The Edward L Kelly Leadership Center
Prince William County School Administration Center

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Thesis Advisor - Dr. Hanagan

Technical Report 1

October 5, 2007

EXECUTIVE SUMMARY

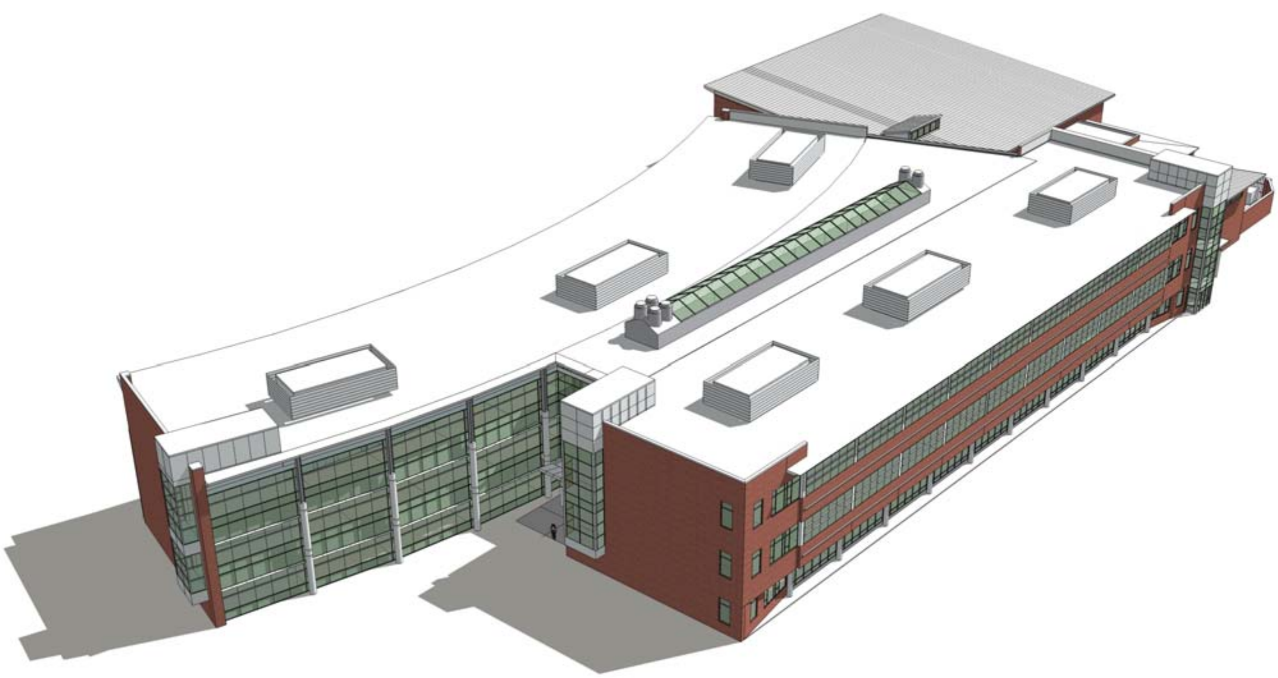
The intent of this technical report is to investigate and analyze the existing structural system of The Edward L Kelly Leadership Center in Manassas, Virginia.

This building has varying level throughout its elevation. There are two main parts of the building that will be referred to in this report: the one-story portion and the two 3-story portions (one rectilinear and the other curvilinear). It was designed by the full-service Architectural/Engineering firm Moseley Architects, located in Richmond, VA. This building will serve as an administration building for Prince William County Schools and is set to open in fall 2008.

The building incorporates an extensive amount of glass into the façade of the building as well as skylighting. Due to the use of large curtain walls prescribed by the architect, the engineer chose to design the frame as a steel moment frame as the most practical system.

In this report, the structural system is examined for wind analysis using the analytical method and seismic analysis using the equivalent force method. Seismic loads are determined to be the controlling lateral force for this building. Without access to the calculations the engineer calculated for the actual design, it is difficult to compare the results of this report to those that are represented in the drawings. Detailed calculations for reference are contained in the appendix following the report. Additional calculations can be made available upon request.

The original design codes the engineer used were based off of ASCE 7-98. This report will reference the most up-to-date standard at this time, ASCE 7-05.



North View

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CODES

The Virginia Uniform Statewide Building Code (VUSBC), 2000 edition was used for the design of the Edward L Kelly Leadership Center. This code, effective October 1, 2003 absorbs much of its code from the International Building Code (IBC). IBC2000 will be used when referencing the original design of this building.

In addition to IBC, the following codes and specifications were also implemented into the design.

ASCE 7-98, Minimum Design Loads for Buildings and Other Structures
ACI 530-99, Building Code Requirements for Masonry Structures With Commentary
AISC Specification for Structural Steel Buildings, Allowable Stress Design and Plastic Design
AISC Code of Standard Practice for Steel Buildings and Bridges
Steel Deck Institute Design Manual for Composite Desks, Form Decks, and Roof Decks
AISI Specification for the Design of Cold Formed Steel Structural Members

LOADING CRITERIA

Dead Load

[redacted]

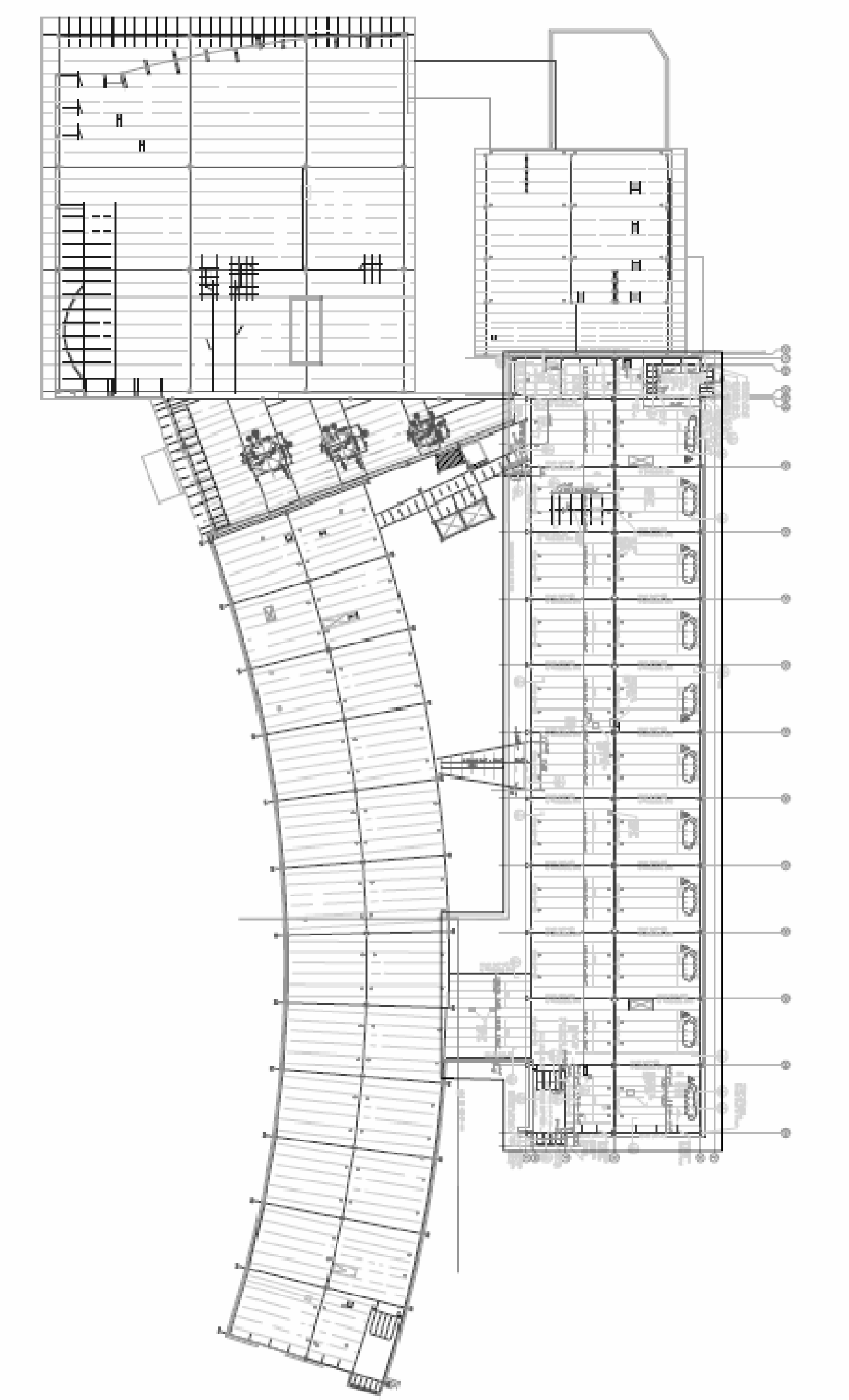
Live Loads

[redacted]

Snow Load

[redacted]

TYPICAL PLANS



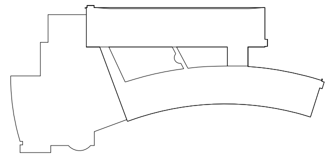


Figure 1. Typical Floor Framing 1

STRUCTURAL SYSTEM

FOUNDATIONS:

Foundations consist of spread footings and strip wall footings. The geotechnical engineer for the project, Dalrymple Poston & Associates, indicated in the report dates November 17, 2005 that the allowable bearing capacity be 3000 PSF. The top of the footings are set at (-2'-0") from grade. Reinforcement for spread footings range from (4)#5 BOT bars for the 3'-0"x3'-0" footings to (11)#7 TOP & BOT for the 11'-0"x11'-0" footings. Exterior column spread footings are typically 4'-0"x4'-0" to 6'-0"x6'-0" in the one-story portion and 7'-0"x7'-0" in the three-story portion. Interior column footings in the one-story portion are typically 6'-0"x6'-0" to 8'-0"x8'-0". The three-story interior column footings are 9'-0"x9'-0" to 11'-0"x11'-0". The strip wall footings are typically 2'-0" wide and 1'-0" thick. Reinforcement for strip footings are (3) continuous #5 bars. The strength of the concrete used for foundations is 3000 psi. The concrete strength for the 4" slab on grade is 3500 psi and contains 6x6-W1.4xW1.4 WWF at mid-depth.

COLUMNS:

All columns in the structural system are steel. In the one-story building, some typical interior columns include W12x79 and W10x68. Exterior columns are often HSS shapes. Typical shapes include HSS8x6x1/4 in the one-story building. In the three-story building, columns are, again, typically W-shapes for the interior and HSS shapes for the exterior. Typical shapes include W14x68 and W14x82 for the interior and HSS12.75x0.375 for the exterior.

FLOOR FRAMING:

Three-story portion:
Built up W21 shapes with HSS2½ (TOP) are typically used for beams while W24 are used for girders. The size of the bays are generally 24' wide and span 30'. Steel joists are used to span inside the bays. 28K8 joists are the most common joist in the framing (Figure 1a). Typical spacing is approximately 4' on center. On the roof, to account for the heavy and asymmetric loads of mechanical equipment, KCS joists are used (Figure 1b). Roof beams are typically W18x35 and girders W21x44.

One-story portion:
This part of the building contains an elevated area that serves as an equipment platform. It covers a good portion of the footprint of this section. The "floor joists" are 26K9 spanning 30' in one part of this platform and 24K3/26K4 spanning 16'/19' respectively. Roof joists in the one-story portion are typically slightly larger than the 3-story building (28K10) since they span a much longer distance of around 47'. The structural plans show an area where the joists become increasingly closer to each

other. This is due to the higher roof causing snow to drift onto the lower roof in addition to windward drift. A few special joists (KSP) are used in certain areas of the one-story roof framing to account for unique loading. This is generally where there are folding partitions, in meeting rooms such as the School Board Meeting room.

LATERAL SYSTEM:

The lateral forces in the building are resisted entirely through moment frames. Because of curtain walls on a great portion of the exterior, shear walls could not be utilized in the design of the lateral system. Therefore, the engineer chose to implement a moment frame to resist these horizontal forces. The particular frame is a space moment frame, meaning that all of the frames are used in the moment frame system.

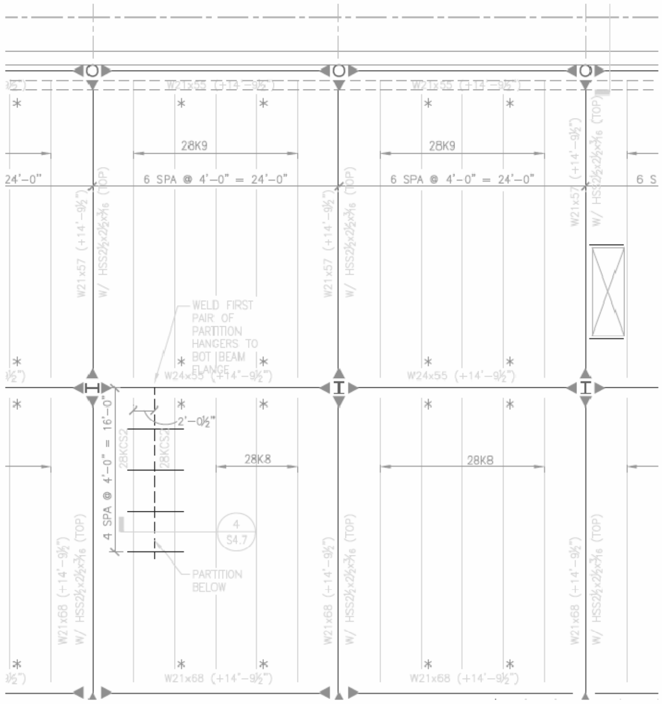


Figure 1a. Rectangular 3-Story Floor Framing

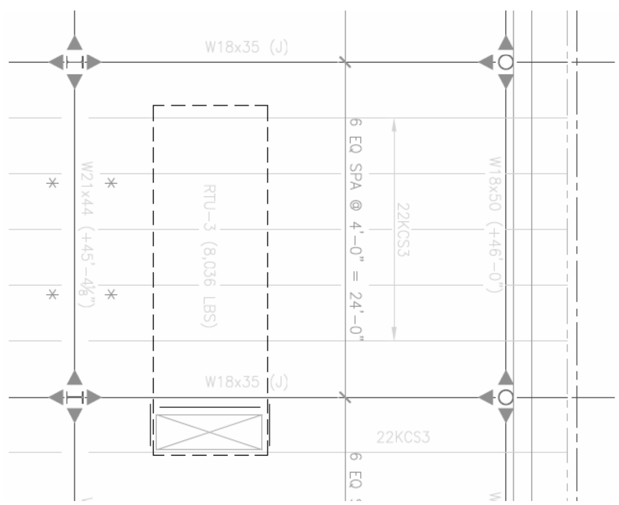


Figure 1b. Rectangular 3-Story Floor Framing 2

Figure 1c. Curved 3-Story Floor Framing 1

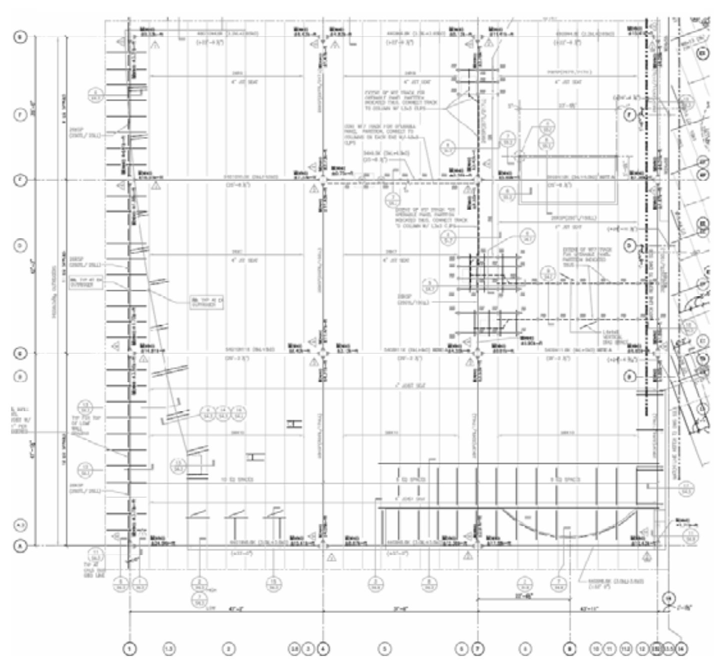


Figure 1d. One-Story Roof Framing

LOADING

SNOW

There will be some areas of the roof that will experience higher than normal snow load because of the surrounding roofs. Areas include: 1) the junction between the 1-story portion of the building and the 3-story portion of the building and 2) the flat roof between two inwardly sloping roofs on the 1-story portion.

Flat Roof Snow Load

$$p_f = 0.7C_eC_tIp_g = 21\,\text{psf}$$
$$C_t = 1.0$$
$$C_e = 1.0$$
$$I = 1.0$$
$$p_g = 30\,\text{psf}$$

Sloped Roof Snow Loads

$$p_s = C_s p_f = 1.0(21) = 21$$

1. Drift from 3-story building onto 1-story building
   The height of the drift is calculated by
   $$h_d = 0.43\sqrt[3]{l_u}\sqrt[4]{p_g + 10} - 1.5$$
   $$h_d = 0.43\sqrt[3]{329}\sqrt[4]{30+10} - 1.5 = 5.97\,\text{ft}$$
   The snow load will be calculated by multiplying the height by the density of snow
   $$\gamma = 0.13p_g + 14 = 0.13(30) + 14 = 17.9\frac{\text{lb}}{\text{ft}^3}$$
   $$\gamma h_d = 17.9\frac{\text{lb}}{\text{ft}^3}(5.97\,\text{ft}) = 106.86\,\text{psf}$$
2. Sliding from two inwardly sloping roofs on 1-story portion
   $$p_{sl} = 0.4p_fW$$
   From the Southern-most roof this equals
   $p_{sl} = 0.4p_fW = 0.4(21)(135) = 1134\,\text{plf}$ distributed over 15 feet
   $$\frac{1134\,\text{plf}}{15\,\text{ft}} = 75.6\,\text{psf}$$
   From the Southern-most roof this equals
   $p_{sl} = 0.4p_fW = 0.4(21)(76) = 638.4\,\text{plf}$ distributed over 15 feet
   $$\frac{638\,\text{plf}}{15\,\text{ft}} = 42.53\,\text{psf}$$

WIND ANALYSIS

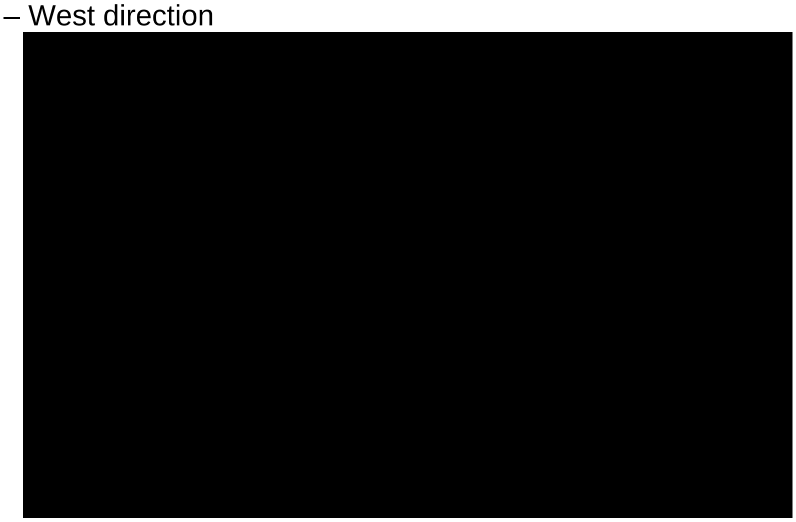
The following charts show the distribution of wind pressures along the height of this building. Appendix A provides complete details of the data.

Wind in the North – South direction

On 3-story portion
Base shear: 258 kips
Overturning Moment: 8714 kip-ft

On 1-story portion
Base Shear: 37 kips
Overturning Moment: 1279 kip-ft

Wind in the East – West direction



On 3-story portion
Base Shear: 98 kips
Overturning Moment: 3367 kip-ft

On 1-story portion
Base Shear: 10 kips
Overturning Moment: 347 kip-ft

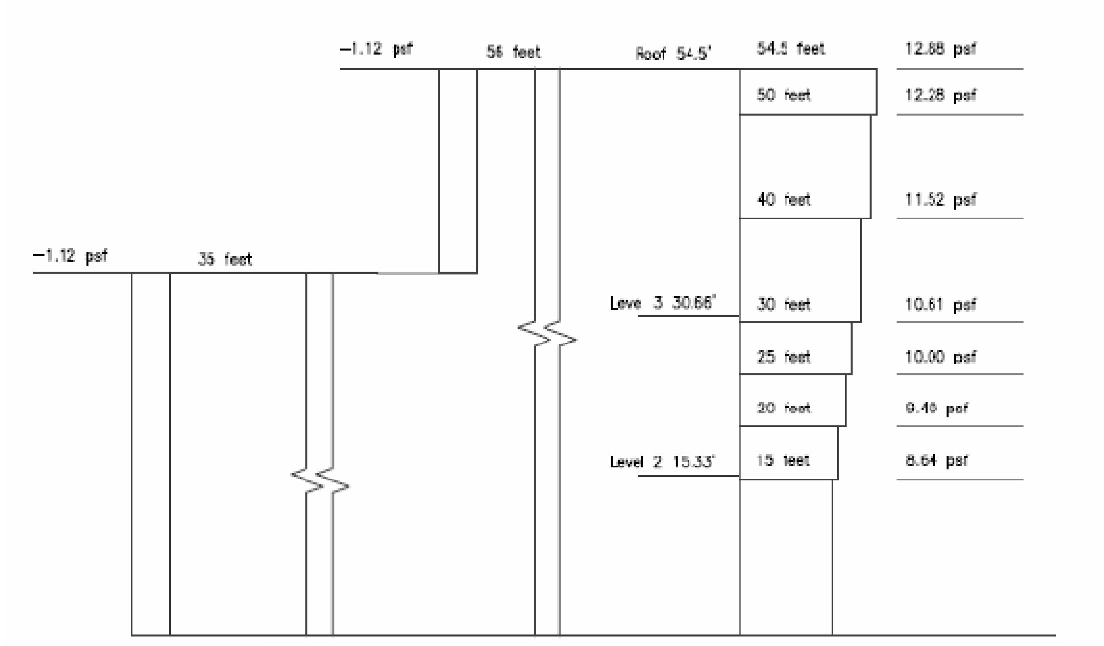


Figure 2. East-West Wind Pressure Diagram on abbreviated South Elevation

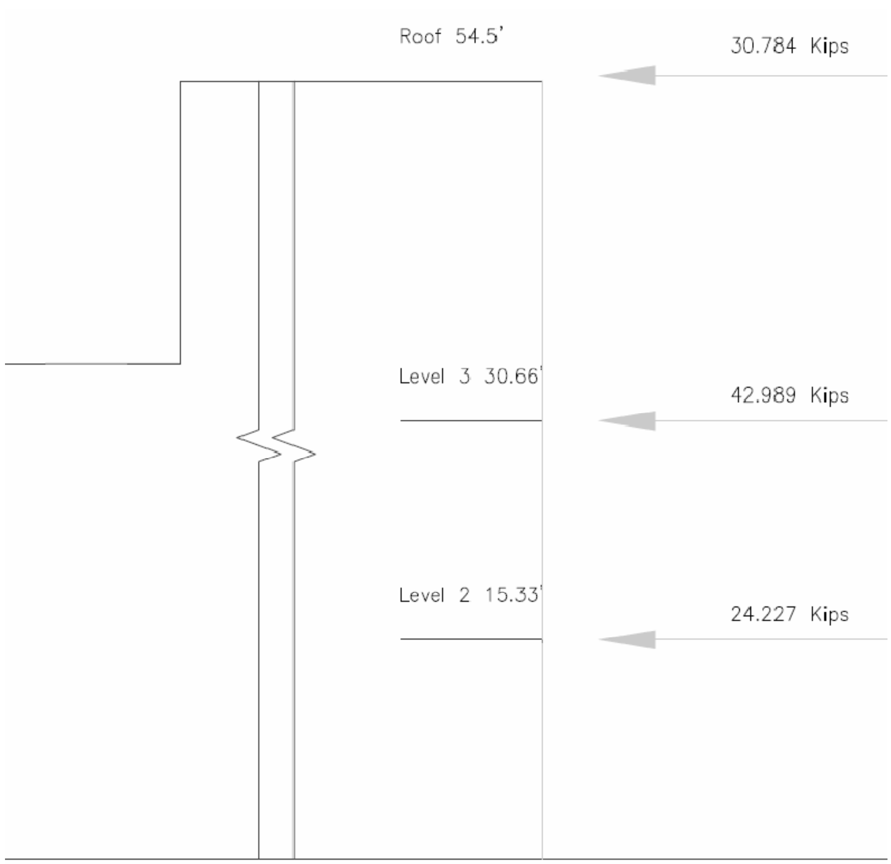


Figure 3. Wind Shear at Each Level, South Elevation

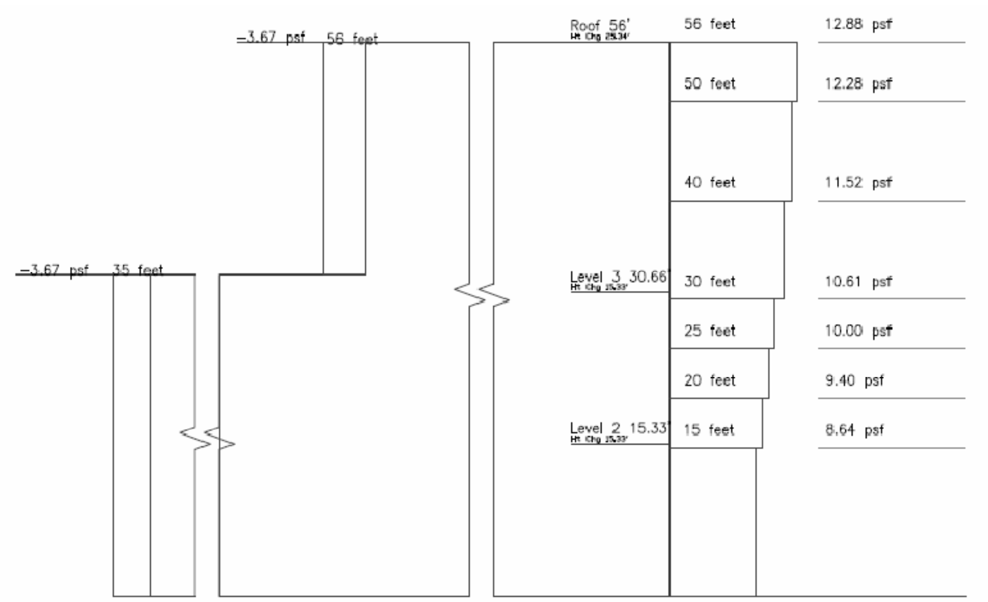


Figure 3. North-South Wind Pressure Diagram on abbreviated East Elevation

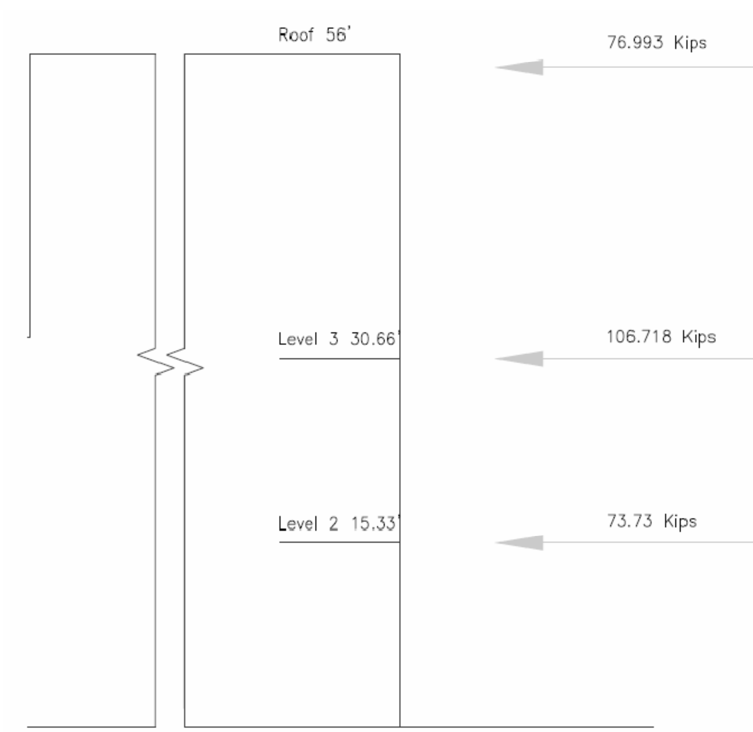
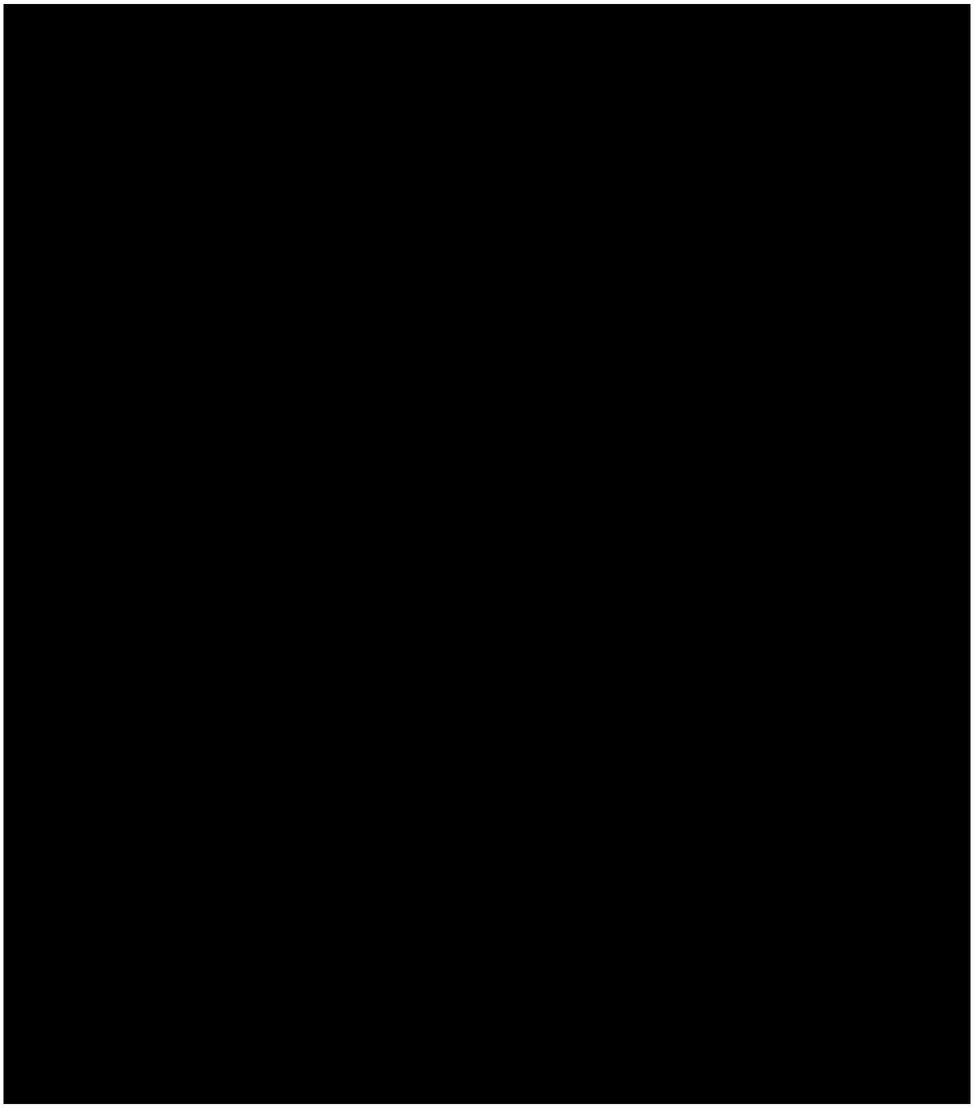


Figure 4. Wind Shear at Each Level, East Elevation

SEISMIC ANALYSIS

The following charts show the Seismic Load calculation summary and the distribution of those forces on the levels of the building. Appendix A contains a detailed walkthrough of the seismic calculation



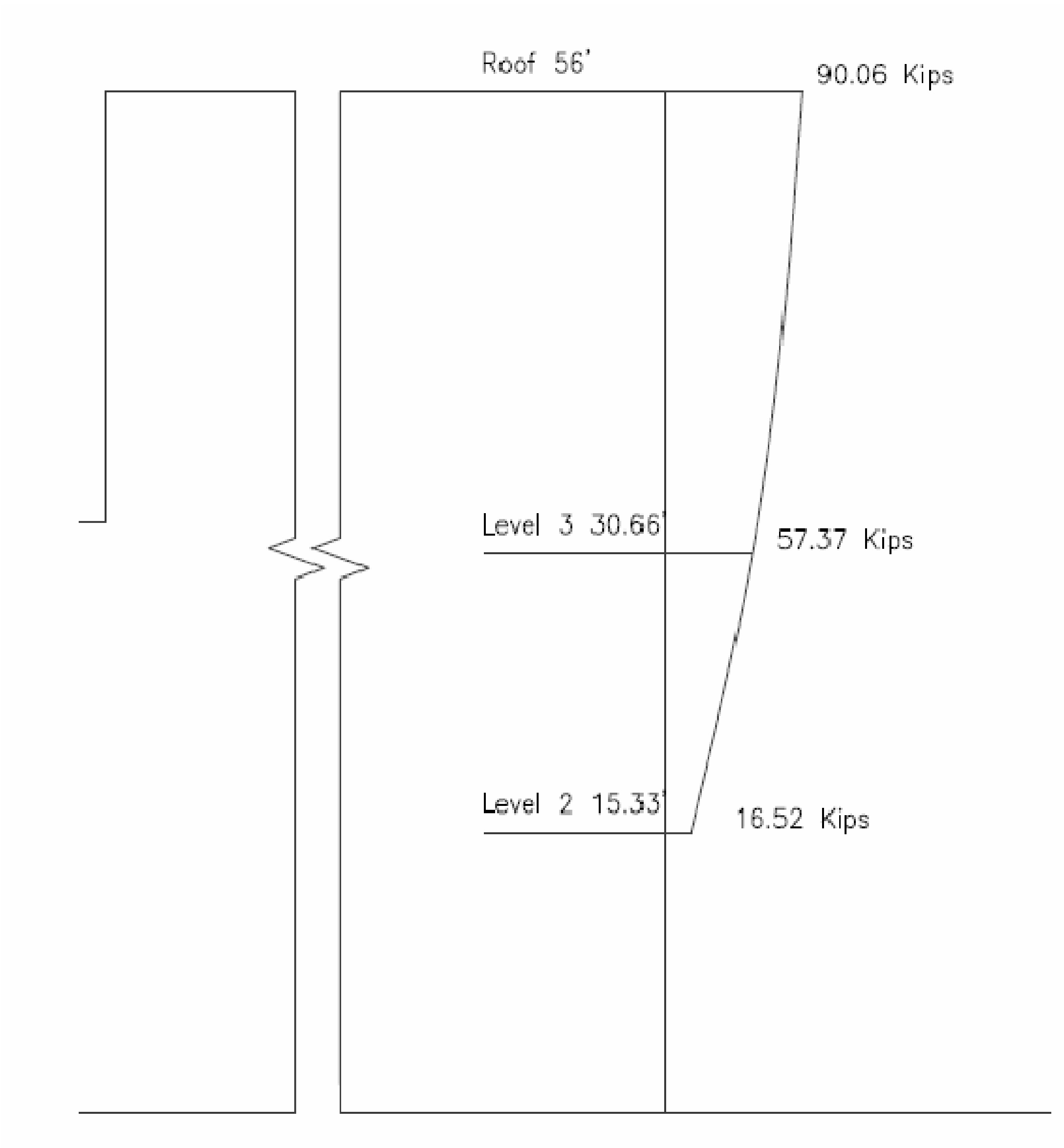


Figure 5. Seismic Shear Force Distribution shown on East Elevation

LATERAL ANALYSIS

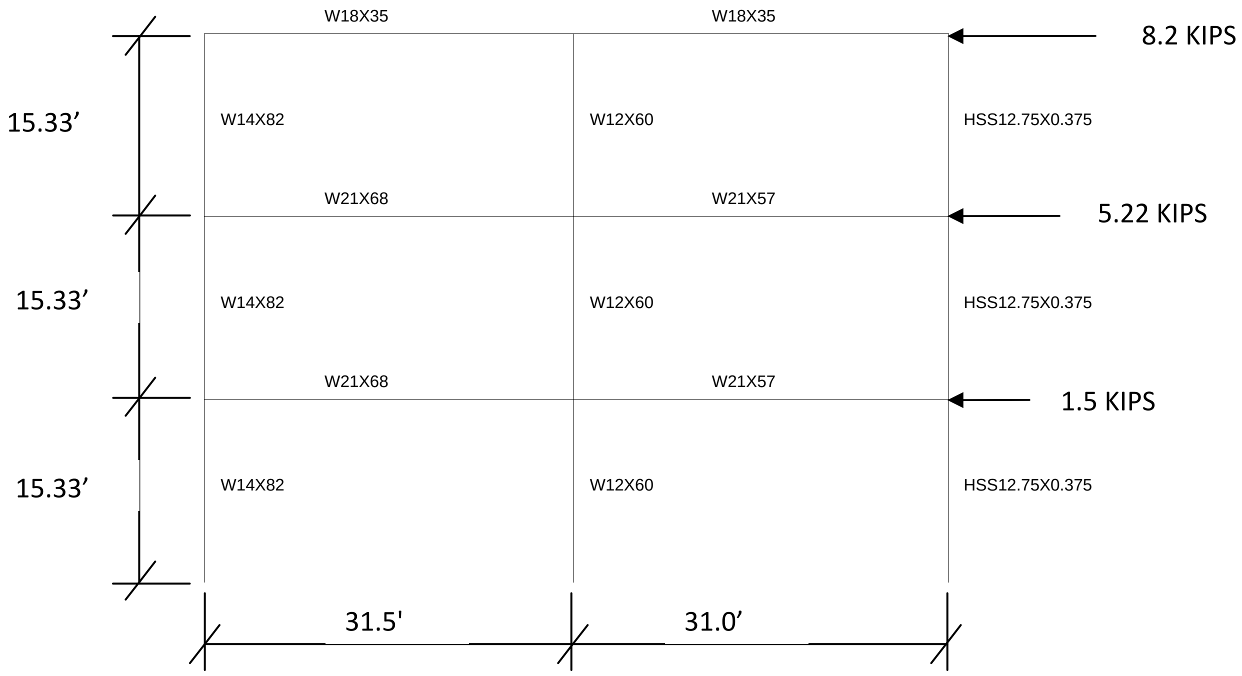
Seismic is the controlling factor for the lateral resisting system. The Seismic Forces will be distributed based upon the tributary area of each frame. This is a simplified approach. It is assumed that each frame, because they are mostly all the same size, has equal stiffness. Therefore, the load will be distributed evenly to each frame. The following is a diagram of one typical frame with the loading applied.

Along the South elevation, there are 11 moment frames. If each frame takes a share of the forces, each frame will see

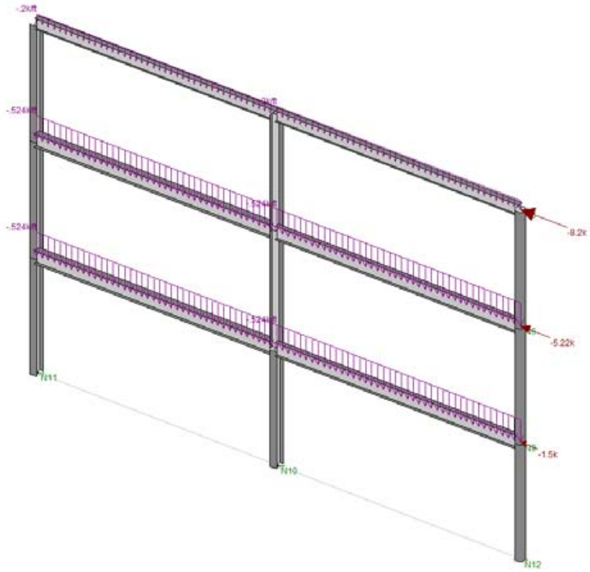
$\frac{1}{11}(90.06\,\text{kips}) = 8.2\,\text{kips}$ at the roof

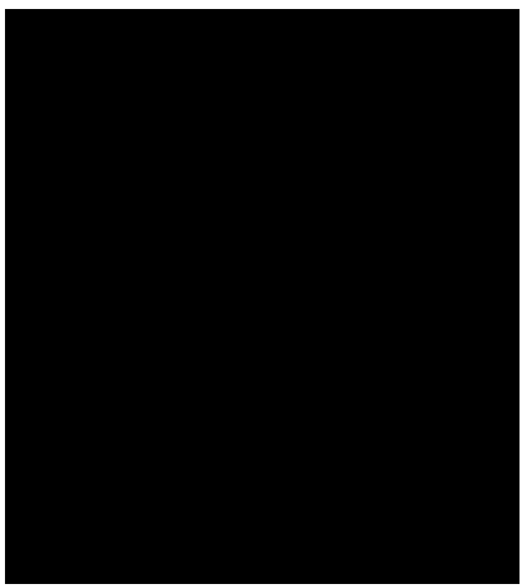
$\frac{1}{11}(57.37\,\text{kips}) = 5.22\,\text{kips}$ at the third level, and

$\frac{1}{11}(16.52\,\text{kips}) = 1.5\,\text{kips}$ at the second level



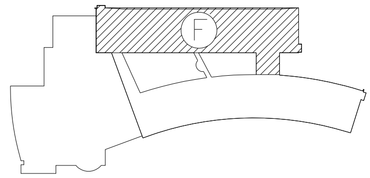
The frame was modeled in RISA with the following results. The “Suggested Shapes” chart shows that all the members work as designed.

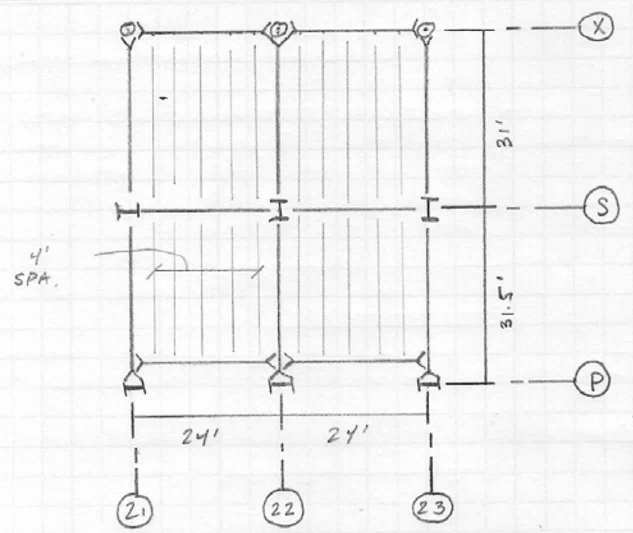




SPOT CHECK

1.) Joist of Second floor framing in "Part F" of building in the bay





JOIST SPOT CHECK

	DECK/SLAB		STEEL		SUPER		
DL =	55 PSF	+	6 PSF	+	10 PSF	=	71 PSF
LL =	50 PSF (OFFICE)	+	10 PSF (PARTITION)			=	60 PSF

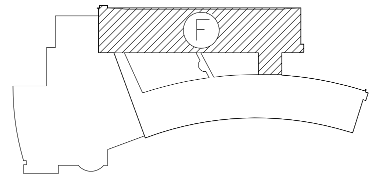
TRIBUTARY WIDTH = 4'-0"

DL: 71 PSF x 4'-0" = 284 PLF
TL = (71+60) PSF x 4'-0" = 524 PLF

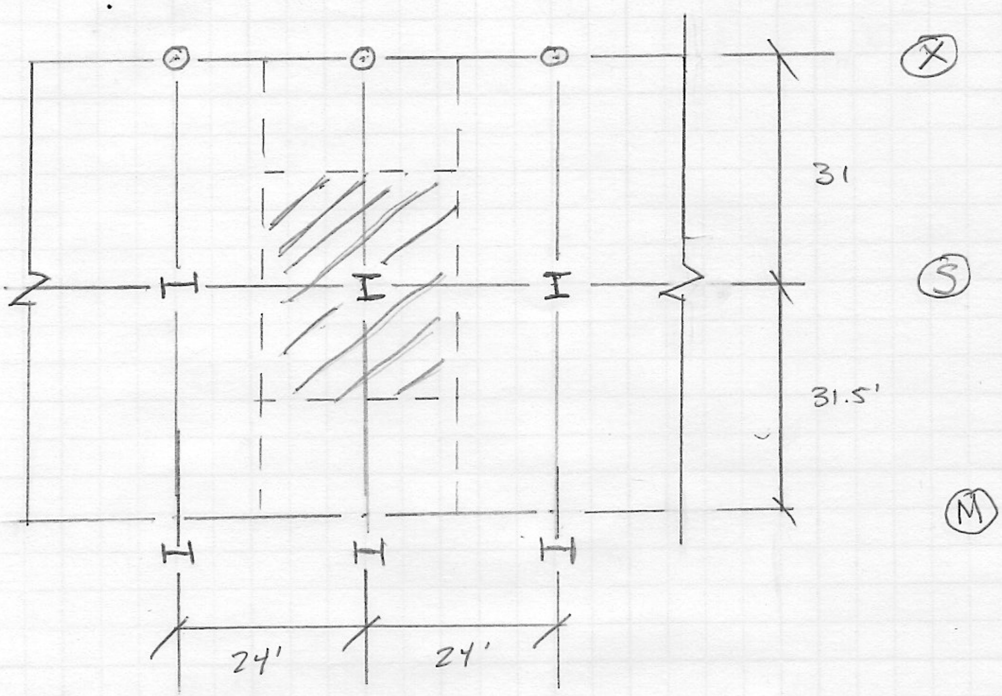
SPAN = 32'

USE 22K9 ALLOWABLE TL = 549 PLF OK
LL = 463 PLF OK

2.) First Story column S-23, Second Floor Framing



COLUMN S23 (KEYPLAN F)



$A_T = \left[\frac{1}{2}(24) + \frac{1}{2}(24)\right] \times \left[\frac{1}{2}(31.5) + \frac{1}{2}(31)\right]$

$= 750^{\square}$

ROOF: DL+LL = 25 + 25 = 50 PSF × 750□
3RD: = 71 + 70 = 141 PSF × 750□
2ND: = 141 PSF × 750

249000 lbs = 249 K

SELF WT: 16 PLF × 46' = 736

$P_u = 249 + 0.736 \approx 250$

KL = 16' W12 × 53 $P_n/\Omega = 301^K$

COMPARE TO W14×68 $P_n/\Omega = 383^K$

CONCLUSIONS FROM SPOT CHECKS:

For the joist design, the result were very close to the engineer's actual design. The 20 PSF additional live load of office partitions could not be used in this spot check because, if it had been included, the total load for the clear span would have exceeded 550 PLF which is the upper bound for normal K-Series joists. The 20 PSF load was taken down to 10 PSF. Since the 71 PSF dead load is likely conservative, this is a valid change. Inconsistencies in the designs could be due to the fact that not all the design loads were disclosed in the drawings. Therefore, assumptions would have to be made on the behalf of the actual designer, who may have had significantly different assumptions for loading conditions. Often times, too, a design may have more to do with aesthetics and workability or consistency with contractors. This could have been a governing factor in the design of the columns.

APPENDIX

A

Seismic Calculations

11.4.1
0.2 Second Spectral Response Acceleration [5% of Critical Damping]
$S_s = 0.162$ [Figure 21-1]
1.0 Second Spectral Response Acceleration [5% of Critical Damping]
$S_1 = 0.052$ [Figure 21-3]

11.4.2
Site Classification: D

11.4.3
Site Coefficients and Adjusted Maximum Considered Earthquake Spectral Response Acceleration
$F_a = 1.6$ [Table 11.4-1]
$S_{MS} = F_a S_s = (1.6)(0.162) = 0.2592$ [Equation 11.4-1]
$F_v = 2.4$ [Table 11.4-2]
$S_{M1} = F_v S_1 = (2.4)(0.052) = 0.1248$ [Equation 11.4-2]

11.4.4
Design Spectral Acceleration
$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.2592) = 0.1728$ [Equation 11.4-3]
$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.1428) = 0.0832$ [Equation 11.4-4]

12.8.2
Period Determination
12.8.2.1
Approximate Fundamental Period
$C_t = 0.028$
$x = 0.8$ [Table 12.8-2]
$h_n = 46$
$T_a = C_t h_n^x = (0.028)(46)^{0.8} = 0.5989$
$C_u = 1.7$ [Table 12.8-1]
$T = C_u T_a = (1.7)(0.5989) = 1.018$

11.4.5
Design Response Spectrum

$$T_0 = 0.2\frac{S_{D1}}{S_{DS}} = 0.2\frac{0.0832}{0.1728} = 0.0930$$

$$T_S = \frac{S_{D1}}{S_{DS}} = \frac{0.0832}{0.1728} = 0.4815$$

$T_L = 8$ [Figure 22-15]

3. For $T > T_S$ and $T \le T_L$

$$S_a = \frac{S_{D1}}{T} = \frac{0.0832}{1.018} = 0.0817$$

11.5.1
Occupancy Category: II [Table 1-1]
Importance Factor: $I = 1.0$ [Table 11.5-1]

11.6 Seismic Design Category
Seismic Design Category Based on 1-s Period Response Acceleration: B [Table 11.6-2]
$0.067 \le S_{D1} < 0.133$

12.8 Equivalent Lateral Force Procedure
12.8.1 Seismic Base Shear
$V = C_s W$ [Equation 12.8-2]

12.2 Structural System Selection
Response Modification Coefficient: $R = 3.5$
System Overstrength Factor: $\omega_0 = 3.0$
Deflection Amplification Factor: $C_d = 3.0$
Structural System Limitations and Building Height Limit: NL for SDC B, C, D, E, F
12.8.1.1 Calculation of Seismic Response Coefficient
For $T \le T_L$

$$C_s = \min\begin{cases} \dfrac{S_{DS}}{\left(\dfrac{R}{I}\right)} = \dfrac{0.1728}{\left(\dfrac{3.5}{1.0}\right)} = 0.0494 \\ \dfrac{S_{D1}}{T\left(\dfrac{R}{I}\right)} = \dfrac{0.0832}{1.018\left(\dfrac{3.5}{1.0}\right)} = 0.0234 \end{cases} \ge 0.01$$

12.8.3 Vertical Distribution of Seismic Forces

$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$ [Equation 12.8-12]

Base Shear

$V = C_s W = (0.0234)(7430) = 173.86\,\text{kips}$

3-Story Portion
Shear at Roof Level

$C_{v1} = 0.518$

$C_{v1}V = (0.518)(173.86) = 90.06\,\text{kips}$

Shear at Third Floor

$C_{v1} = 0.388$

$C_{v1}V = (0.388)(173.86) = 57.37\,\text{kips}$

Shear at Second Floor

$C_{v1} = 0.095$

$C_{v1}V = (0.095)(173.86) = 16.52\,\text{kips}$

1-Story Portion
Shear at Roof Level

$V = C_s W = (0.0234)(766) = 17.92\,\text{kips}$

Notes: All frames have approximately the same relative stiffness; lateral load will be transferred based upon tributary area of the frame at each level

There are 11 Frames to carry the lateral force. Each frame will carry 1/1th the load.

Wind Calculations

WINDWARD

HEIGHT	KZ	KZT	KD	V	I	QZ	GF	CP	qGCP	(+)GCP	(-)GCPI	Qi(+)GCP	Qi(-)GCPI	p=qGCP-qi(+)GCPI	p=qGCP-qi(-)GCPI
0-15	0.57	1	0.85	90	1	10.05	0.85	0.8	6.83	0.18	-0.18	1.808387	-1.80839	5.02	8.64
20	0.62	1	0.85	90	1	10.93	0.85	0.8	7.43	0.18	-0.18	1.967017	-1.96702	5.46	9.40
25	0.66	1	0.85	90	1	11.63	0.85	0.8	7.91	0.18	-0.18	2.093921	-2.09392	5.82	10.00
30	0.7	1	0.85	90	1	12.34	0.85	0.8	8.39	0.18	-0.18	2.220826	-2.22083	6.17	10.61
40	0.76	1	0.85	90	1	13.40	0.85	0.8	9.11	0.18	-0.18	2.411182	-2.41118	6.70	11.52
50	0.81	1	0.85	90	1	14.28	0.85	0.8	9.71	0.18	-0.18	2.569812	-2.56981	7.14	12.28
60	0.85	1	0.85	90	1	14.98	0.85	0.8	10.19	0.18	-0.18	2.696717	-2.69672	7.49	12.88
70	0.89	1	0.85	90	1	15.69	0.85	0.8	10.67	0.18	-0.18	2.823621	-2.82362	7.84	13.49

LEEWARD

HEIGHT	KZ	KZT	KD	V	I	QZ	GF	CP	qGCP	(+)GCP	(-)GCPI	Qi(+)GCP	Qi(-)GCPI	p=qGCP-qi(+)GCPI	p=qGCP-qi(-)GCPI
60	0.85	1	0.85	90	1	14.98	0.85	-0.5	-6.37	0.18	-0.18	2.696717	-2.69672	-9.06	-3.67

LEEWARD

HEIGHT	KZ	KZT	KD	V	I	QZ	GF	CP	qGCP	(+)GCP	(-)GCPI	Qi(+)GCP	Qi(-)GCPI	p=qGCP-qi(+)GCPI	p=qGCP-qi(-)GCPI
60	0.85	1	0.85	90	1	14.98	0.85	-0.3	-3.82	0.18	-0.18	2.696717	-2.69672	-6.52	-1.12

Wind Force Calculation North/South

3-Story Portion

Level 2:

Windward

$$\left(7.33\,\text{ft}\right)\left(8.64\frac{\text{lb}}{\text{ft}^2}\right)+\left(5\,\text{ft}\right)\left(9.4\frac{\text{lb}}{\text{ft}^2}\right)+\left(3\,\text{ft}\right)\left(10\frac{\text{lb}}{\text{ft}^2}\right)=140.33\frac{\text{lb}}{\text{ft}}$$

@ $373\,\text{ft}$

$$\left(140.33\frac{\text{lb}}{\text{ft}}\right)\left(373\,\text{ft}\right)=52344\,\text{lb}=52.344\,\text{kips}$$

Leeward

$$\left(15.33\,\text{ft}\right)\left(-3.74\frac{\text{lb}}{\text{ft}^2}\right)=-57.33\frac{\text{lb}}{\text{ft}}$$

@ $373\,\text{ft}$

$$\left(-57.33\frac{\text{lb}}{\text{ft}}\right)\left(373\,\text{ft}\right)=-21386\,\text{lb}=-21.386\,\text{kips}$$

Total Shear at Level 1: $52.344\,\text{kips}+21.386\,\text{kips}=73.73\,\text{kips}$

Level 3:

Windward

$$\left(2.00\,\text{ft}\right)\left(10\frac{\text{lb}}{\text{ft}^2}\right)+\left(5.00\,\text{ft}\right)\left(10.61\frac{\text{lb}}{\text{ft}^2}\right)+\left(10.00\,\text{ft}\right)\left(11.52\frac{\text{lb}}{\text{ft}^2}\right)+\left(3.33\,\text{ft}\right)\left(12.28\frac{\text{lb}}{\text{ft}^2}\right)=229.14\frac{\text{lb}}{\text{ft}}$$

@ $373\,\text{ft}$ (only 3-story portion)

$$\left(229.14\frac{\text{lb}}{\text{ft}}\right)\left(373\,\text{ft}\right)=85469\,\text{lb}=85.469\,\text{kips}$$

Leeward

$$\left(20.25\,\text{ft}\right)\left(-3.74\frac{\text{lb}}{\text{ft}^2}\right)=-75.74\frac{\text{lb}}{\text{ft}}$$

@ $373\,\text{ft}$

$$\left(-75.74\frac{\text{lb}}{\text{ft}}\right)\left(373\,\text{ft}\right)=-28249\,\text{lb}=-21.249\,\text{kips}$$

Total Shear at Level 3: $85.469\,\text{kips}+21.249\,\text{kips}=106.718\,\text{kips}$

Roof:

Windward

$$\left(6.66\,\text{ft}\right)\left(12.28\frac{\text{lb}}{\text{ft}^2}\right)+\left(6\,\text{ft}\right)\left(12.88\frac{\text{lb}}{\text{ft}^2}\right)=159.065\frac{\text{lb}}{\text{ft}}$$

@ $373\,\text{ft}$ (only 3-story portion)

$$\left(159.065\frac{\text{lb}}{\text{ft}}\right)(373\,\text{ft}) = 59332\,\text{lb} = 59.332\,\text{kips}$$

Leeward

$$(12.66\,\text{ft})\left(-3.74\frac{\text{lb}}{\text{ft}^2}\right) = -47.35\frac{\text{lb}}{\text{ft}}$$

@ $373\,\text{ft}$

$$\left(-47.35\frac{\text{lb}}{\text{ft}}\right)(373\,\text{ft}) = -17661\,\text{lb} = -17.661\,\text{kips}$$

Total Shear at Roof Level: $59.332\,\text{kips} + 17.661\,\text{kips} = 76.993\,\text{kips}$

1-story portion, taken as uniformly 34.5 feet high for simplicity

Roof Level:
Windward

$$(2.66\,\text{ft})\left(9.40\frac{\text{lb}}{\text{ft}^2}\right) + (5\,\text{ft})\left(10.00\frac{\text{lb}}{\text{ft}^2}\right) + (5.00\,\text{ft})\left(10.61\frac{\text{lb}}{\text{ft}^2}\right) + (4.50\,\text{ft})\left(11.07\frac{\text{lb}}{\text{ft}^2}\right) = 177.869\frac{\text{lb}}{\text{ft}}$$

@ $120\,\text{ft}$ (1 story portion)

$$\left(177.869\frac{\text{lb}}{\text{ft}}\right)(120\,\text{ft}) = 21345\,\text{lb} = 21.345\,\text{kips}$$

Leeward - East Elevation (1-story portion, taken as uniformly 34.5 feet high for simplicity)

$$(34.5\,\text{ft})\left(-3.67\frac{\text{lb}}{\text{ft}^2}\right) = -126.615\frac{\text{lb}}{\text{ft}}$$

@ $120\,\text{ft}$ (only 3-story portion)

$$\left(-126.615\frac{\text{lb}}{\text{ft}}\right)(120\,\text{ft}) = -15193.8\,\text{lb} = -15.194\,\text{kips}$$

Total Shear at Roof Level: $21.345\,\text{kips} + 15.194\,\text{kips} = 36.539\,\text{kips}$

Wind Force Calculation East/West

3-Story Portion

Level 2:

Windward

$$\left(7.33\,\text{ft}\right)\left(8.64\frac{\text{lb}}{\text{ft}^2}\right)+\left(5\,\text{ft}\right)\left(9.4\frac{\text{lb}}{\text{ft}^2}\right)+\left(3\,\text{ft}\right)\left(10\frac{\text{lb}}{\text{ft}^2}\right)=140.33\frac{\text{lb}}{\text{ft}}$$

$@173\,\text{ft}$

$$\left(140.33\frac{\text{lb}}{\text{ft}}\right)\left(173\,\text{ft}\right)=24227\,\text{lb}=24.227\,\text{kips}$$

Leeward = 0

Total Shear at Level 1: $24.227\,\text{kips}$

Level 3:

Windward

$$\left(2.00\,\text{ft}\right)\left(10\frac{\text{lb}}{\text{ft}^2}\right)+\left(5.00\,\text{ft}\right)\left(10.61\frac{\text{lb}}{\text{ft}^2}\right)+\left(10.00\,\text{ft}\right)\left(11.52\frac{\text{lb}}{\text{ft}^2}\right)+\left(2.58\,\text{ft}\right)\left(12.28\frac{\text{lb}}{\text{ft}^2}\right)=219.93\frac{\text{lb}}{\text{ft}}$$

$@173\,\text{ft}$

$$\left(219.93\frac{\text{lb}}{\text{ft}}\right)\left(173\,\text{ft}\right)=38048\,\text{lb}=38.048\,\text{kips}$$

Leeward

$$\left(25.5\,\text{ft}\right)\left(-1.12\frac{\text{lb}}{\text{ft}^2}\right)=-28.56\frac{\text{lb}}{\text{ft}}$$

$@173\,\text{ft}$

$$\left(-28.56\frac{\text{lb}}{\text{ft}}\right)\left(173\,\text{ft}\right)=-4941\,\text{lb}=-4.941\,\text{kips}$$

Total Shear at Level 3: $38.048\,\text{kips}+4.941\,\text{kips}=42.989\,\text{kips}$

Roof:

Windward

$$\left(7.4167\,\text{ft}\right)\left(12.28\frac{\text{lb}}{\text{ft}^2}\right)+\left(4.5\,\text{ft}\right)\left(12.88\frac{\text{lb}}{\text{ft}^2}\right)=149.037\frac{\text{lb}}{\text{ft}}$$

$@173\,\text{ft}$

$$\left(149.037\frac{\text{lb}}{\text{ft}}\right)\left(173\,\text{ft}\right)=25784\,\text{lb}=25.874\,\text{kips}$$

Leeward

$$\left(25.5\,\text{ft}\right)\left(-1.12\frac{\text{lb}}{\text{ft}^2}\right)=-28.56\frac{\text{lb}}{\text{ft}}$$

$@173\,\text{ft}$

$$\left(-28.56\frac{\text{lb}}{\text{ft}}\right)(173\,\text{ft}) = -4941\,\text{lb} = -4.941\,\text{kips}$$

Total Shear at Roof Level: $25.874\,\text{kips} + 4.941\,\text{kips} = 30.784\,\text{kips}$

TOTAL SHEAR

1-story portion, taken as uniformly 34.5 feet high for simplicity

Roof Level:

Windward = 0

Leeward

$$(34.5\,\text{ft})\left(-1.12\frac{\text{lb}}{\text{ft}^2}\right) = -38.64\frac{\text{lb}}{\text{ft}}$$

$@\,238\,\text{ft}$

$$\left(-38.64\frac{\text{lb}}{\text{ft}}\right)(238\,\text{ft}) = -9197\,\text{lb} = -9.197\,\text{kips}$$

Total Shear at Roof Level: $-9.197\,\text{kips}$