TECHNICAL REPORT #1 . EXISTING CONDITIONS

CITY VISTA.

BUILDING 2. 5TH AND K STREET . WASHINGTON D. C.

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STRUCTURAL

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OCTOBER 5, 2007

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WASHINGTON D.C
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EXECUTIVE SUMMARY :

City Vista is a three building mixed used complex in downtown Washington D.C. Building 2 is strictly residential and contains 149 condos along with a community room, library, steel frame pedestrian bridge, and outdoor patio. This 11 story 324,298 square feet building reaches a height of 114'-0".

Building 2 is slab on grade with a shear wall lateral system. The crane footings serve double duty as the pedestrian bridge's foundation system. Due to the strict height restriction in Washington D.C the building uses a post tension floor system allowing shallower floor plenum, longer clear spans resulting in a lighter building, and faster cheaper construction. *The District of Columbia Building Code* was used to design City Vista.

This report provides an in-depth description of each structural system, along with wind and seismic analysis, and spot checks of major structural elements. These findings are then compared with the original design.

The appendix located at the end of this report contains more detailed calculations for a better understanding of the analysis done.

1. STRUCTURAL SYSTEM

Foundation System

Building 2 is 4” slab on grade construction with a deep foundation system. Drilled at a depth of 60-65’ below grade there are over 250 16” diam. augured cast in place piles. This foundation system was chosen due to the median to stiff clay located up to 22’ below grade. Piles have an assumed service capacity of 125 tons and typically are reinforced with 1 #8 x 15’-0” LG. Piles under shear walls are reinforced at 25’ with 4-#8 vert. and #4 ties. The slab is thicken at interior CMU walls and location of increased service loads. Grade beams at a width of 1’- 0” are placed around the buildings perimeter at varying depths.

Pedestrian Bridge

The Pedestrian Bridge connects building 2 to building 1A and 1B, having a minimum clearance of 14' from grade. The footings serve double purpose, supporting the crane during construction, and then as the bridge footing. The footings are 5’-6” below grade with (20) HP 12x53x60’0 piles at 60-65ft. The pad is to be reinforced with 2 #10 each crane leg at mid depth. (Fig. 1) The bridge is supported by (4) W12x58 columns that tie into W24x55 concrete incased girder with full moment connection. Framed with composite beams constructed using (8) 24 ¾” Diam. 3 ½” length shear studs spaced at 24” OC and cont. L4x4x5/16” angles coped and welded to beam ends. The floor slab is 3 ½” of lightweight concrete on a 2” 20 gage galvanized metal deck. The slab is reinforced with #3@12’ O.C x 6’-0” long. The roof is a non-composite steel frame system with full moment connections at column locations. The roof is a 1 ½” 20 gage metal roof deck with insulation and rubber asphalt topping.

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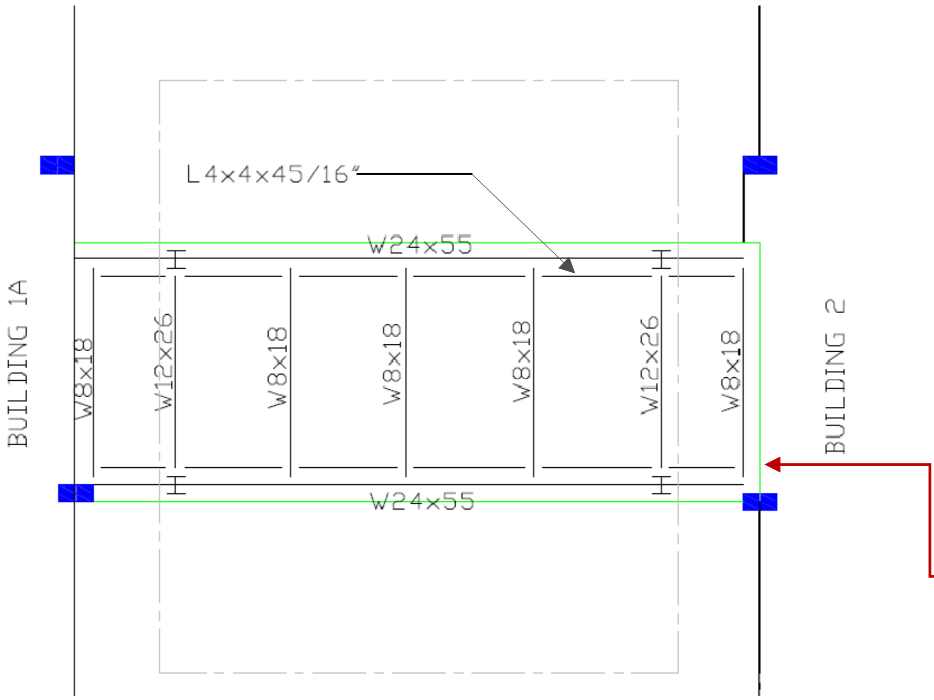

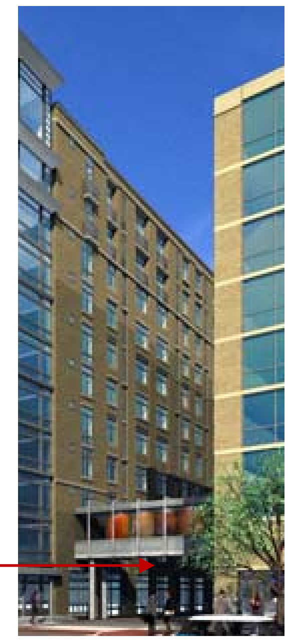

FIG. 1 : Bridge Framing Plan

Framing System

Building two is a joint less structure with a central core. A flat slab system is used, that is there are no beams other than at slab openings (i.e. stair wells). The slab is a two way post tension system. The slabs are supported by a grid of (52) cast in place concrete columns. (4) Concrete shear walls are used for lateral stability, three of which surround the elevator shaft (i.e. the central core). Cold form metal studs are used for most wall construction with the exception of stairwells, mechanical rooms and storage areas which are masonry construction.(See FIG.2)

Shear Walls: Shear walls footings are to be reinforced at a depth of 25'-0" with vertical bars and ties. Typical shear wall reinforcing are #4@12" vertical and horizontal, 8#8 in the middle, and #3 ties in various arrangements.

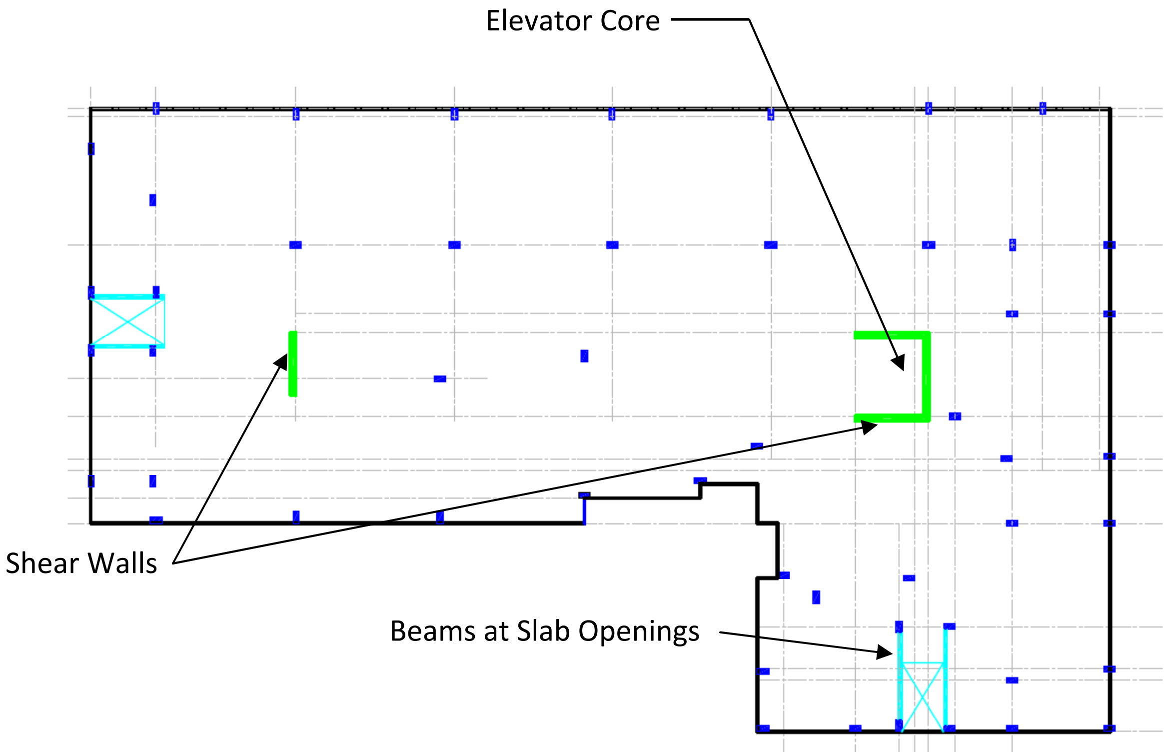

FIG. 2 : Basic Framing Plan

Post-Tension Slab

A two way post-tension slab construction is used on for all floors. The tendons are unbounded and span in both directions with a minimum of (2) tendons above columns (Fig. 3). Banded and uniformed distributed tendons are used. Bundles size varies but restricted to a minimum of 4 tendons per bundle. The 7 ½" slab is reinforced two-ways with #4@24" bottom mesh reinforcement and #5 top bars at various locations, rebar is also provided around the perimeter. Where tendons and rebar intersect chairs should be placed with #4 ties for lateral stability. Tendons stressing will be done with a hydraulic jack, anchorage blockouts are grouted and tendons cut 1" from slab edge, stressing sequence is as followed; Balconies are conventionally reinforced with #4 @ 12" O.C and 2-#5 top & bottom.

1. Stress 50% banded tendons
2. Stress 50% of uniform tendons
3. Stress remaining 50% banded tendons
4. Stress remaining 50% uniform tendons

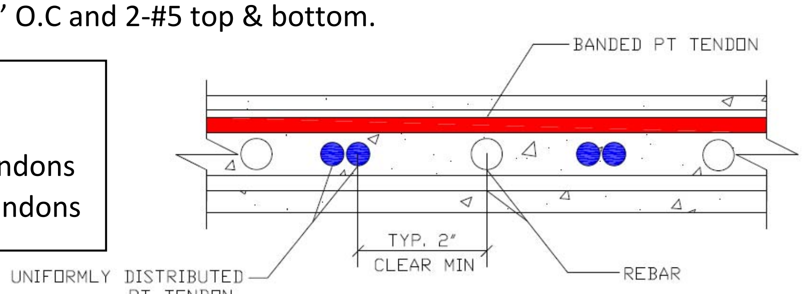

FIG. 3 : Section though slab

Roof

Post-tension roof slab is to be 10" deep with #5@24" reinforcing. A 1 ½" galvanized metal roof deck continuous over 3 spans. Followed by an asphalt membrane, ridged insulation and ballast

2. CODES

Design of City Vista was govern by the following codes:

- *District of Columbia Building Code*
- *ACI 318-89* : *Building Code Requirements for Structural Concrete*
- *ACI 530-99* : *Building Code Requirements for Masonry Structures*
- *Post Tensioning Institute Standards*
- ASCE 7-05 *: American Society Of Civil Engineers 2005 edition*

3. LOADING CONDITIONS

DEAD LOAD

7 ½" Post Tension Slab	150 PCF
Beams	VARIES
Façade #1 (4" Brick, 8" CMU)	95 PSF
Façade #2 (4" Brick, Glass, Cold form)	35 PSF
Superimposed Dead Loads:	
Partitions	20 PSF
Mechanical/Electrical	5 PSF

LIVE LOAD

Residential Units:	40 PSF
Lobbies/Corridors:	100 PSF
Balconies:	100 PSF
Mechanical/Storage:	125 PSF
Canopy:	60 PSF
Public Areas:	100 PSF
Snow:	30 PSF
Elevator Rooms:	150 PSF

4. SESMIC

⟶ See Appendix for complete calculations
Site Classification : D
Building Height : 114'-0"
Latitude / Longitude :
RESULTS FROM SOFTWARE :

$S_s = 0.153$
$S_1 = 0.05$
Fa = 1.6 (Table 11.4-1)
Fv = 2.4 (Table 11.4-2)
$Sm_s = FaS_s = 0.245g$
$Sm_1 = FvS_1 = 0.12g$
$SD_s = 0.163g$
$SD_1 = 0.08g$

*** Concrete Moment Frame with ordinary reinforced shear walls ***
T : Fundamental Period of Structure = $C_tH_n^x$= .697
T_L = Long-Period transition period = 8 Sec
Seismic Use Group: Group Importance Factor: 1.0

Table 12.2-1 : Design Coefficients → *Shear wall-frame interactive system with ordinary reinforced concrete moment frames and ordinary reinforced concrete shear walls*

Seismic Force–Resisting System	ASCE 7 Section where Detailing Requirements are Specified	Response Modification Coefficient, R^a	System Overstrength Factor, Ω_0^g	Deflection Amplification Factor, C_d^b	Structural System Limitations and Building Height (ft) Limit[c]				
					Seismic Design Category				
					B	C	D[d]	E[d]	F[e]
F. SHEAR WALL-FRAME INTERACTIVE SYSTEM WITH ORDINARY REINFORCED CONCRETE MOMENT FRAMES AND ORDINARY REINFORCED CONCRETE SHEAR WALLS	12.2.5.10 and 14.2	4½	2½	4	NL	NP	NP	NP	NP

W = Weight of Building = <u>57,029 Kips</u>
$T \leq T_L$ → C_s = .0191

Base Shear : V = C_sW = <u>1089 Kips</u>

Overturning Moment : M=V*h
= <u>77,962 Kip-Ft</u>

Seismic Loading								
K = 1.1	Level	W_x	H_x	$W_xH1.1_x$	C_{vx} (k)	F_x (kips)	Vx (kips)	Mx (kip-Ft)
Penthouse	13	615.2	117.00	115883.28	0.03	28.64	----	3350.66
Roof	12	1635	107.21	279754.18	0.06	69.14	28.64	7412.01
Residential	11	4980	95.30	748568.48	0.17	184.99	97.78	17629.86
Residential	10	4980	85.97	668360.79	0.15	165.17	282.77	14199.80
Residential	9	4980	76.64	589020.47	0.13	145.56	447.94	11156.04
Residential	8	4980	67.31	510642.42	0.12	126.19	593.50	8494.17
Residential	7	4980	57.98	433346.58	0.10	107.09	719.70	6209.23
Residential	6	4980	48.65	357289.68	0.08	88.30	826.79	4295.64
Residential	5	4980	39.32	282685.94	0.06	69.86	915.09	2746.90
Residential	4	4980	29.99	209847.33	0.05	51.86	984.95	1555.26
Residential	3	4980	20.66	139274.85	0.03	34.42	1036.81	711.09
Residential	2	4980	11.33	71925.39	0.02	17.77	1071.23	201.39
Lobby	1	4980	0.00	0.00	0.00	0.00	1089.00	0.00
TOTAL								77962.06

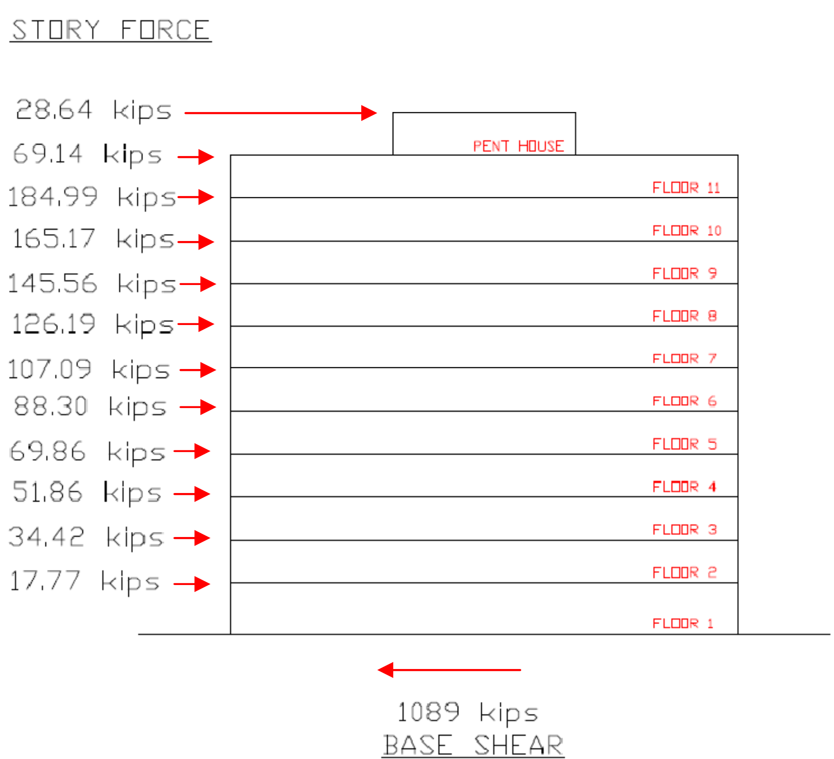

FIG. 4 : Story Force

5. WIND

Wind From W-E				
Windward		Leeward		
h	P	h	P	TOTAL (psf)
0-15	9.27	0-15	-10.2	19.47
20	9.8	20	-10.2	20
25	10.3	25	-10.2	20.5
30	10.7	30	-10.2	20.9
40	11.4	40	-10.2	21.6
50	12.04	50	-10.2	22.24
60	12.5	60	-10.2	22.7
70	12.9	70	-10.2	23.1
80	13.4	80	-10.2	23.6
90	13.7	90	-10.2	23.9
100	14.1	100	-10.2	24.3
120	14.69	120	-10.2	24.89

Wind From N-S				
Windward		Leeward		
h	P	h	P	TOTAL(psf)
0-15	9.44	0-15	-7.32	16.76
20	10.03	20	-7.32	17.35
25	10.5	25	-7.32	17.82
30	10.9	30	-7.32	18.22
40	11.6	40	-7.32	18.92
50	12.26	50	-7.32	19.58
60	12.73	60	-7.32	20.05
70	13.2	70	-7.32	20.52
80	13.6	80	-7.32	20.92
90	14.02	90	-7.32	21.34
100	14.37	100	-7.32	21.69
120	14.9	120	-7.32	22.22

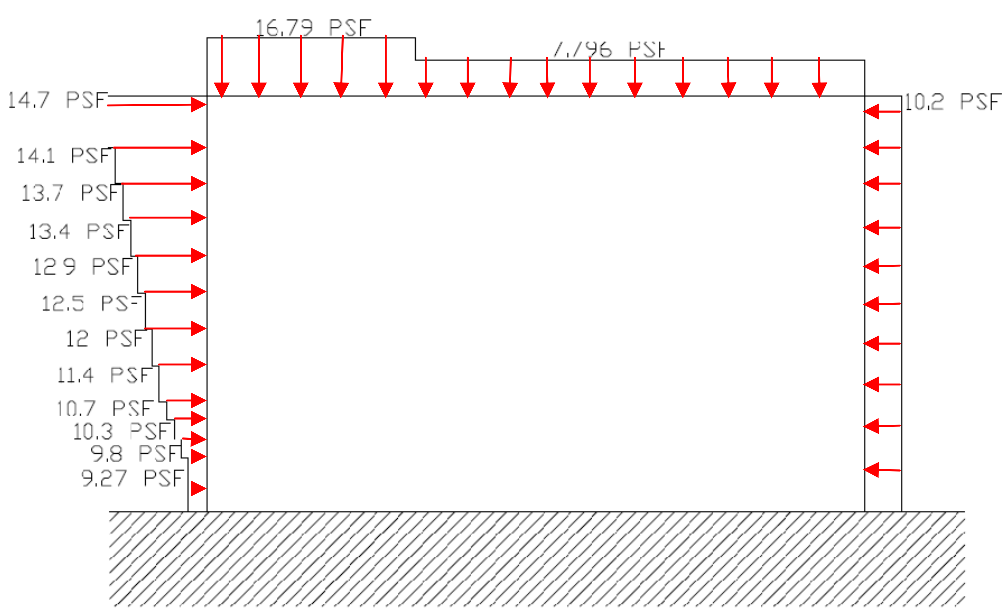

FIG. 5 : WIND PRESSURE FROM W-E

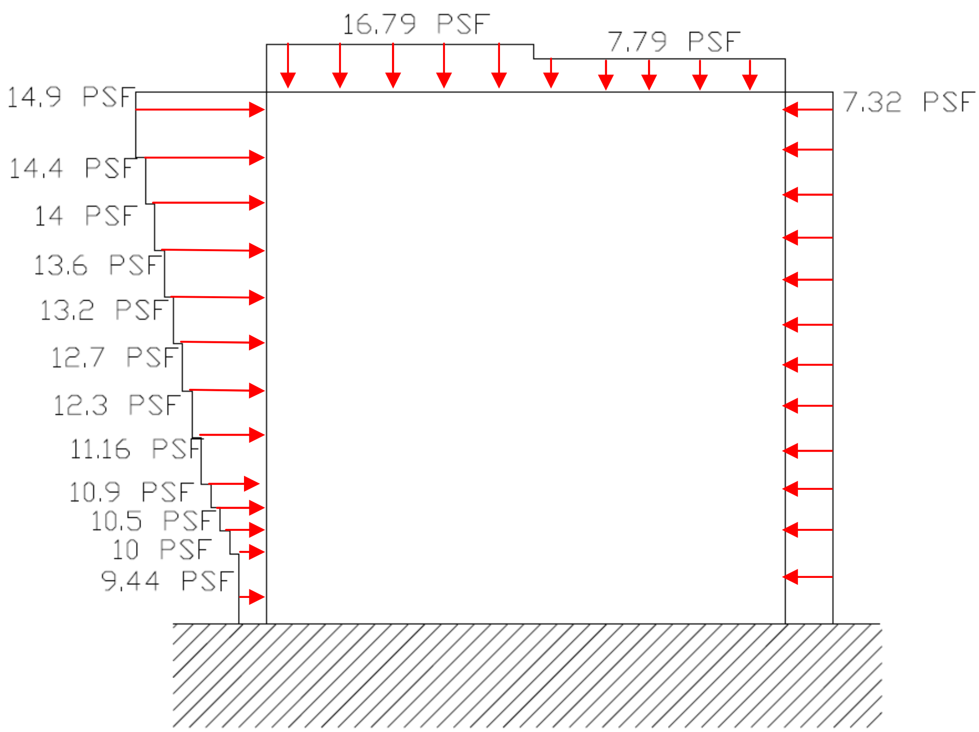

FIG. 6 WIND PRESSURE FROM N-S

ASCE7-05 Chapter 6 → See Appendix for complete calculations
Rigid Building T=.697 Sec < 1 Sec
Exposure Category = B
Enclosure Category = Enclosed Building
Basic Wind Speed: V = 90 mph
Importance Factor : I = 1.0
Mean Roof Height = 114'-0"

GC_{pi} = +/- 0.18	
East/West	$C_{P\ Windward}$ = 0.80 $C_{P\ Leeward}$ = -0.50 $C_{P\ Side}$ = -0.70
North/South	$C_{P\ Windward}$ = 0.80 $C_{p\ Leeward}$ = -0.30 $C_{P\ Side}$ = -0.70

Roof:	
East/West	$C_{P\ Windward}$ = 0-h/2 → -1.3 $C_{P\ Windward}$ = >h/2 → -0.7
North/West	$C_{P\ Windward}$ = 0-h/2 → -1.3 $C_{P\ Windward}$ = >h/2 → -0.7

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6. DESIGN CHECKS

Shear Wall Check:

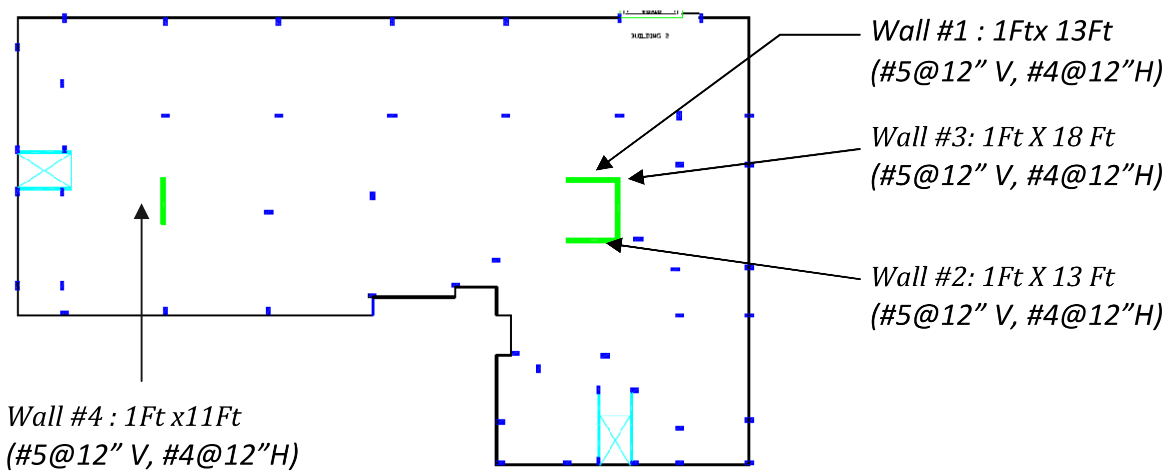


FIG. 7 : Shear Wall Locations

All (4) shear walls extend the full building height of 114'-0". A spot check was done using the *Direct Shear Method (calc in appendix).* Seismic Shear Controls so a V of 1089 kips is used. After completing these calculations the shear walls are sufficient to carry the required base shear. Torsion was not considered in this report and probably is the controlling factor, this explains the high values calculated for shear capacity. Torsion will be investigated in future reports.

Column Check:

For this report it will be assumed that 100% of the lateral shear will be taken by the shear walls, therefore columns will only be checked for pure axial. Column # 22 will be used in the analysis. Column #22 is an interior column with dimensions 16"x28" and an f'c of 6 KSI. Column #22 is sufficient to carry the 1150 Kips load listed in the specs. The nominal strength of the column is close to 50% larger than the 1150 kip gravity load. As a result it can be assumed the column also sees bending, this will be examined further in future reports. *(calc in appendix)*

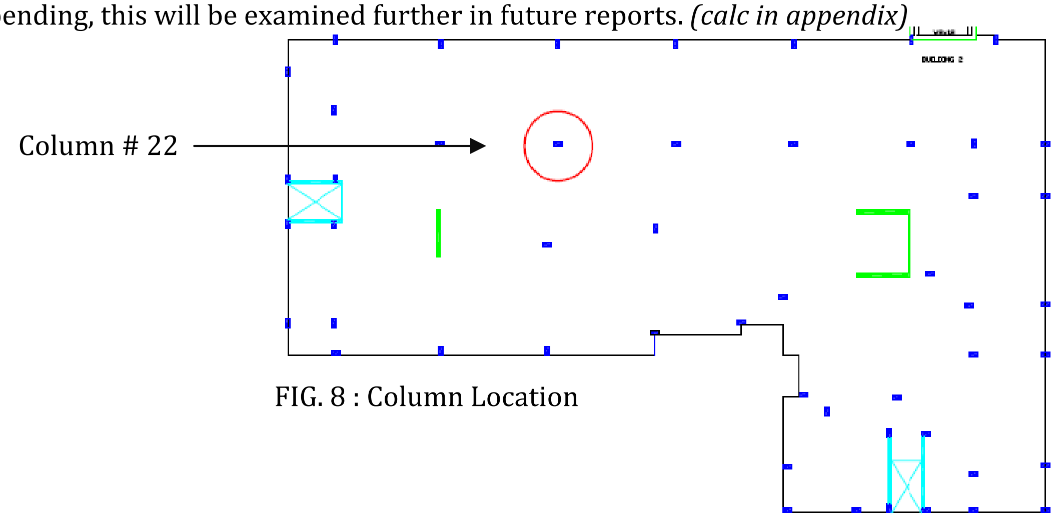
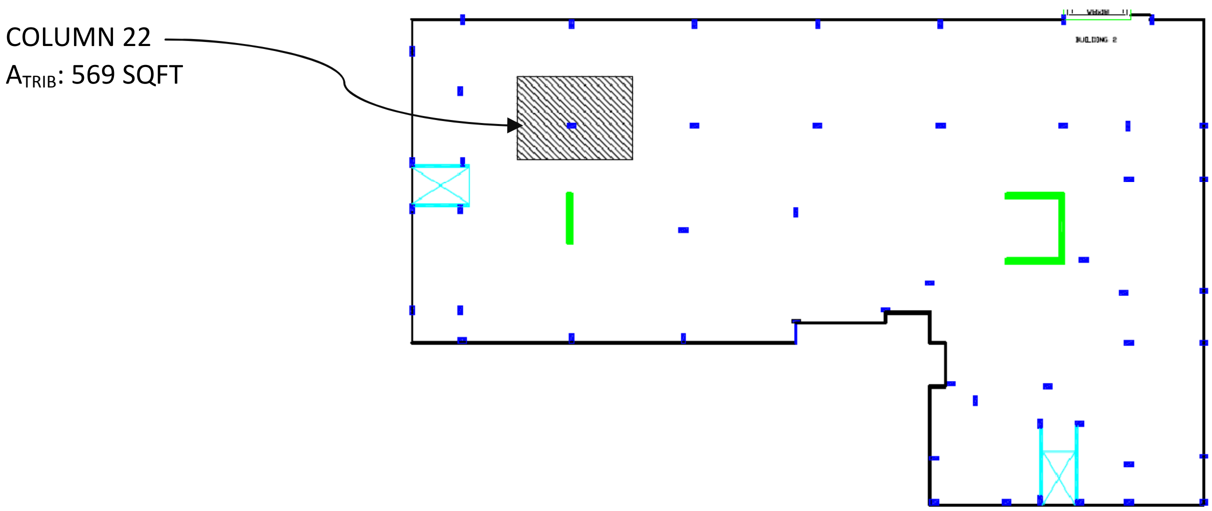


FIG. 8 : Column Location

Punching Shear

City Vista is a flat plate post tension system although for this report no spot checks will be done on this system. Instead punching shear will be checked, since this is a problem in flat plate construction due to under reinforcement. Punching shear was considered for column #22 on the 6th floor. The slab is sufficient for punching shear.



7. CONCLUSION:

Tech assignment #1 has helped me draw conclusions about City Vista's structural system. After doing wind and seismic analysis I have concluded seismic base shear controls. Due to the height of the building this was expected. Spot checks are compared with the plans, and it can be concluded that the shear walls take a large amount of the bending, although the columns also see bending forces. The slab design will be inspected further in future tech assignments although it is sufficient for punching shear. All calculations are available in the appendix.

APPENDIX

WIND CALCULATIONS:

Windward From W-E							
h	Kz	$q_z = q$	$q_h = q_i$	G	Cp	Cpi	P (psf)
0-15	0.57	10.04659	18.33062	0.818	0.8	-0.18	9.27349
20	0.62	10.92787	18.33062	0.818	0.8	-0.18	9.8502
25	0.66	11.6329	18.33062	0.818	0.8	-0.18	10.31157
30	0.7	12.33792	18.33062	0.818	0.8	-0.18	10.77294
40	0.76	13.39546	18.33062	0.818	0.8	-0.18	11.46499
50	0.81	14.27674	18.33062	0.818	0.8	-0.18	12.0417
60	0.85	14.98176	18.33062	0.818	0.8	-0.18	12.50306
70	0.89	15.68678	18.33062	0.818	0.8	-0.18	12.96443
80	0.93	16.39181	18.33062	0.818	0.8	-0.18	13.4258
90	0.96	16.92058	18.33062	0.818	0.8	-0.18	13.77183
100	0.99	17.44934	18.33062	0.818	0.8	-0.18	14.11785
120	1.04	18.33062	18.33062	0.818	0.8	-0.18	14.69456

Leeward From W-E							
h	Kz	$q_h = q$	$q_h = q_i$	G	Cp	Cpi	P (psf)
0-15	0.57	18.33	18.33	0.818	-0.5	0.18	-10.1959
20	0.62	18.33	18.33	0.818	-0.5	0.18	-10.1959
25	0.66	18.33	18.33	0.818	-0.5	0.18	-10.1959
30	0.7	18.33	18.33	0.818	-0.5	0.18	-10.1959
40	0.76	18.33	18.33	0.818	-0.5	0.18	-10.1959
50	0.81	18.33	18.33	0.818	-0.5	0.18	-10.1959
60	0.85	18.33	18.33	0.818	-0.5	0.18	-10.1959
70	0.89	18.33	18.33	0.818	-0.5	0.18	-10.1959
80	0.93	18.33	18.33	0.818	-0.5	0.18	-10.1959
90	0.96	18.33	18.33	0.818	-0.5	0.18	-10.1959
100	0.99	18.33	18.33	0.818	-0.5	0.18	-10.1959
120	1.04	18.33	18.33	0.818	-0.5	0.18	-10.1959

Total
19.46
20.04
20.50
20.96
21.65
22.23
22.69
23.15
23.62
23.96
24.31
24.88

Windward From N-S							
h	Kz	$q_z = q$	$q_h = q_i$	G	Cp	Cpi	P (psf)
0-15	0.57	10.04659	18.33062	0.833	0.8	-0.18	9.443542
20	0.62	10.92787	18.33062	0.833	0.8	-0.18	10.03083
25	0.66	11.6329	18.33062	0.833	0.8	-0.18	10.50066
30	0.7	12.33792	18.33062	0.833	0.8	-0.18	10.97048
40	0.76	13.39546	18.33062	0.833	0.8	-0.18	11.67523
50	0.81	14.27674	18.33062	0.833	0.8	-0.18	12.26251
60	0.85	14.98176	18.33062	0.833	0.8	-0.18	12.73234
70	0.89	15.68678	18.33062	0.833	0.8	-0.18	13.20217
80	0.93	16.39181	18.33062	0.833	0.8	-0.18	13.67199
90	0.96	16.92058	18.33062	0.833	0.8	-0.18	14.02437
100	0.99	17.44934	18.33062	0.833	0.8	-0.18	14.37674
120	1.04	18.33062	18.33062	0.833	0.8	-0.18	14.96402

Total
16.76
17.35
17.82
18.29
19.00
19.58
20.05
20.52
20.99
21.34
21.70
22.28

Leeward From N-S							
h	Kz	$q_z = q$	$q_h = q_i$	G	Cp	Cpi	P (psf)
0-15	0.57	18.33062	18.33062	0.833	-0.3	0.18	-7.32932
20	0.62	18.33062	18.33062	0.833	-0.3	0.18	-7.32932
25	0.66	18.33062	18.33062	0.833	-0.3	0.18	-7.32932
30	0.7	18.33062	18.33062	0.833	-0.3	0.18	-7.32932
40	0.76	18.33062	18.33062	0.833	-0.3	0.18	-7.32932
50	0.81	18.33062	18.33062	0.833	-0.3	0.18	-7.32932
60	0.85	18.33062	18.33062	0.833	-0.3	0.18	-7.32932
70	0.89	18.33062	18.33062	0.833	-0.3	0.18	-7.32932
80	0.93	18.33062	18.33062	0.833	-0.3	0.18	-7.32932
90	0.96	18.33062	18.33062	0.833	-0.3	0.18	-7.32932
100	0.99	18.33062	18.33062	0.833	-0.3	0.18	-7.32932
120	1.04	18.33062	18.33062	0.833	-0.3	0.18	-7.32932

ASCE7-05

K_z: Table 6-3

K_{zt} : 1.0 :

K_d: Table 6-4 / *Building Main wind force resisting System*

G = .0925 $(1+1.7gI_zQ)/(1+1.7*gI_z)$ → n-s = 0.833
→ w-e = 0.818

Iz = $.3(33/68.4)^{1/3}$=907.4

G = 3.4

Q = → w-e = 0.885
→ n-s = 0.833

C_{pi} : FIG 6-5 = +/- 0.18

Cp: FIG 6.6 → w-e : LEEWARD = -0.5 (L/B = .616)
WINDWARD= 0.8
n-s: LEEWARD = -0.3 (L/B = 1.6)
WINDWARD = 0.8

Wind Base Shear:

W-E		
Area (ft^2)	P (lbs)	Shear
2700	52569	488745
900	18000	436176
900	18450	418176
900	18810	399726
1800	38880	380916
1800	40032	342036
1800	40860	302004
1800	41580	261144
1800	42480	219564
1800	43020	177084
1800	43740	133344
3600	89604	89604
Total Shear	488745 lbs 488.7 KIPS	

N-S		
Area (ft^2)	P(lbs)	Shear
1665	27905.4	247838.6
222	3851.7	219933.2
222	3956.04	216081.5
222	4044.84	212125.4
1110	21001.2	208080.6
1110	21733.8	187079.4
1110	22255.5	165345.6
1110	22777.2	143090.1
1110	23221.2	120312.9
1110	23687.4	97091.7
1110	24075.9	73404.3
2220	49328.4	49328.4
Total Shear	247838.6 lbs 247.83 Kips	

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SEISMIC :

Building Weight (W) = DEAD LOAD + 20% SNOW LOAD + ROOFTOP UNITS+20PSF PARTITION
Post Tension Slab = 150 PCF
Wall #1 = 35 PSF (4" Face Brick, Glass, Cold Form Framing)
Wall #2 = 95 PSF (4" Brick, 8" CMU Back Up)
Snow = 30 PSF
Roof Top Units = 8000 lbs

SESMIC → Weight, Base shear 1 OF 3

PENT HOUSE:

(1,738 SQFT)(.625 FT)(150 PCF) + (95 PSF)[2(20FT+27FT)(10.7FT) + 4(15.9FT+10.1FT)(10.7FT) + (21FT+27FT+27FT+4FT)(17FT)] + (1,738 SQFT)(6 PSF)

= 162,938 (SLAB) + 10,428 + 95,555 + 52,858 + 52,858 + 127,585 (WALLS) + 113,050 (SNOW)

= <u>615.2 KIPS</u>

ROOF:

(15,405 SQFT)(.625 FT)(150 PCF) + (15,405 SQFT)(6 PSF) + (10.67FT/2)(109FT + 62FT + 316FT + 116FT + 736FT + 117FT)(35 PSF) + 2(4 KIPS)

1444.2 (SLAB) + 90.316 (SNOW) + 92.43 (WALL) + 8 (ROOF UNITS)

= <u>1634.94 KIPS.</u>

11TH FLOOR

(15,405 SQFT)(.625 FT)(150 PCF) + $\left(\frac{9.4FT}{2} + \frac{10.97FT}{2}\right)$ (573FT)(35 PSF) + (15,405)(20 PSF)

= 1444.2 (SLAB) + 169.8 (WALLS) + 3081 (PART)

= <u>4695 KIPS</u>

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10TH - 2ND FLOOR:

1444.2 (SLAB) + 3081 (PART) + (9.4)(573)(33 PSF)

= 4684 KIPS

1ST FLOOR

1444.2 (SLAB) + 3081 (PART) + $\left(\frac{9.4}{2}\right)$(573)(33 PSF)

= 4604.76 KIPS

COLUMNS:

4 → (114 FT)(150 PCF)(1.33 FT)(2.33) = 51.12 KIPS

2 → (9.4 FT)(150 PCF)(1.5 FT)(1.5 FT) = 3.17 KIPS

46 → (103.5)(150 PCF)(2.33)(1.3) = 46.42 KIPS

= 2346.14 KIPS / 11 FLOOR = 213.12 KIPS

SHEAR WALL:

#1 : (150 PCF)(1 FT)(114 FT)(13 FT) = 222.3 KIPS

#2 : 222.3 KIPS

#3 : (150 PCF)(1 FT)(16 FT)(114 FT) = 273.6 KIPS

#4 : (150 PCF)(1 FT)(114 FT)(11 FT) = 1881.1 KIPS

= 906.3 KIPS / 11 FLOORS = 82.39 KIPS.

PENT : 615.2 KIPS

ROOF : 1634.94 KIPS

11TH : 4695 + 213.2 + 82.39 = 4990.6 KIPS

10TH–2ND : 4684 + 213.2 + 82.39 = 4979.4 KIPS

1ST : 4604.76 + 213.2 + 82.39 = 4900.4 KIPS

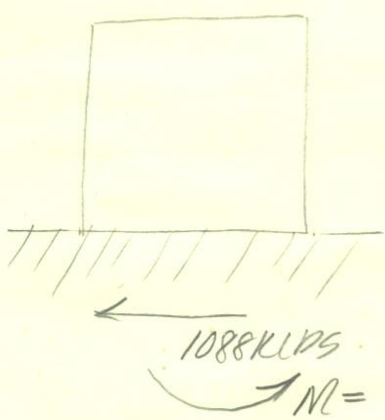
$W_{TOTAL} = 56955.74$ KIPS

BASE SHEAR = $W_T(.0919)$

$56955.74(.0919) = \boxed{1087.8 \text{ KIPS}}$

$T \le T_L$

$C_s = \dfrac{S_{D1}}{T\left(\frac{R}{I}\right)} = \dfrac{.08}{1.864\left(\frac{6}{1}\right)} = .0919$



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MATERIALS :

Cast in Place Concrete	
Member	*Compressive Strength 28 Days*
Slab On Grade	3500 PSI
Post Tension Slab	5000 PSI
Grade Beam	4000 PSI
(Grout) Piles	4000 PSI
Pile Caps	4000 PSI
Masonry Units	1900 PSI

Reinforcing Materials		
Material	*ASTM Standard*	*Fy (KSI)*
Reinforcing Bars	ASTM A615 Grade 60	60
Balconies Reinforcing	ASTM 775	60
Wire Welded Fabric	ASTM A 185	70

Structural Steel		
Material	*ASTM Standard*	*Fy (KSI)*
Pipes	A53 Grade B	50
Tubes	ASTM 500 Grade B	46
Wide Flange	A992 Grade 50	50
Cold Form Studs	16 Gage >16 Gage	50 33
Angles	A36	36
Plates	A36	36
Shear Studs	3/4 “ Diam	50
Anchor Bolts	A36	36

Shear Walls : *Check of Base Shear Capacity*

DIRECT SHEAR: SHEAR WALLS 1 OF 4

* CANTELIVER SHEAR WALL

$V = 1089$ KIPS

$R_1 \rightarrow$ WALL #1 (1FT × 13FT), WALL #2

Rigidity = $1/\Delta$

$$\Delta = \Delta_F + \Delta_S = \frac{PH^3}{3E_m I} + \frac{1.2PH}{E_v A}$$

Assumption: $f'c = 4000$ PSI

$E_m = 5700\sqrt{f'c} = 36E6$ PSI

$E_V = 0.4E_m = 1.44E6$ PSI

$A = (T)(D) = (1)(13) = 13\ ft^2$

$I = \frac{TD^3}{12} = D^3 = 5832$

$P = 1089$ KIPS

$$\frac{(1089E3)(114)^3}{3(36E6)(2197)} + \frac{1.2(1089E3)(114)}{(1.44E6)(12)(13)} =$$

$$6.79 + .663 = 7.45$$

$$\text{Rigidity} = \frac{1}{7.45} = \underline{.134}$$

$R_2 \rightarrow$ WALL #3 1FT × 18FT

$$\frac{(1089E3)(114)^3}{3(36E6)(5832)} + \frac{1.2(1089E3)(114)}{(1.44E6)(12)(18)} =$$

$$2.561 + .478 = 3.039$$

$$\text{Rigidity} = \frac{1}{3.039} = \underline{.329}$$

2 OF

R4 → WALL #4 1FT x 11FT

$$\frac{(1089E3)(114)^3}{3(3600E0)(11)^3} + \frac{12(1089E3)(114)}{(1.44E0)(12)(11)} =$$

$$11.22 + .7837 = 11.98$$

$$1/11.98 = \underline{.083}$$

DIRECT SHEAR

WALL #1 $\frac{.134}{.134+.134+.329+.083}(1089) = \underline{214.59 \text{ KIPS}}$

WALL #2 $= \underline{214.59 \text{ KIPS}}$

WALL #3 $\frac{.329}{.134+.134+.329+.083}(1089) = \underline{526.8 \text{ KIPS}}$

WALL #4 $\frac{.083}{.08}(1089) = \underline{132.9 \text{ KIPS}}$

3 OF 4

Nominal shear capacity

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

→ WALL # 1, 2 (#5@12" V, #4@12 H)

$A_{cv} = (1')(13FT)(12''/FT) = 1872 in^2$

$H/L_W = 114/13 = 8.76 > 2 \; ; \; \alpha_c = 2$

$\rho_T = \dfrac{2(.31)}{(12)(12)} = .0043$

$V_n = 1872[2\sqrt{4000} + .0043(60,000)] = 719.76$ KIPS.

$\phi V_n = .6(719.76) = 413.856.$

$\boxed{413.85 > 214.59 \; \therefore OK.}$

→ WALL #3 (#5@12" V, #4@12 H)

$A_{vc} = (12)(18)(12) = 2592$

$\alpha_c = 2$

$\rho_t = .0043$

$V_n = 2592(2\sqrt{4000} + .0043(60000)) = 996.6$ KIPS

$\phi V_n = .6(996.6) = 597.96$

$\boxed{597.96 > 526.8 \; \therefore OK}$

4 OF 4.

→ WALL #4 (#5@12" V, #4@12" H)

$A_{vc} = (12)(11)(12) = 1584 in^2$

$\alpha_c = 2$

$\rho_t = .00433$

$V_n = 1584(2\sqrt{4000} + .00433(60{,}000)) = 609.03$ KIPS

$\phi V_n = .6(609.03) = 365.42$ KIPS

$\boxed{365.42 > 132.9 \therefore OK}$

SHEAR WALL #4 DESIGN

$f'_c = 4KSI$

$f_y = 60KSI$

12"

11FT

DIRECT SHEAR $V_n = 365.42$ KIPS.

$V_u = 132.9$ KIPS

→ Long & TRAN REINF. (ACI 21.7.2.2)

$2A_{vc}\sqrt{f'_c} = 2(12 \cdot 11 \cdot 12)\sqrt{4000} = 200.36^K$

$200.36 > 132.9 \rightarrow$ 2 CURTIAN REINF.

ρ_L & $\rho_T \geq .0025$

$A_{CV} = (12)(12) = 144 in^2/ft$

A_{LONG} REQ'D $= 0.0025(144) = .36 in^2/ft$

ASSUME #4 $2(.20) = .4 in^2/s$

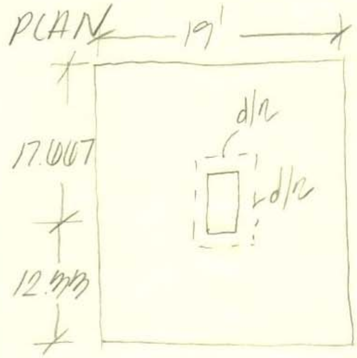
$\frac{.36}{12} = \frac{.4}{s} \longrightarrow S_{REQ} = 13.3" < 18" \therefore OK.$

$\boxed{\text{TRY (2) CURTIAN OF \#4 @13" HORZ \& VERT}}$

Punching Shear:

PUNCHING SHEAR CHECK 1 OF 2

6TH FLOOR COLUMN #22: SIZE: 16"x28"
2-WAY POST TENSION FLAT SLAB
THICKNESS 7 1/2"
$f'c = 5\ KSI$



$\alpha_s = 40$

$d = 7\tfrac{1}{2} - \tfrac{3}{4} = 6.75$

$b_o = 2\left(16 + \frac{6.75}{2}\right) + 2\left(28 + \frac{6.75}{2}\right)$
$= 101.5''$

$\beta c = \frac{28}{16} = 1.75 \leq 2$

Self weight slab:
$(150\ PCF)(.625) = 43.75\ PSF$

Superimposed $= 20\ PSF$

Live load $= 40\ PSF$

Reduction:

$K_a = 0.6 + \frac{.4}{\sqrt{A_x/A_{ref}}} = 0.6 + \frac{0.4}{\sqrt{3135/505}} = .76$

$.76(40) = 30.4\ PSF$

$W_u = 1.2(43.7 + 20) + 1.6(30.4) = 125\ PSF$

$V_u = W_u \cdot A$
$(125\ PSF)(569\ SQFT) = 71.12\ KIPS$

2 OF 2

SHEAR STRENTH

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

$4\sqrt{5000}\,(101.5)(6.75)/1000 = 193.78$ KIPS *

$\rightarrow V_c = \left(2 + \frac{4}{\beta_c}\right)\sqrt{f'_c}\, b_o d = \left(2 + \frac{4}{1.75}\right)\sqrt{5000}\,(101.5)(6.75)/1000$

$= 207.6$ KIPS

$\phi V_c = .75(193.7) = 145.2K$

$\boxed{\phi V_c = 145.2 > V_u = 71.12 \text{ KIPS}}$ $\therefore$ OK

Column Check:

COLUMN CHECK

COLUMN #22 → 28X16
A_{TRIB} = 569 SQFT
$f'c$ = 6KSI

load at base

slab = (150PCF)(.625)(569) = 53.37^k
(53.37)(11 FLOOR) = 586.7^K

self weight = (150PCF)(114')(1.33)(2.33) = 52.9^K

superimposed = (20PSF)(569SQFT) = 11.38^K

Live = (40PSF)(.76)(569) = 17.29^K
(17.29)(11 FLOORS) = 190^K

TOTAL P_u = 840.9 KIPS
* given → 1150 KIPS

1. check pure axial

$P_o = 0.85f'cA_c + A_sf_y$

A_c = (16x28) = 448 − 17.78 = 430.2 in^2
A_s = 14#10 = 14(1.27) = 17.78 in^2

.85(6)(430.2) + 17.78(60)

P_o = 3260.82K > 1150 KIP ∴ OK

* columns take bending will be examin further in future

REQ'D STEEL

· Min steel $\rho g = \frac{Ast}{Ag} = \frac{17.78}{448} = .0397 \geq .001 \therefore OK$

· Max steel $\rho g \leq .06 \therefore OK$