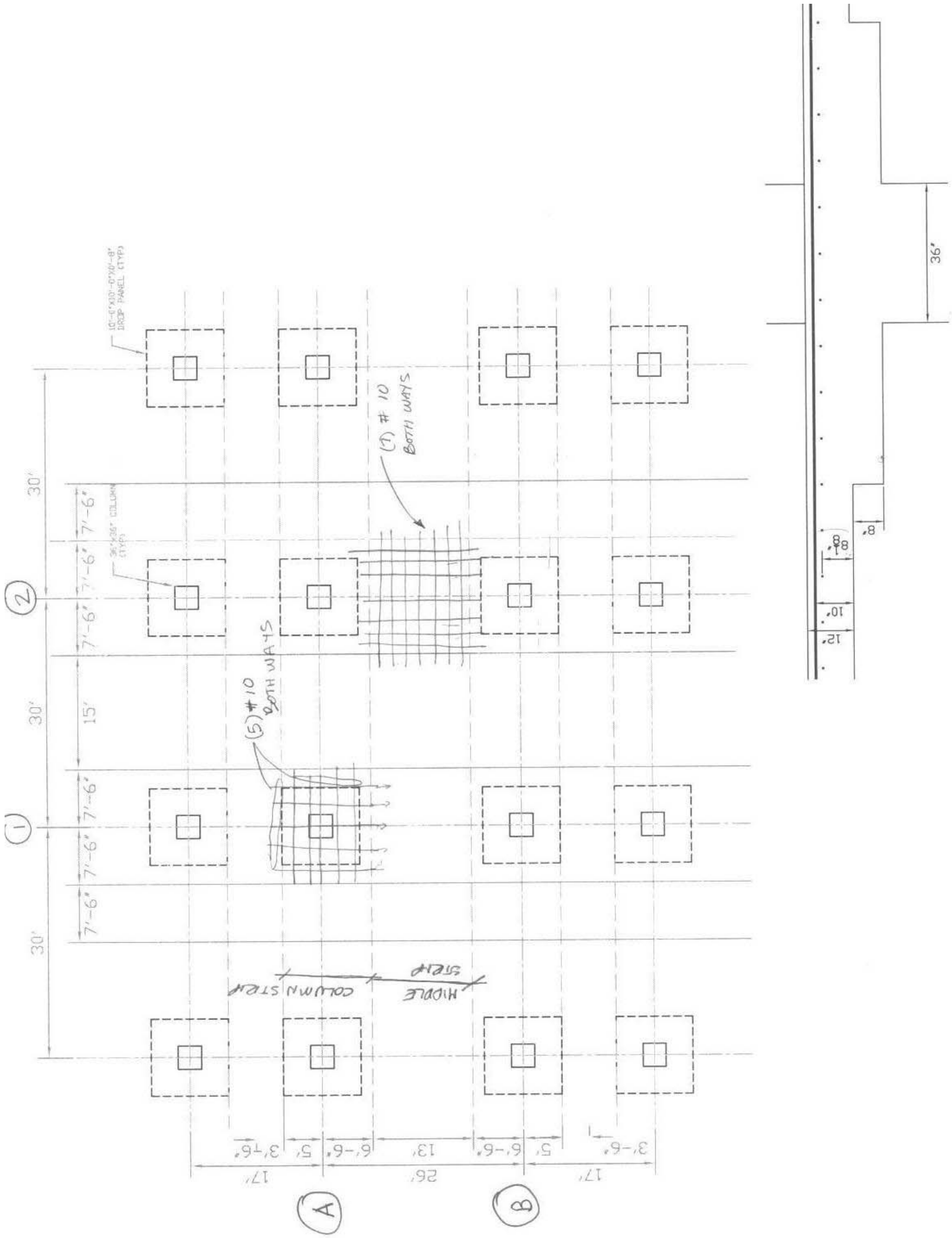
@ FLOOR 25 10 #10 BARS

@ FLOOR 3 12 # 10 BARS

CONCLUSION:

AT THIS TIME, I AM UNAWARE OF POSSIBLE MECHANICAL LOADS ON THE ROOF AS WELL AS POSSIBLE CONSTRUCTION LOADS THAT COULD PRESENT HIGHER LOADS.

Second Floor Slab



Slab Flexural Reinforcement Calculations

TECH 1 — 1/1

TWO WAY SLAB DESIGN — $W_D = 145 + 35$, $W_L = 100\text{psf}$

FRAME A

$$M_0 = \frac{W_u l_2 \cdot l_n^2}{8} = \frac{[1.2(180)+1.6(100)] \times 21.5 \times (27)^2}{8} = 737 \text{ kft}$$

SINCE LOOKING @ INT SPAN

$M^- = 0.65 M_0 = 0.65(737) = 479 \text{ kft}$

$M^+ = 0.35 M_0 = 0.35(737) = 258 \text{ kft}$

PERCENT TO COLUMN STRIP / MIDDLE STRIP

$\alpha = 1.0 \quad \alpha \frac{l_2}{l_1} = 1.0 \frac{27}{30} = 0.9$

INTERPOLATE

$\left(\frac{0.9-0}{1-0}\right)(90-75)+75 = 88.5$

$\frac{0.9}{1}(88.5-75)+75 = 87.2$

$M_{CS} = 0.872(479) = 418 \text{ kft}$ — $\frac{M_U}{\Phi} = M_n =$

$M_{MS} = 0.128(479) = 61 \text{ kft}$

DESIGN STEEL

CS: $R = \frac{418}{(0.9)4(18.625)} \approx 75$ — $\left(\frac{75-60}{89-60}\right) \times (0.0015-0.001) + 0.001 = 0.0013$

$A_s = \rho b d = 0.0013(120 \times 18.625 + 18 \times 10.625) = 3.4 \text{ in}^2$

$A_{s\,min} = 0.002 bt = 0.002 \times (120 \cdot 20 + 18 \times 12) = 5.23 \text{ in}^2$ — SPACING: $\frac{12 \times 13.5}{}$

$N = \frac{3.4}{1.23} = 3 \text{ BARS}$ — USE 5 #10 BARS

$N_{min} = \frac{120}{2 \cdot 20} + \frac{18}{2 \cdot 12} = 4$

MS: $R = \frac{61(12)}{0.9(4)(8.625)} \approx 24 \quad \rho = 0.0005$

$A_s = 0.0005(156 \cdot 10.625) = 0.83 \text{ in}$

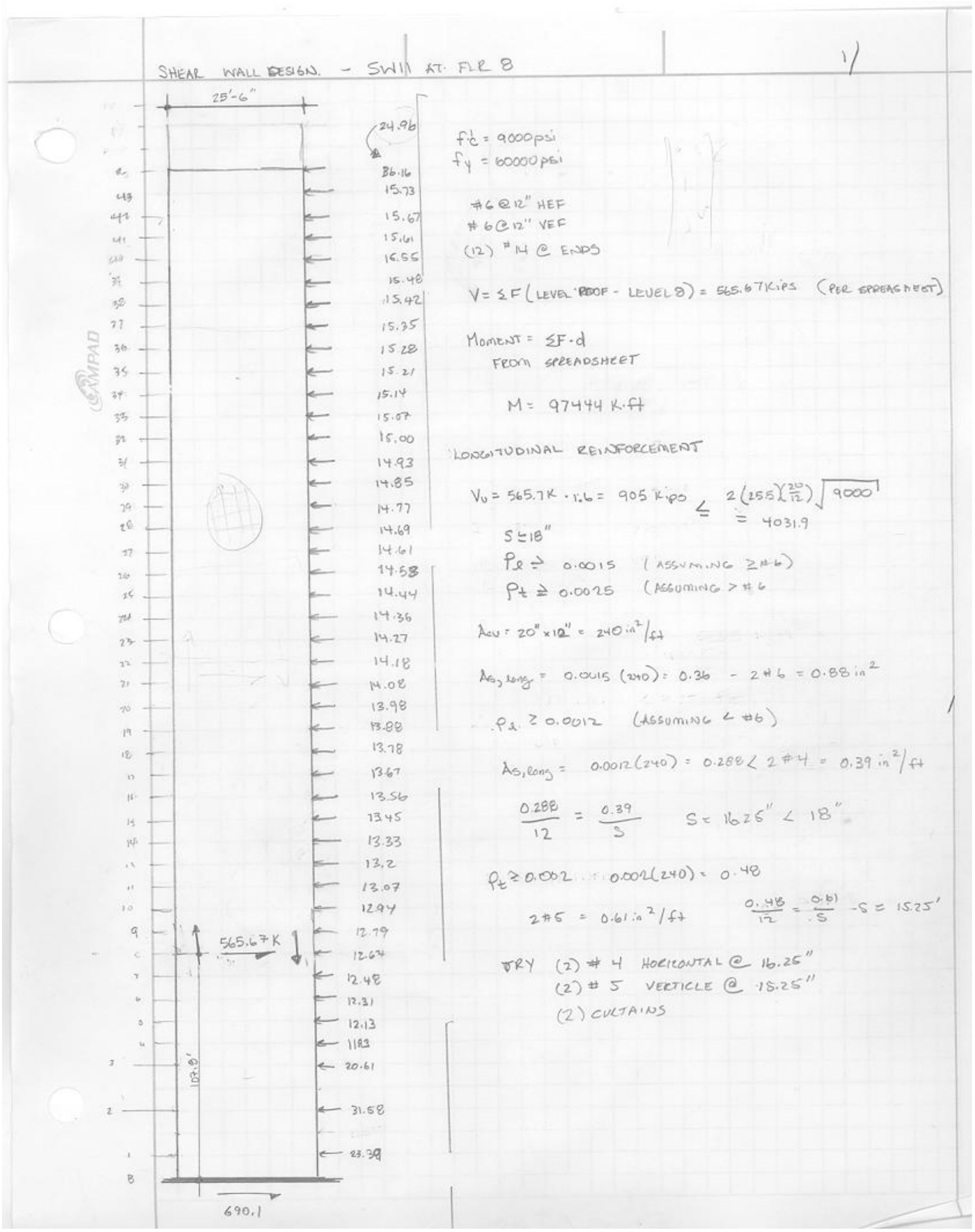
$A_{s\,min} = 0.002(152)(12) = 3.75 \text{ in}^2$

$N = \frac{.83}{1.23} = 1$

$N_{min} = \frac{156}{2(12)} = 6.5$ — USE (7) #10'S, $A_s = 8.61 \text{ in}$

* FOR SIMPLICITY, USE THIS STEEL ARRANGEMENT IN BOTH DIRECTIONS. CONSERVATIVE BECAUSE SPANS IN FRAME A ARE GREATER THAN IN FRAME 1.

Shear Wall



SHEAR WALL DESIGN. – SW11 AT FLR 8

f'c = 9000 psi
fy = 60000 psi

#6 @ 12" HEF
#6 @ 12" VEF
(12) #14 @ ENDS

V = ΣF (LEVEL ROOF – LEVEL 8) = 565.67 Kips (PER SPREADSHEET)

MOMENT = ΣF·d
FROM SPREADSHEET

M = 97444 K·ft

LONGITUDINAL REINFORCEMENT

$V_u = 565.7K \cdot 1.6 = 905$ Kips $\leq 2(255)(\frac{20}{12})\sqrt{9000} = 4031.9$

$S \leq 18"$
$\rho_\ell \geq 0.0015$ (ASSUMING ≥ #6)
$\rho_t \geq 0.0025$ (ASSUMING > #6)

$A_{cv} = 20" \times 12" = 240\ in^2/ft$

$A_{s,long} = 0.0015(240) = 0.36 - 2\#6 = 0.88\ in^2$

$\rho_\ell \geq 0.0012$ (ASSUMING < #6)

$A_{s,long} = 0.0012(240) = 0.288 < 2\#4 = 0.39\ in^2/ft$

$\frac{0.288}{12} = \frac{0.39}{S}$ $S = 16.25" < 18"$

$\rho_t \geq 0.002$ $0.002(240) = 0.48$

$2\#5 = 0.61\ in^2/ft$ $\frac{0.48}{12} = \frac{0.61}{S}$ $S = 15.25'$

TRY (2) #4 HORIZONTAL @ 16.25"
(2) #5 VERTICLE @ 15.25"
(2) CURTAINS

SHEAR WALL DSGN. SW11 @ FLR 8 2/

NOMINAL SHEAR CAPACITY

$V_n = A_{cv}(\alpha_c \sqrt{f'_c} + \rho_t f_y)$

$\frac{h_w}{l_w} = \frac{445}{25.5} = 17.5 > 2 \quad \therefore \alpha_c = 2.0$

$A_{cv} = 20'' \cdot 25.5' \cdot \frac{12''}{1'} = 6120 in^2 \qquad \rho_t = \frac{0.61 in^2}{15 \cdot 20} = 0.00203$

$V_n = 6120(2\sqrt{9000} + 0.00203(60000)) = 1906.6$ kips

$\phi V_n = 0.6 V_n = 0.6(1906.6) = 1144$ kips > 905 ok.

BOUNDARY ELEMENT REQUIRED?

$f_c > 0.2f'_c$ – BOUNDARY ELEMENT IS NEEDED

FOR NOW, NEGLECT DEAD LOAD

$M_u = 1.6(97444) = 155910$ k·ft

$A_g = (20/12)(25.5) = 42.5^2 \qquad f_c = \frac{155910 \cdot \frac{25.5}{2}}{2303} = 863.2 k/ft^2 = 6 ksi > 0.2(9000) = 1.8 ksi$

$I = \frac{(\frac{20}{12})(25.5)^3}{12} = 2303 ft^3$

* A BOUNDARY ELEMENT IS NECESSARY

$\phi P_n = 0.8\phi(0.85f'_c(A_g - A_{st})) + F_y A_{st}$

ASSUME SIZE 20x40"

$P_u = \frac{155910}{25.5} = 6114.1$ kips

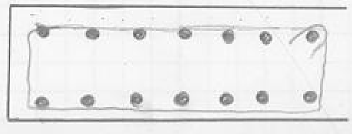
$6114.1 = 0.8(0.9)[0.85(9000)(20 \cdot 40 - A_{st}) + 60000 A_{st}]$

$6114100 = 4406400 + 37692 A_{st} \qquad A_{st} = 45.3 in^2$ ← TO HIGH

TRY 20x45 SIZE

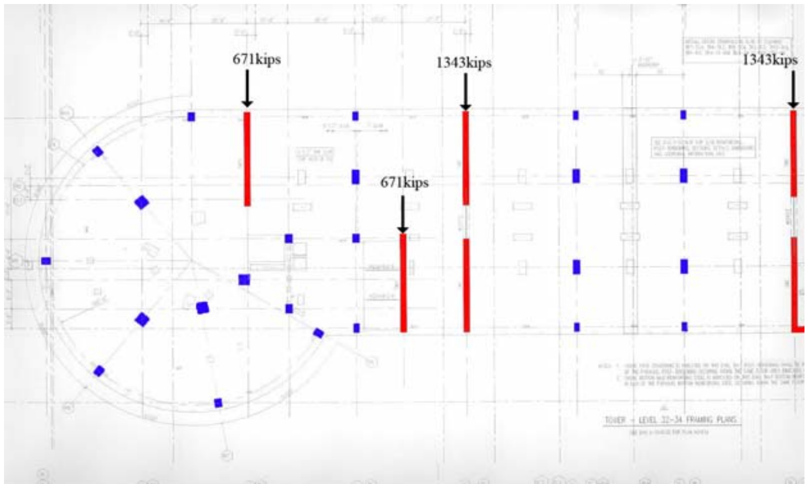
$6114100 = 0.8(0.9)\{[0.85(9000)(20 \cdot 45) - A_{st}] + 60000 A_{st}\}$

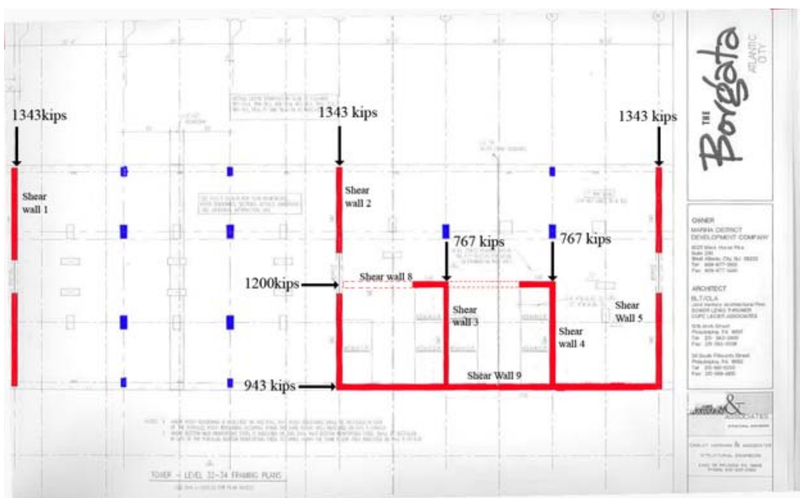
$1156900 = 37692 A_{st} \qquad A_{st} = 30.7 in^2 \rightarrow 12.76 \rightarrow 14$ # 14 BARS

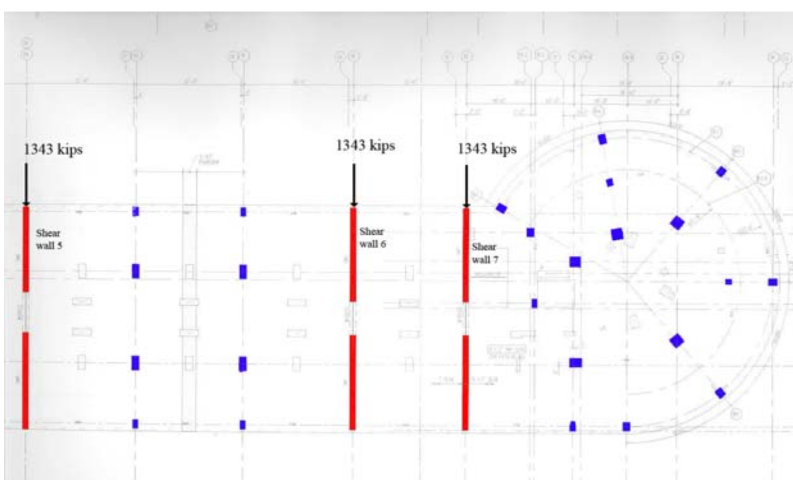


APPENDIX

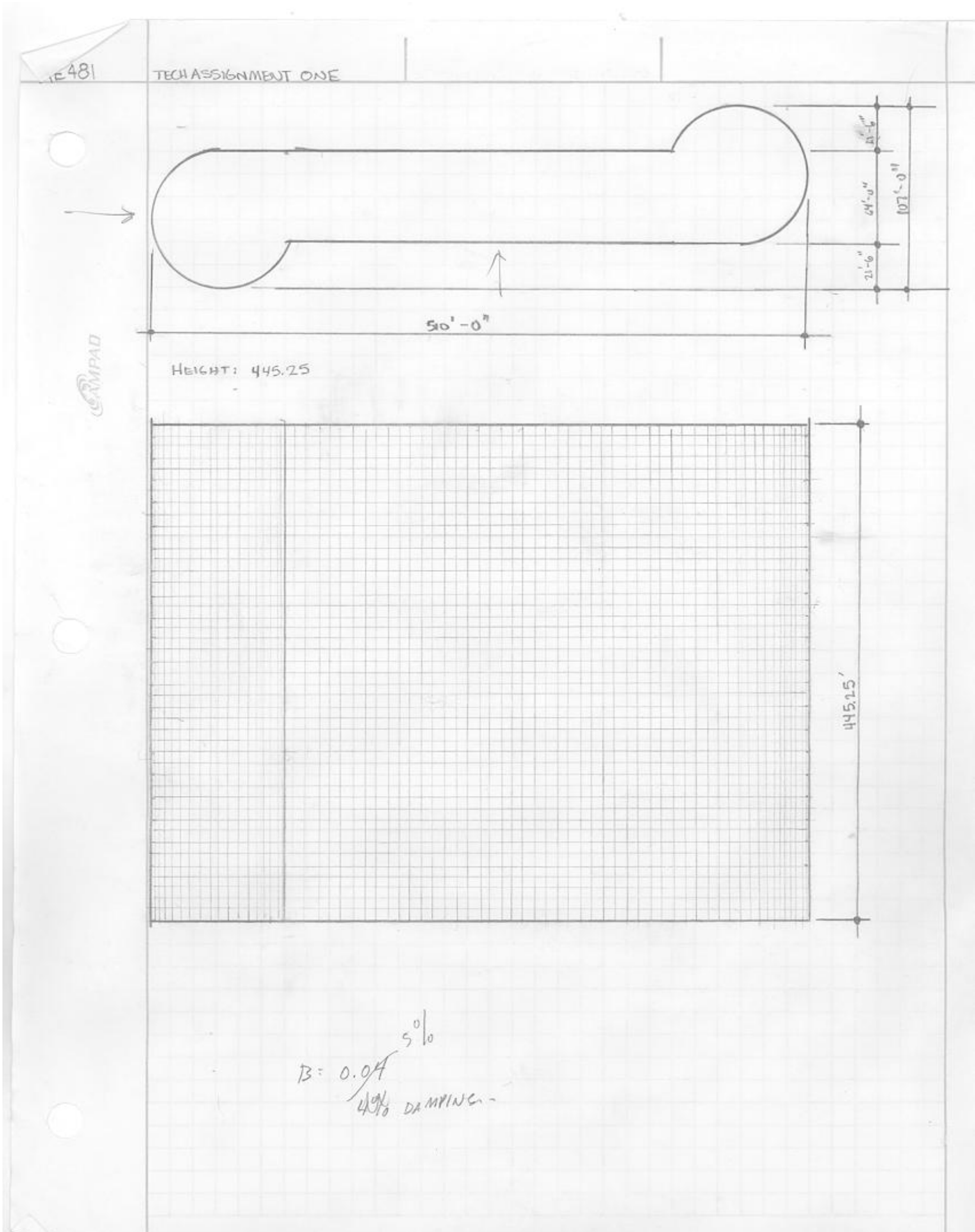
Typical Floor Plan







Wind Load Calculations



AE 481 TECH ASSIGNMENT ONE

WIND LOAD

1. BASIC WIND SPEED, V : 120MPH
   WIND DIRECTIONALITY FACTOR Kd. = 0.85

2. IMPORTANCE FACTOR, I: 1.15
   OCCUPANCY CATEGORY: III, MORE THAN 300 PEOPLE CONGREGATE IN ONE AREA

3. EXPOSURE CATEGORY : B
   SURFACE ROUGHNESS CATEGORY B
   VELOCITY PRESSURE COEFFICIENTS - SEE SPREADSHEET

4. TOPOGRAPHIC FACTOR, K_{zt}

5. GUST EFFECT FACTOR G OR Gf

$$G = 0.925\left[\frac{(1+1.7 g_Q I_{\bar{z}} Q)}{1+1.7 g_v \cdot I_{\bar{z}}}\right]$$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.2\left(\frac{33}{267}\right)^{1/6} = 0.141$$

$$g_Q = g_z = 3.4$$

$I_{\bar{z}}$ =

$\bar{z}$ = { 0.6h = 0.6(445.25) = 267.15 ; greater ; 15ft

c = 0.20

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{510+445}{759.7}\right)^{0.63}}} = 0.761$$

OR

$$\sqrt{\frac{1}{1+0.63\left(\frac{107+445}{759.7}\right)^{0.63}}} = 0.812$$

B = 510ft, 107ft
h = 445ft

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 500\left(\frac{267.15}{33}\right)^{\frac{1}{5.0}} = 759.7$$

ℓ = 500ft
$\bar{\epsilon}$ = 1/5.0

(TRANSVERSE TO LONG DIRECTION)

$$G = 0.925\left[\frac{(1+1.7(3.4)(267.15)(0.761))}{1+1.7(3.4)(267.15)}\right] = 0.704$$

COMPARE TO 0.85
17% DIFF.

(TRANSVERSE TO SHORT DIRECTION)

$$G = 0.925\left[\frac{1+1.7(3.4)(267.15)(0.812)}{1+1.7(3.4)(267.15)}\right] = 0.751$$

COMPARE TO 0.85
11.6% DIFF

AE481 TECH ASSIGNMENT ONE CHRIS SHIPPER

5. CONTINUED...

DYNAMICALLY SENSITIVE? FUNDAMENTAL FREQUENCY < 1HZ?

$T_a = C_t h_n^x$

$C_t = 0.02$

$x = 0.75$

$T_a = 0.02(445.25)^{0.75} = 1.939$

$f = 0.516 < 1.0$

$$G_f = 0.925\left(\frac{1+1.7 I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1+1.7 g_v I_{\bar{z}}}\right)$$

$g_Q = g_v = 3.4$

$$g_R = \sqrt{2\ln(3600 n_1)} + \frac{0.577}{\sqrt{2\ln(3600 n_1)}}$$

$$= \sqrt{2\ln(3600\cdot 1.939)} + \frac{0.577}{\sqrt{2\ln(3600\cdot 1.939)}} = 4.344$$

$$R = \sqrt{\frac{1}{\beta}\cdot R_n R_h R_B (0.53 + 0.47 R_L)}$$

$$R_n = \frac{7.47 N_1}{(1+10.3N_1)^{5/3}} = \frac{7.47(9.335)}{(1+10.3(9.335))^{5/3}} = 0.036$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{(1.939)(759.7)}{(157.8)} = 9.335$$

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} \cdot V\left(\frac{88}{60}\right) = 0.65\left(\frac{267.15}{33}\right)^{\frac{1}{6.5}} \cdot 120\left(\frac{88}{60}\right) = 157.8$$

$$R_\ell = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right)$$

$$R_h \rightarrow \eta = \frac{4.6 n_1 h}{\bar{V}_{\bar{z}}} = \frac{4.6(1.939)(445)}{157.8} = 25.16$$

$$R_h = \frac{1}{25.16} - \frac{1}{2(25.16)^2}\left(1 - e^{-2(25.16)}\right) = 0.039$$

$$R_B \rightarrow \eta = \frac{4.6 n_1 B}{\bar{V}_{\bar{z}}} = \frac{4.6(1.939)(510 \text{ OR } 107)}{157.8} = 28.83 \text{ OR } 6.05$$

$$R_B = \frac{1}{28.83} - \frac{1}{2(28.83)^2}\left(1 - e^{-2(28.83)}\right) = 0.034$$

OR

$$R_B = \frac{1}{6.05} - \frac{1}{2(6.05)^2}\left(1 - e^{-2(6.05)}\right) = 0.152$$

$$R_L \rightarrow \eta = \frac{15.4 n_1 L}{\bar{V}_{\bar{z}}} = \frac{15.4(1.939)(107 \text{ OR } 510)}{157.8} = 20.25 \text{ OR } 96.51$$

$$R_L = \frac{1}{20.25} - \frac{1}{2(20.25)^2}\left(1 - e^{-2(20.25)}\right) = 0.048$$

OR

$$R_L = \frac{1}{96.51} - \frac{1}{2(96.51)^2}\left(1 - e^{-2(96.51)}\right) = 0.0103$$

AE 481 | TECH ASSIGNMENT ONE | CHRIS SHIPPER

5. CONTINUED

ASSUME β AS 0.05 OR 5% OF CRITICAL

$$R_{long} = \sqrt{\frac{1}{0.05}\cdot(0.036)(0.039)(0.034)(0.53+0.47(0.048))} = 0.023$$

$$G_{f_{LONG}} = 0.925\left(\frac{1+1.7(0.141)\sqrt{(3.4)^2(0.761)^2+(4.344)^2\cdot(0.023)^2}}{1+1.7(3.4)(0.141)}\right) = 0.925\left(\frac{1.621}{1.81498}\right) = 0.826$$

$$\frac{0.85-0.826}{0.85}(100) = 2.8\%$$

$$R_{short} = \sqrt{\frac{1}{0.05}(0.036)(0.039)(0.152)(0.53+0.47(0.0103))} = 0.0478$$

$$G_{f_{short}} = 0.925\left[\frac{1+1.7(0.141)\sqrt{(3.4^2)(0.761)^2+(4.344)^2(0.0478)^2}}{1+1.7(3.4)(0.141)}\right] = \frac{0.925(1.6222)}{1.81498} = 0.82$$

TECH ASSIGNMENT ONE — CHRIS SHIPPER

6. ENCLOSURE CLASSIFICATION : ENCLOSED

7. INTERNAL PRESSURE COEFFICIENT, GC_{pi} : ± 0.18

8. EXTERNAL PRESSURE COEFFICIENTS, C_p (FIG. 6-6,7,8) OR GC_{pf} (FIG. 6-10), OR FORCE COEFFICIENTS, C_f

FIND C_p:

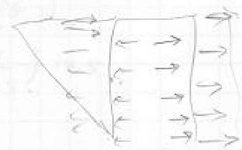
WINWARD WALL: $C_p = 0.8$

LEEWARD WALL: $C_p = ((1.14-1)/(2-1))(-0.3-0.5) + -0.3 = -0.472$

SIDE WALL $C_p = -0.7$

ROOF PRESSURE - WHO CARES

9. VELOCITY PRESSURE, q_z OR q_h



$q_z = 0.00256 K_z \cdot K_{zt} \cdot K_d \cdot V^2 I$ (lb/ft²)

K_z = SEE SPREAD SHEET

$K_{zt} = 1.0$

$K_d = 0.85$

q TABULATED IN SPREADSHEET

10. DESIGN WIND LOAD, P

$p = q \cdot G_f \cdot C_p - q_i (GC_{pi})$ ± 0.18

$q = q_z$ FOR WIND WARD WALLS @ HEIGHT z

$q = q_h$ FOR LEEWARD WALLS, SIDE WALLS, AND ROOFS, @ h

$q = q_h$ FOR WINDWARD WALLS SIDE WALLS & ROOFS OF ENCLOSED BUILDINGS AND FOR NEGATIVE INTERNAL PRESSURE EVALUATION IN PARTIALLY ENCLOSE BLDGS

WINDWARD: $p = q_z \cdot G_f \cdot C_p - q_h (GC_{pi})$

LEEWARD : $p = q_h \cdot G_f C_p -$

SIDEWALL: $p = q_h \cdot G_f C_p - q_h \cdot GC_{pi}$

FROM SPREADSHEET

TRANSVERSE TO LONG DIRECTION : $V_w = 9592^k$

TRANSVERSE TO SHORT DIRECTION : $V_w = 2012^k$

Wind Load Dist. Table

Story	Height	Kz	qz	p, windward	p, leeward	Area(long)	Force(long)	Area(short)	Force(short)
B	0.0	0.57	18.01		-18.44	0	0	0	0
1	14.0	0.57	18.01	11.91	-18.44	10710	325	2247	68
2	42.0	0.77	24.17	15.99	-18.44	12750	439	2675	92
3	64.0	0.87	27.26	18.03	-18.44	7854	286	1648	60
4	72.8	0.90	28.28	18.71	-18.44	4463	166	936	35
5	81.5	0.93	29.21	19.32	-18.44	4463	169	936	35
6	90.3	0.96	30.07	19.90	-18.44	4463	171	936	36
7	99.0	0.99	30.88	20.43	-18.44	4463	173	936	36
8	107.8	1.01	31.63	20.93	-18.44	4463	176	936	37
9	116.5	1.03	32.35	21.40	-18.44	4463	178	936	37
10	125.3	1.05	33.02	21.85	-18.44	4463	180	936	38
11	134.0	1.07	33.67	22.27	-18.44	4463	182	936	38
12	142.8	1.09	34.28	22.68	-18.44	4463	184	936	39
14	151.5	1.11	34.87	23.07	-18.44	4463	185	936	39
15	160.3	1.13	35.43	23.44	-18.44	4463	187	936	39
16	169.0	1.15	35.97	23.80	-18.44	4463	189	936	40
17	177.8	1.16	36.50	24.15	-18.44	4463	190	936	40
18	186.5	1.18	37.00	24.48	-18.44	4463	192	936	40
19	195.3	1.20	37.49	24.80	-18.44	4463	193	936	40
20	204.0	1.21	37.96	25.12	-18.44	4463	194	936	41
21	212.8	1.23	38.42	25.42	-18.44	4463	196	936	41
22	221.5	1.24	38.86	25.71	-18.44	4463	197	936	41
23	230.3	1.25	39.30	26.00	-18.44	4463	198	936	42
24	239.0	1.27	39.72	26.28	-18.44	4463	200	936	42
25	247.8	1.28	40.13	26.55	-18.44	4463	201	936	42
26	256.5	1.29	40.53	26.81	-18.44	4463	202	936	42
27	265.3	1.31	40.92	27.07	-18.44	4463	203	936	43
28	274.0	1.32	41.30	27.32	-18.44	4463	204	936	43
29	282.8	1.33	41.67	27.57	-18.44	4463	205	936	43
30	291.5	1.34	42.04	27.81	-18.44	4463	206	936	43
31	300.3	1.35	42.39	28.05	-18.44	4463	207	936	44
32	309.0	1.36	42.74	28.28	-18.44	4463	208	936	44
33	317.8	1.38	43.09	28.51	-18.44	4463	210	936	44
34	326.5	1.39	43.42	28.73	-18.44	4463	210	936	44
35	335.3	1.40	43.75	28.95	-18.44	4463	211	936	44
36	344.0	1.41	44.07	29.16	-18.44	4463	212	936	45
37	352.8	1.42	44.39	29.37	-18.44	4463	213	936	45
38	361.5	1.43	44.70	29.58	-18.44	4463	214	936	45

39	370.3	1.44	45.01	29.78	-18.44	4463	215	936	45
40	379.0	1.45	45.31	29.98	-18.44	4463	216	936	45
41	387.8	1.46	45.61	30.17	-18.44	4463	217	936	46
42	396.5	1.46	45.90	30.37	-18.44	4463	218	936	46
43	405.3	1.47	46.19	30.56	-18.44	4463	219	936	46
Roof	418.0	1.49	46.60	30.83	-18.44	10200	503	2140	105
T/Parapet	445.3	1.51	47.45	31.39	-18.44	6961	347	1460	73
						Base Shear(k)	9592	Base Shear(l)	2012

Seismic Loads

AE481 TECH ASSIGNMENT ONE

SEISMIC LOADS

$S_S = 0.166$
$S_1 = 0.048$

SITE CLASS - ASSUME SITE CLASS D FOR TIME BEING

$F_a = 1.6$
$F_v = 2.4$

$S_{MS} = F_a S_S = 1.6(0.166) = 0.266$
$S_{M1} = F_v S_1 = 2.4(0.048) = 0.115$

$C_u = 1.7$

$S_{DS} = (2/3) S_{MS} = (2/3)(0.266) = 0.177$
$S_{D1} = (2/3) S_{M1} = (2/3)(0.115) = 0.077$

$$T_0 = \frac{0.2\, S_{D1}}{S_{DS}} = \frac{0.2(0.177)}{0.077} = 0.460 < T = 1.939$$

$$T_S = \frac{S_{D1}}{S_{DS}} = \frac{0.177}{0.077} = 2.30 > T = 1.939$$

$\therefore S_a = S_{DS}$

IMPORTANCE FACTOR - CATEGORY III $\therefore$ $I = 1.25$

SEISMIC DESIGN CATEGORY B ($0.167 < S_{DS} = 0.177 < 0.33$ & OCC. CAT = III)

EQUIVALENT LATERAL FORCE PROCEDURE

$C_S =$ least of:

$$\frac{S_{DS}}{(R/I)} = \frac{0.177}{} = 0.044$$

$$\frac{S_{D1}}{T\cdot(R/I)} = \frac{0.077}{1.939(4)} = 0.0099 < 0.01 \quad \text{USE } C_S = 0.01$$

$$\frac{S_{D1} T_L}{T^2 \cdot (R/I)} = \frac{(0.077)(6)}{(1.939)^2 \cdot 4.0} = 0.031$$

$R = 4.0$ (ORDINARY REINFORCED SHEAR WALL) $(R/I) = 4.0$
$I = 1.0$

$T = C_t \cdot h_n^x = 0.02(445.25)^{0.75} = 1.939$
$C_t = 0.02$
$x = 0.75$

$C_u T_a = 1.7(1.939) = 3.30$

OR

$T_a = \frac{0.0019}{\sqrt{C_w}} h_n$ → CHECK LATER

$T_L = 6$ secs
(FIG 22:15)

$C_S = 0.01$
$V_S = C_S W = 0.01\, W = 0.01(214168 \text{ KIPS}) = 2142 \text{ KIPS}$

Effective Seismic Weight

TECH 1 | SEISMIC EFFECTIVE WEIGHT

LEVEL 1

SLAB: $\frac{16''}{12''/1} \times 145 pcf \times 35093 SF =$ 6785 Kips

COLS:

✓ (11) 54" ϕ	$\pi\left(\frac{54}{2}\right)^2 \cdot 145 \times 11 \times \frac{1}{144} \times \frac{1}{1000}$	= 25.4 K/ft
✓ (8) 18x18	$18 \times 18 \times 145 \times 8 \times \frac{1}{144} \times \frac{1}{1000}$	= 1.3 K/ft
(4) 42x42	$42 \times 42 \times 145 \times 4 \times \frac{1}{144} \times \frac{1}{1000}$	= 7.1 K/ft
(15) 36x36	$36 \times 36 \times 145 \times 15 \times \frac{1}{144} \times \frac{1}{1000}$	= 19.6 K/ft
(5) 24x30	$24 \times 30 \times 145 \times 5 \times \frac{1}{144} \times \frac{1}{1000}$	= 3.6 K/ft
✓ (18) 30x30	$30 \times 30 \times 145 \times 18 \times \frac{1}{144} \times \frac{1}{1000}$	= 16.3 K/ft
✓ (8) 48x48?	$48 \times 48 \times 145 \times 8 \times \frac{1}{144} \times \frac{1}{1000}$	= 18.6 K/ft

91.9 K/ft × 14ft = 1287 Kips

SHEAR WALLS	T(in)	D(ft)	WEIGHT/FT
SW1	20	53	$20 \times 53 \times 145 \cdot \frac{1}{12} \cdot \frac{1}{1000} = 12.8$ K/ft
SW2	20	51.8	$20 \times 51.8 \times 145 \times \frac{1}{12} \times \frac{1}{1000} = 12.5$ K/ft
SW3	16	31	$16 \times 31 \times 145 \times \frac{1}{12} \times \frac{1}{1000} = 6$ K/ft
SW4	16	31	$16 \times 31 \times 145 \times \frac{1}{12} \times \frac{1}{1000} = 6$ K/ft
SW5	20	51	$20 \times 51 \times 145 \times \frac{1}{12} \times \frac{1}{1000} = 12.3$ K/ft
SW6	20	53	$20 \times 53 \times 145 \times \frac{1}{12} \times \frac{1}{1000} = 12.8$ K/ft
SW7	20	53	$20 \times 53 \times 145 \times \frac{1}{12} \times \frac{1}{1000} = 12.8$ K/ft
SW8	20	115	$20 \times 115 \times 145 \times \frac{1}{12} \times \frac{1}{1000} = 27.8$ K/ft
SW9	20	90	$20 \times 90 \times 145 \times \frac{1}{12} \times \frac{1}{1000} = 21.8$ K/ft
SW10	20	26.5	$26.5 \times 20 \times 145 \times \frac{1}{12} \cdot \frac{1}{1000} = 6.4$ K/ft
SW11	20	25.5	$20 \times 25.5 \times 145 \times \frac{1}{12} \times \frac{1}{1000} = 6.2$ K/ft

137.4 K/ft × 14' = 1924 K

SUPER IMPOSED: $35 PSF \times 35093 SF \times \frac{1}{1000} = 1228^K$

WALL LOAD: NO LOAD - FIRST FLOOR (IN LOW RISE BLDG W/ OPEN FLR PLAN)

TOTAL LOAD = 11224 Kips

TECH 1 | SEISMIC - EFFECTIVE WEIGHT

LEVEL 2

SLAB = $\frac{12''}{12''/'} \times 145\ pcf \times 35093 \times \frac{1}{1000} = 5089\ K$

COLS: SAME AS LEVEL 1
$W = 91.9\ K/ft \times 28\ ft = 2573.2\ K$

SHEAR WALLS: SAME AS L1
$W = 137.4\ K/ft \times 28\ ft = 3847.2\ K$

SUPER IMPOSED
$W. = 35 \times 35093 = 1228^{K}$

WALL LOAD: ZERO

TOTAL LOAD: 12737.4

LEVEL 3

SLAB: $\frac{16''}{12''/'} \times 145 \times 35093 \times \frac{1}{1000} = 6785$ Kips

COLS.
(16) 24x30 — $16 \times 24 \times 30 \times 145 \times \frac{1}{144000} = 11.6\ K/ft$
(14) 18x30 — $14 \times 18 \times 30 \times 145 \times \frac{1}{144000} = 7.6\ K/ft$
(8) 36x36 — $8 \times 36 \times 36 \times 145 \times \frac{1}{144000} = 10.4\ K/ft$
(1) 30x42 — $30 \times 42 \times 145 \times 1/144000 = 1.3\ K/ft$
(11) 24x48 — $11 \times 24 \times 48 \times 145 \times \frac{1}{144000} = 12.8\ K/ft$

43.7 K/ft × 22' = 962 Kips

SHEAR WALLS - SAME AS LEVEL 1
$W = 137.4\ K/ft \times 22\ ft = 3023\ K$

SUPERIMPOSED SA. LEVEL 1
$W = 1228\ K$

WALL LOAD: $15\ PSF \times (2 \times 510 + 2 \times 107) = 18.5\ K/ft \times 22\ ft = 407$ Kips

TOTAL = 12405 K.

TECH 1 | SEISMIC – EFF. WEIGHT

FLOORS 4–20 (SW9 REDUCES TO 2/3 DEPTH AT TOP LEVEL 20)

SLAB: $\frac{8.5}{12} \times 145 \times 12800SF \times \frac{1}{1000} = 1315$ k/flr

COLS: S.A. L3 (SAME AS LEVEL "X")

W = 43.7 k/ft × 8.75' = 383 k/flr

SHEAR WALLS (SAME AS L3)

W = 137.4 k/ft × 8.75' = 1203 k/flr

SUPERIMPOSED: SAME

W = 1228 k/flr

WALL LOAD

W = 18.5 k/ft × 8.75' = 162 k/flr

TOTAL LOAD / FLOOR = **4291 k/flr** × 17 FLOORS = 72947 k

FLOORS 21 – 34

ALL SAME AS 4–20 EXCEPT SW9 REDUCED TO 1/3 ORIGINAL DEPTH ABOVE)

TOTAL LOAD/FLOOR = 4291 – 1/3(21.8 k/ft × 8.75 ft) = 4228 k/flr × 14 flrs

= 59192 KIPS

FLOORS 35 & 36

SAME AS 21-34 EXCEPT SW9 IS 1/3 ORIGINAL SIZE

TOTAL LOAD/FLOOR = 4228 k/flr – 1/3(21.8 × 8.75) = 4165 k/flr × 2 flr = 8330 k

FLOORS 37–43

SAME AS 21-34 EXCEPT SW8 IS REDUCED BY 29.5 ft DEEP &
SW4 STOPS @ FLR 36.

TOTAL LOAD/FLOOR

$4165 \text{ k/flr} - 20 \cdot 29.5 \times 149 \times \frac{1}{12000} \times 8.75 - 6 \times 8.75 = 4051$ k/flr × 7 flrs

= 28357

TECH 1 | SEISMIC EFFECTIVE WEIGHT

ROOF

SLABS: 1315 K

COLS: 0

SW: 0

SUPERIMPOSED: 1228 K

WALL LOAD = 18.5 k/ft × 40' = 740K   TOTAL = 3381 K

MECH EQUIPMENT: 98 KIPS

Seismic Load Distribution By floors

Story	Elevation	MidH-MidH	FLR-FLR (ft)	Weight Floor (k)	Wx*hx^k	Cvx	Fx
B		0	0.00	9885			
1	14.0	21	14.00	11224	819883	0.001	1.3
2	42.0	25	28.00	12738	5552705	0.004	8.6
3	64.0	15.375	22.00	12405	10726188	0.008	16.6
4	72.8	8.75	8.75	4291	4569819	0.003	7.1
5	81.5	8.75	8.75	4291	5496684	0.004	8.5
6	90.3	8.75	8.75	4291	6488072	0.005	10.1
7	99.0	8.75	8.75	4291	7541563	0.005	11.7
8	107.8	8.75	8.75	4291	8655039	0.006	13.4
9	116.5	8.75	8.75	4291	9826627	0.007	15.2
10	125.3	8.75	8.75	4291	11054653	0.008	17.1
11	134.0	8.75	8.75	4291	12337607	0.009	19.1
12	142.8	8.75	8.75	4291	13674117	0.010	21.2
14	151.5	8.75	8.75	4291	15062930	0.011	23.3
15	160.3	8.75	8.75	4291	16502895	0.012	25.6
16	169.0	8.75	8.75	4291	17992947	0.013	27.9
17	177.8	8.75	8.75	4291	19532100	0.014	30.3
18	186.5	8.75	8.75	4291	21119435	0.015	32.7
19	195.3	8.75	8.75	4291	22754094	0.016	35.3
20	204.0	8.75	8.75	4291	24435271	0.018	37.9
21	212.8	8.75	8.75	4228	25778100	0.019	39.9
22	221.5	8.75	8.75	4228	27524071	0.020	42.6
23	230.3	8.75	8.75	4228	29313764	0.021	45.4
24	239.0	8.75	8.75	4228	31146551	0.023	48.3
25	247.8	8.75	8.75	4228	33021834	0.024	51.2
26	256.5	8.75	8.75	4228	34939046	0.025	54.1
27	265.3	8.75	8.75	4228	36897645	0.027	57.2
28	274.0	8.75	8.75	4228	38897115	0.028	60.3
29	282.8	8.75	8.75	4228	40936964	0.030	63.4
30	291.5	8.75	8.75	4228	43016718	0.031	66.6
31	300.3	8.75	8.75	4228	45135927	0.033	69.9
32	309.0	8.75	8.75	4228	47294155	0.034	73.3
33	317.8	8.75	8.75	4228	49490985	0.036	76.7
34	326.5	8.75	8.75	4228	51726017	0.037	80.1
35	335.3	8.75	8.75	4165	53194245	0.038	82.4
36	344.0	8.75	8.75	4165	55470111	0.040	85.9
37	352.8	8.75	8.75	4051	56200945	0.041	87.1
38	361.5	8.75	8.75	4051	58485249	0.042	90.6
39	370.3	8.75	8.75	4051	60804430	0.044	94.2
40	379.0	8.75	8.75	4051	63158179	0.046	97.9
41	387.8	8.75	8.75	4051	65546194	0.047	101.6

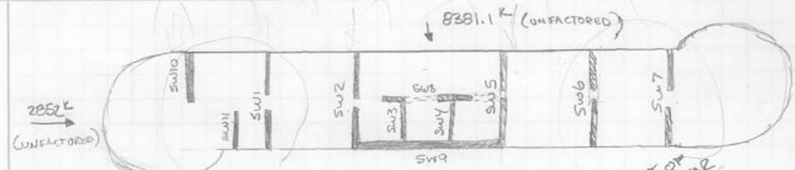
42	396.5	8.75	8.75	4051	67968185	0.049	105.3
43	405.3	10.75	8.75	4051	70423869	0.051	109.1
Roof	418.0	20	12.75	3381	61812713	0.045	95.8
T/Parapet	445.3	13.65	27.25	0	0	0.000	0.0
			total weight =	214168	1382325639	1.000	

Vs=CsW (kips)	2142

k =	1.626

Lateral Load Distribution to Shear Walls

AE481 SHEAR WALL - FORCE DISTRIBUTION



DIRECTION		f'c	E	t	L	H	EI	AMOUNT OF BASE SHEAR	FORCE TO WALL (KIPS)
N-S	SW1	9000	5400	20"	53'	445	4.86 E 7	0.14	1,343
N-S	SW2	9000	5400	20"	51.8'	445	"	0.14	1343
N-S	SW3	9000	5400	16"	31'	445	"	0.08	767
N-S	SW4*3	9000	5400	16"	31'	335.3	"	0.08	767
N-S	SW5	9000	5400	20"	51'	445	"	0.14	1343
N-S	SW6	9000	5400	20"	53'	445	"	0.14	1343
N-S	SW7	9000	5400	20"	53'	445	"	0.14	1343
E-W	SW8	9000	5400	20"	28.5 ; 30.5 / 29* ; 27	445, 335.3	"	0.56	1200
E-W	SW9	9000	5400	20"	90*2	212.8, 317.8, 445	"	0.44	943
N-S	SW10	9000	5400	20"	26.5	445	"	0.07	671
N-S	SW11	9000	5400	20"	25.5	445	"	0.07	671

* STOPS @ FLOOR 35

*2, SHORTENS LENGTH BY 30ft @ 21 & 33 STORIES

*3, STOPS @ FLR 35

DISTRIBUTION OF WIND LOAD TO

$SW10 = 0.07 V_s = 9592 \text{ kips} = 671.0 \text{ kips}$