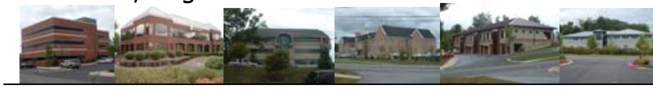


EXECUTIVE SUMMARY

The Signal Hill Professional Center, designed to be an addition to the Manassas Town Center in Northern Virginia, is a 68,000 square foot, four story office building. The building is made up of two sections: a 75' x 165' office structure, with appropriate open office loads, resting on a 110' x 200' parking structure, which must support relatively large 250 psf fire-engine live loads. Like many suburban office structures, this building employs a composite steel system with moment frames. Though the lateral forces were only assessed at above-ground diaphragms, lateral load resistance continues into the underground parking area with shear walls, piers, and moment frames.

Two methods were used to assess lateral force resistance in the two perimeter moment frame systems in the Signal Hill Professional Center. The first uses a STAAD analysis of each moment frame to determine stiffness for lateral force distribution, and then combines lateral forces and gravity loads from a previous RAMSteel analysis to determine maximum moments and shears. The second alters the original RAMSteel model to incorporate the two moment frames and lateral loads. Both seek to follow the lateral forces from the floor diaphragms, to the first floor, and then down into the foundation.

These two analyses revealed that:

- Generally, members in both moment frames were sufficient for the given lateral loadings. Where one beam was at over capacity in the STAAD Analysis, the same beam was well within its strength in the RAM model.
- More attention needs to be brought to the connection between the moment frame columns and the first floor / parking diaphragm. After modeling only the top four floors in RAM with approximated pin connections at the first floor diaphragm, all columns at the first floor were at over capacity. Possibly, moment distribution to moment frames in the underground parking structure would help reduce moments in these members.
- The large length and thickness of the basement walls indicates that shear forces will not control basement wall reinforcement design. Rather, lateral earth pressures will be most critical. The piers adjoining the basement walls, assumed to take all vertical loads from the moment frames, were determined sufficient for both the maximum compressive and tensile forces. Like in the basement walls, lateral earth pressures will again play a larger role in pier design.
- Due to the relatively low height, small lateral forces, and large building weight, overturning moment will not be a concern in structural design.
- Drifts were calculated both on RAMSteel and by hand using a Fleischer Drift Analysis. Both determined maximum drift to be around 2.3", which is greater than the accepted maximum drift of H/400 (1.6"). This discrepancy in drift control is most likely due to improper modeling of support conditions; as the supports range from pinned to fixed, drift should reduce.

INTRODUCTION AND BUILDING SYSTEMS SUMMARY

The Signal Hill Professional Center, designed to be an addition to the Manassas Town Center in Northern Virginia, is a 68,000 square foot, four story office building. The building is made up of two sections: a 75′ x 165′ office structure, with appropriate open office loads, resting on a 110′ x 200′ parking structure, which must support relatively large 250 psf fire-engine live loads.

Like many low-rise steel structures, a system of composite beams and moment frames were used to resist gravity and lateral loads.

Gravity Load System Reviewed

The Signal Hill Professional Center employs a composite steel system, which allows large beam spacing, spans, and shallower floor depths. Typical infill beams span 17-6″ or 20′-0″, and girders typically span 25′-0″ or 30′-0″, supporting a 3″ deck with 3.5″ lightweight concrete slab in the office area, and a 2″ deck with 4.5″ normal weight concrete slab with 4″ asphalt topping in the parking area. The roof system is non-composite, supporting a 2″ deck. Gravity loads include:

- Office Areas: 70 psf dead, 100 psf live
- Parking Areas: 93 psf dead, 250 psf live, 30 psf snow
- Roof: 10 psf dead, 30 psf snow
- Exterior Walls: 440 plf dead load from precast concrete walls

Results from a RAMSteel gravity load model revealed that:

- Infill beams were slightly oversized in both the office and parking areas for the given gravity loads,
- Girders on interior spans were very similar in plan compared to the model output,
- Girders and beams on the perimeter of the building were incredibly smaller in the model than in the plan. For example, RAM results found W16x31 beams in the East-West direction and W10x12 beams in the North-South direction when W21x44 and W18x40 beams were used.

Since the perimeter of the building is the moment frame, it was determined that the larger sizes were the result of resisting additional shears and moments from lateral loads.

Lateral Load System

Steel moment frames were used on the perimeter of the building to resist lateral loads. Since wind loads on a low-rise building in suburban Virginia are not extreme, no additional shear walls were used on the same level as moment-frames [see Lateral Loads]. Moment frames are outlined on the floorplan in Figure 1.

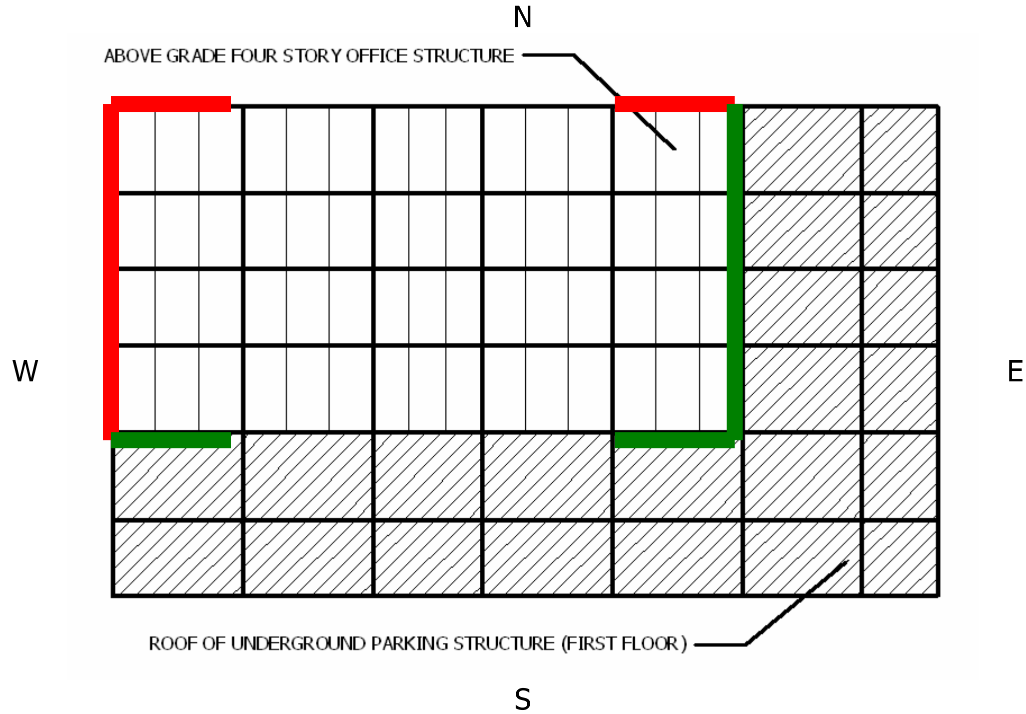


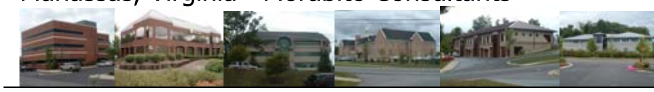
Figure 1. Floorplan, with Moment Frames Highlighted.

Two four-bay, four-story tall moment frames are responsible for resisting loads in the North-South direction, while four one-bay, four-story tall moment frames are responsible for resisting loads in the East-West direction.

Though wind loads will only be present on the exposed four above-ground floors, the lateral resisting system does extend into the underground parking area in two different ways:

1. Moment Frames highlighted in red rest primarily on 18" thick, 13'-9" tall concrete basement walls that act as shear walls. The actual columns rest on 24" square concrete piers which are poured monolithically with the basement wall.
2. Moment Frames highlighted in green extend as moment frames into the basement area. Therefore, these moment frames are five stories tall in their entirety.

Since the hatched basement area is entirely underground, wind and seismic forces on these members were not considered.



DETERMINING LATERAL LOADS

Wind Loads.

The non-simplified ASCE7-02 wind load procedure was used to determine wind loads on the structure. As compared to the simplified analysis used in Technical Assignment 1, these wind loads vary greater and are generally less, which confirms that the simplified procedure is more conservative. Assumptions for this analysis include:

V = 90 mph
K_D = 0.85 [Main Wind Force Resisting System]
I = 1.0 [see building Specs]
K_H = 0.83
K_{ZT} = 1.0 [gradually sloping site]
G = 0.85 [rigid structure]
Completely Enclosed Structure

Wind loads, as summarized in Figure 2, are calculated in Appendix A.

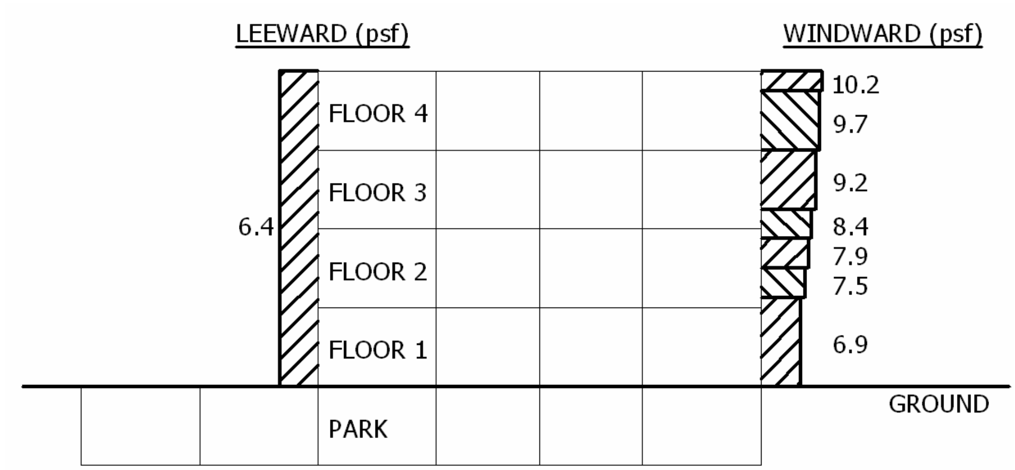


Figure 2. Summary of Wind Pressures

This assumes that the parking area is entirely underground where it will not be influenced by wind forces, even though elements in the parking area will be responsible for resisting some lateral forces.

Seismic Loads.

Seismic Loads were determined using the Equivalent Lateral Force Procedure per ASCE7-02. See Appendix A for calculations. Assumptions include:

Seismic Use Group I
Importance Factor = 1.0
Site Class "D"
Sds = 0.186 (from specifications)
Sd1 = 0.065 (from specifications)
R = 3.0 (Structural steel system not specifically designed for seismic resistance)
Ta = 0.60 = 0.028(building height)$^{0.80}$
Cs = Sds / (R/I) = 0.062 [largest, most critical]
W = weight of structure (total DL): Roof: 629 k
Floors 2-4: 1064 k
V (base shear) = CsW = 170 k [from specs]

Loading Summary.

Wind pressures were assumed to be distributed equally along exterior wall surfaces; therefore, tributary wall areas per floor were multiplied by pressures for total lateral force applied at floor diaphragms. Loadings for both the North-South and East-West directions are summarized in Figures 3a and 3b:

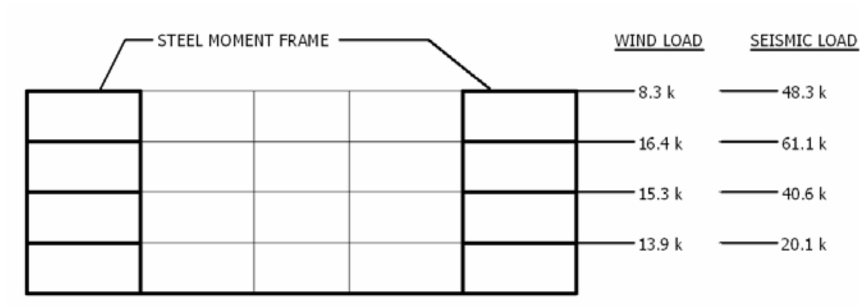


Figure 3a. Lateral Loads to East-West Frame

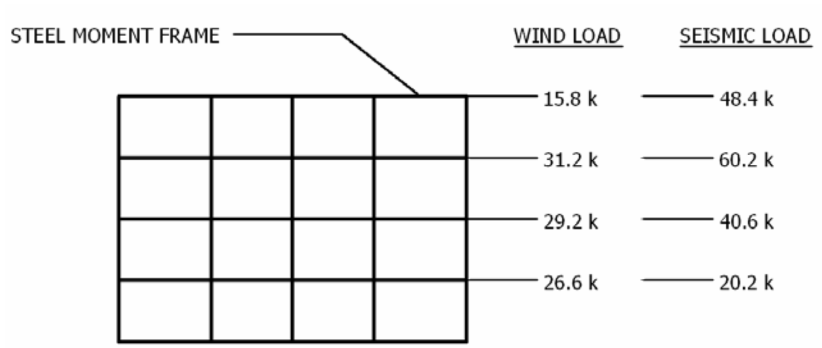
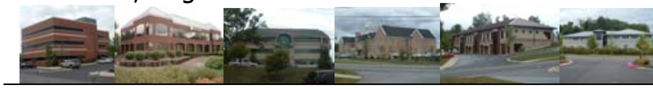


Figure 3b. Lateral Loads to North-South Frame.

Three key load combinations were considered for this analysis, per ASCE7-02:

1. 1.2D +1.6L
2. 1.2D + 0.5L + 1.6W
3. 1.2D + 0.5L + 1.0E

These combinations assume that maximum wind load and seismic load will not occur simultaneously. As further analysis will show, the third load combination including seismic forces controlled for both directions, which is appropriate due to the larger seismic loads.



DISTRIBUTING LOADS TO THE MOMENT FRAMES

Though a quick examination of the symmetric moment frame layout indicates an equal load distribution among the frames, stiffnesses of each moment frame was determined through a STAAD analysis for the purposes of further exploration.

<u>Determining Frame Stiffnesses.</u>

A representative four-bay and one-bay moment frame were modeled in STAAD to represent lateral force resisting elements in the north-south and east-west directions, respectively. Assumptions for this model include:

- Fixed connections between the members,
- Only plane movement in the vertical and horizontal directions,
- A pinned connection at the base of each frame; these column members do not extend deep into the ground but rather attach to the first floor diaphragm.

A 1-kip load was applied to the upper exterior joint, and deflections and stiffness of this frame were determined, as summarized in Table 1.

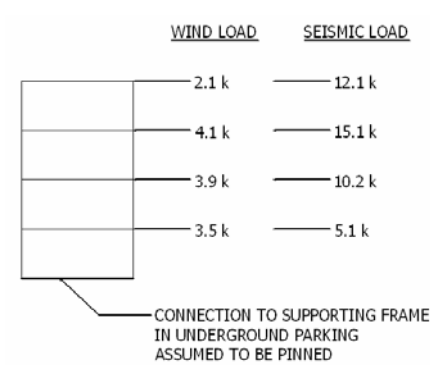
Frame	Maximum Joint Deflection (Δ)	Stiffness (k = 1 / Δ)
North-South (4 bay)	0.089”	11.24
East-West (1 bay)	0.274‟	3.65

Table 1. Summary of Moment Frame Stiffnesses

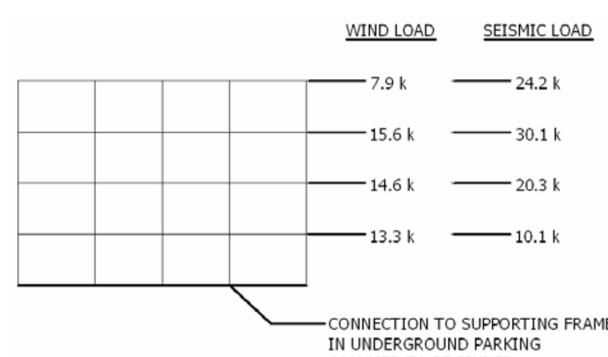
<u>Loads to Each Moment Frame.</u>

Direct Lateral Forces. Since both moment frames in the North-South direction and all four moment frames in the East-West direction are identical and have equal stiffnesses, lateral forces in each direction will be distributed equally.

Torsional Forces. Given that the moment frames are distributed in a symmetric manner about the perimeter of the structure, the center of rigidity coincides with the center of the building, and lateral force eccentricity and torsionally induced shear forces should be close to zero. Loads to each moment frame are summarized in Figure 4.



EW Moment Frame



NS Moment Frame

Figure 4. Lateral Forces presented to each frame for analysis.

MOMENT FRAME CAPACITY

In order to assess the structure's capability to resist lateral forces, both frames were analyzed in STAAD three ways:

1. As a four-story frame, pinned at two supports, with wind loads on all floors. This assumes that no forces in the lateral system are continued into the underground basement structure; this conservatively models the frames in the office building as unable to distribute moments to lower members.
2. As a five-story frame, pinned at two supports, with wind loads on the top four floors. This is designed to portray the frames that extend through the floor of underground parking.
3. Using the reactions at the supports from the first analysis, these forces were placed on the concrete basement walls which were in turn analyzed as shear walls.

These three analyses attempt to follow the lateral load from wall surfaces tributary to the floor diaphragms, to the uppermost member, down to the first floor diaphragm, and then further to either moment-frame columns or a system of concrete piers and shear walls integrated with the underground parking structure. Gravity loads were gathered from the RAMSteel analysis, and moment, shear and deflection capacities were assessed under the three loading conditions [see Lateral Loads].

For analysis, critical members for analysis were chosen at the uppermost level, where lateral loads would play the largest role on the smallest beams and columns, and at the bottom level, where large overall shears and gravity loads would make larger members necessary. STAAD outputs consistently showed that these members featured the greatest stresses.

Results from these analyses show that:

- The moment frames are sufficient under given lateral and gravity loads, though some members are approaching their maximum moment capacity.
- With the exception of the beam in the East-West moment frame, all members are sufficient for given lateral and gravity loads.
- The basement shear walls, with a very long length and relatively large thickness, are sufficient for lateral loads, though earth pressures will control flexure and shear design.
- These findings do not take into account composite action in the beams; therefore, they should be a conservative account of capacity. Since moments in beams would primarily be controlled by vertical loads and therefore composite strength, a non-composite analysis indicates that all members should be sufficient.

See Appendix B for hand calculations pertaining to moment frame and shear wall capacity.

North-South Frame, First Analysis.

This frame assumed that supports are all pinned, and includes beams connected at the support level to distribute moments more effectively. Even though Loading Case 3 controlled in three of the members, full 1.2D + 1.6L gravity loads, found from the RAM analysis, were assumed for axial compression in the column members. This should provide a more conservative analysis, since the large shears and moments would be present in the columns in Loading Cases 2 and 3, where the factor for Live Load is 0.5. Results are summarized in Figure 5 and Table 2.

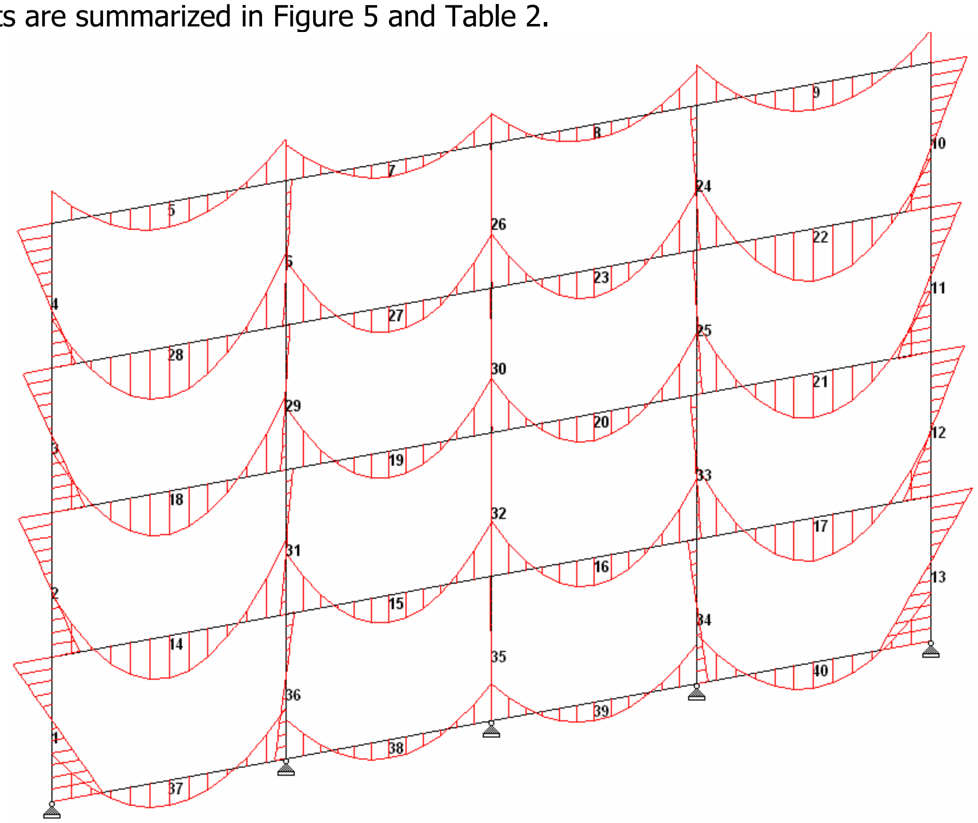


Figure 5. STAAD Output, Maximum Moment Diagram for North-South Frame.

No.	Section	Mu ft-k	ΦMn ft-k	Vu k	ΦVn k	Δ in	Δmax in	Pu k	ΦPn k	Load Case	OK?
4	W12x40	29.0	186.0	7.9	94.8	----	----	18.9	287.0	1	Y
5	W14x22	40.8	123.0	8.2	85.1	0.147	1.0	23.5	275.8	3	Y
13	W12x96	279.0	542.0	109.2	189.0	----	----	330	966.0	3	Y
17	W21x44	340.0	359.0	43.6	196.0	0.163	0.667	9.9	552.5	3	Y

Table 2. Summary of Critical Loadings to Members in North-South Moment Frame.

Though these members were sufficient for the applied loads, it is of concern that moments in Member 17 approach flexural capacity. This is most likely due to modeling the beam as noncomposite, and these members should be analyzed further. See Appendix B for hand calculations.

East-West Frame, First Analysis.

Like the North-South Frame, this frame assumed that supports are all pinned, and includes beams connected at the support level to distribute moments more effectively. Full 1.2D + 1.6L axial loads found in the RAM analysis were used for Member 5, but these were found to be excessive for Member 36. 1.2D + 0.5L loads were used instead, which were acceptable, since the large moments and shears in the column would only be present in Load Cases 2 and 3. Results are summarized in Figure 6 and Table 3.

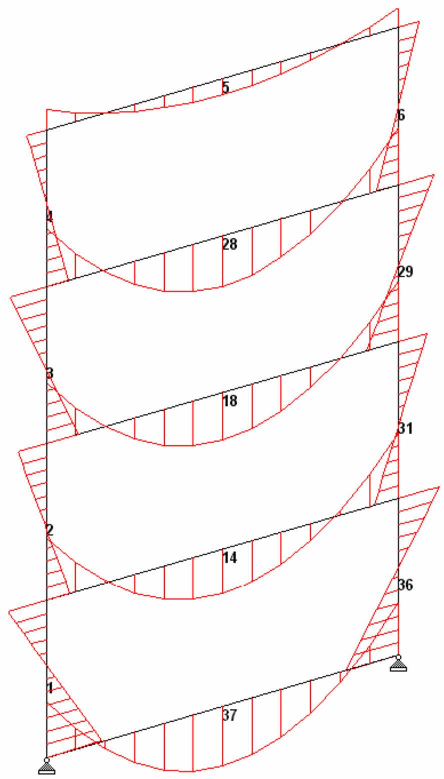
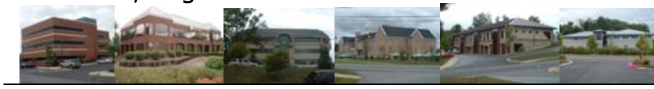


Figure 6. STAAD Output, Maximum Moment Diagram for East-West Frame.

No.	Section	Mu ft-k	ΦMn ft-k	Vu k	ΦVn k	Δ in	Δmax in	Pu k	ΦPn k	Load Case	OK?
4	W12x40	47.0	186.0	10.1	94.8	----	----	18.9	287.0	1	Y
5	W16x26	69.4	166.0	10.2	106.0	0.360	1.5	10.4	326.4	3	Y
14	W21x44	319.2	359.0	35.7	196.0	0.463	1.0	5.9	552.5	3	Y
36	W10x68	230.5	302.0	108.1	132.0	----	----	252	625.0	3	Y

Table 3. Summary of Critical Loadings to Members in East-West Moment Frame.

Like in the North-South frame, the lower beam and column featured moments and loads closest to capacity. Given that both frames in Analysis 1 had this problem, it is assumed that support conditions and interactions with other members within the building frame would further distribute loads.



North-South Frame, Second Analysis.

To account for the moment distribution capacities of the columns in the underground parking area, this frame was extended one floor downward, with no further lateral forces. The leftmost column does not extend; this models the direct connection to the foundation at one end. Every column meets the foundation at a pier or footing with a base plate, therefore, all supports are considered to be pinned. Axial Loads to Column 45 were determined for 1.2D + 1.6L output from the RAM model. Results are summarized in Figure 7 and Table 4.

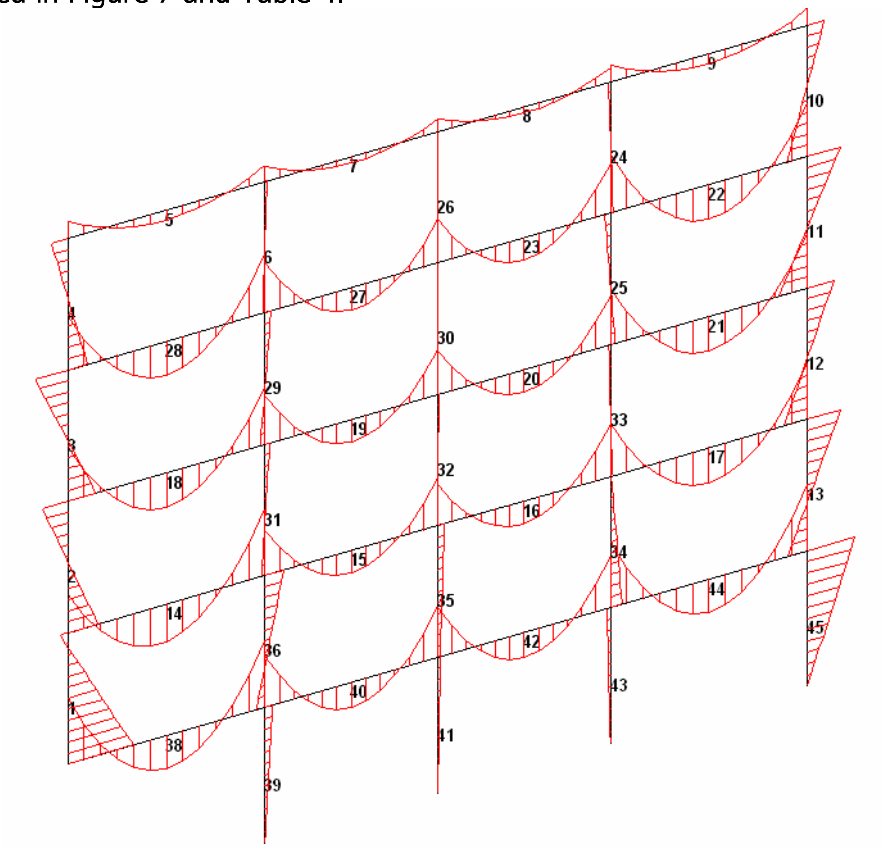
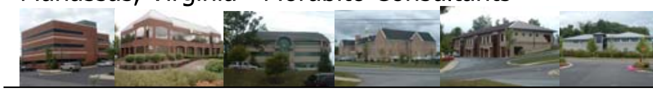


Figure 7. Extended North-South Moment Frame

No.	Section	Mu ft-k	ΦMn ft-k	Vu k	ΦVn k	Δ in	Δmax in	Pu k	ΦPn k	Load Case	OK?
44	W18x40	110.3	294	33.1	152	0.190	0.667	1	501	1	Y
45	W12x96	144.5	542	10.5	189	----	----	330	966	3	Y

Table 4. Summary of Critical Loadings to Members in North-South Moment Frame.

Though most critical, both members are sufficient for this design, and moments in Members 13 and 17, most critical for the first analysis, have slightly smaller moments, shears, and deflections.



East-West Frame, Second Analysis.

This frame was assumed to be the most critical case for analysis; therefore, both sides were extended into the underground parking area. Like the North-South Frame, the supports were assumed to be pinned. Results are summarized in Figure 8 and Table 5.

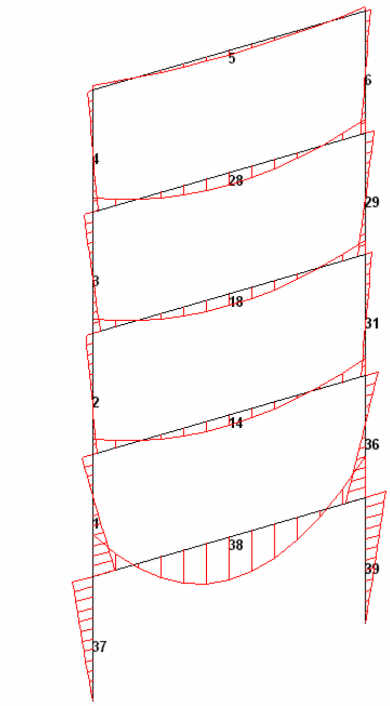


Figure 8. East-West Moment Frame Extending Underground.

No.	Section	Mu ft-k	ΦMn ft-k	Vu k	ΦVn k	Δ in	Δmax in	Pu k	ΦPn k	Load Case	OK?
38	W24x62	648	578	99.8	275	0.994	1	8.7	778	3	N
39	W10x88	220	408	16.1	176	----	----	330	817	2	Y

Table 5. Summary of Critical Loadings to Members in the East-West Frame.

The most critical problem is that the moment capacity in Member 38 is exceeded. This could be possible for many reasons:

- The conservative pinned support condition caused large moments. If this frame were modeled as fixed, the moment in Member 38 would reduce to 474.4 ft-k. Though a fixed condition is not accurate, perhaps some level of fixity would be necessary for analysis.
- Member 38 was modeled as non-composite, which does not take into account the strength of the concrete slab. Since this is a beam member, moments would most likely be from gravity floor loads, rather than lateral loads. Therefore, this frame, using composite steel beams, should be acceptable for lateral loads, upon further analysis.

<u>North-South and East-West Frames, Analysis 3.</u>

Vertical and horizontal reactions from Analysis 1 were used for Analysis 3. Since the columns rest on 24" square concrete piers that extend outside the basement wall, the piers were assumed to support all vertical loads while the walls were assumed to support all horizontal loads.

The concrete piers were assessed for maximum uplift (Load Case 2) and maximum compression (Load Case 1). Due to the small tensile forces and large cross sectional area, only minimum reinforcement for shrinkage and temperature would be necessary. See Appendix A for calculations.

The basement walls were treated as a very deep beam under cantilever loads. Given controlling shears of 132k and 85k for the north-south and east-west shear walls, respectively, they were found to be sufficient in both flexure and shear. Indeed, as pure shear walls, their relatively large size means that minimum reinforcement for shrinkage and temperature would only be necessary. See Appendix B for calculations.

However, since these are basement walls, flexure and shear from lateral earth pressures would play a much more significant role in shear and flexure design.

FURTHER CONSIDERATIONS.

<u>Drift</u>

A Fleischer Drift Analysis is used as an approximation of lateral drift to determine total movement of the structure. Values were interpolated to account for the shorter four-story structure, which should produce larger and therefore more conservative drifts:

- Wind Total Drift: 1.06" at the Roof Level
- Seismic Total Drift: 2.28" at the Roof Level

Assuming a widely accepted H/400 drift limit, the maximum allowable drift is 1.6". Though drift from wind is satisfactory, drift from seismic forces is higher. Further analysis into seismic design and perhaps a less conservative drift calculation approach is necessary to check allowable building movement. See Appendix C for appropriate factors and hand calculations.

<u>Overturning Moment</u>

Since this building has a simple shallow foundation, overturning was also considered. Using the larger seismic loads applied in the North-South direction, the given building weight of 4883k found in the Seismic Load analysis, and the uplift value of 30 psf given in the building specifications, the large building weight was determined to overcome the overturning moment of 19274 ft-k. See Appendix D for moment calculations.

RAMSTEEL ANALYSIS.

Using the RAMSteel model constructed previously for gravity load analysis, the perimeter beams and columns were changed to frame elements for lateral analysis, with the following assumptions. See Figure 9 for the RAMSteel model.

- The underground parking level was not modeled since lateral loads are only indirectly presented to its shear walls and frame elements. Rather, the bases of the columns at the first floor are approximated as pin connections, as in the STAAD models.
- Rather than using the automatically calculated Wind and Seismic loads from the RAM program, the values found via hand calculations were used.
- Weights of floor diaphragms used in this model were calculated in the Seismic Loads section.
- Wind and Seismic loads were considered from both directions.

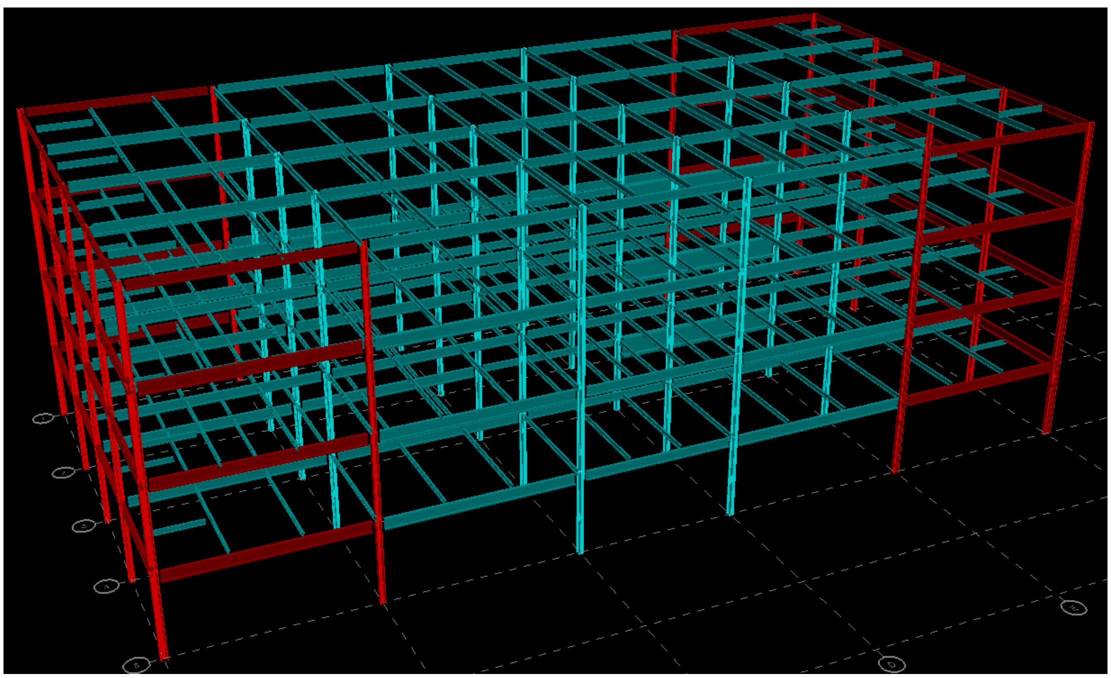
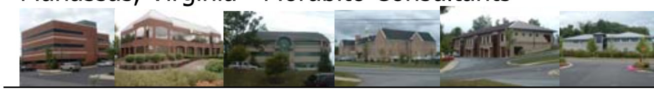


Figure 9. RAMSteel Model, with Frames highlighted in red.

<u>Moment Frame Capacity</u>

Using the Steel adequacy feature of RAMSteel, with an LRFD analysis, it was determined that:

- The four frames in the east-west direction were adequate for lateral loads, with most members sufficiently over designed. Even the critical member determined from the STAAD analysis was sufficient in the model.
- The upper floors of the two north-south frames were less overdesigned, but still adequate for the given lateral loads. However, as shown in Figure 10, the columns at the base are insufficient.



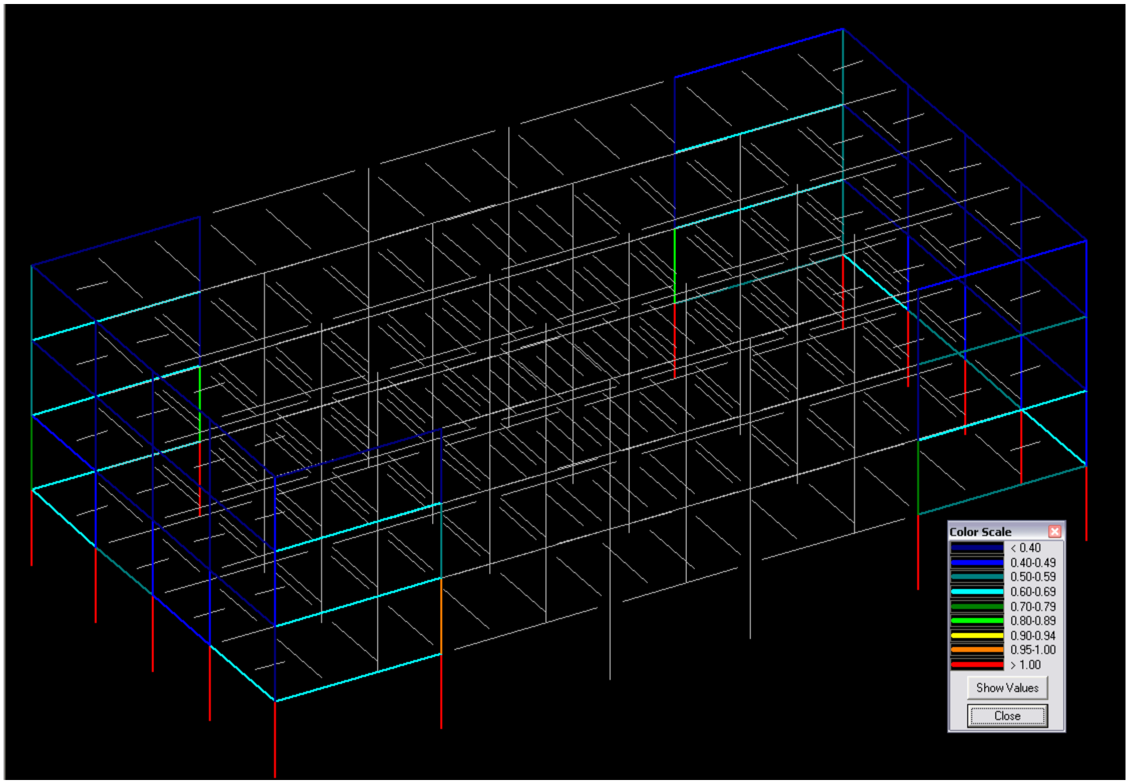


Figure 10. Visual Representation of Frame Capacity in North-South Direction.
The colors range from blue to red to represent extreme over capacity to failure, respectively.

Though there are many possible reasons for this finding including appropriate data entry, load application, and story data, it is interesting to note that only the columns at the base of the frame were insufficient. This indicates that possibly the support conditions were not an accurate portrayal of the connection from the building to the first floor diaphragm; moment frames and shear walls extending into the underground parking area would absorb some of the large moments affecting those columns.

Drift

Story drifts are summarized in Table 1. Given the allowable H/400 drift limit of 1.6", these are too large.

Story	Δ, N-S Loading (in)	Δ, E-W Loading (in)
2	0.46	1.09
3	1.27	1.99
4	1.98	2.83
R	2.20	2.63

Compared to the Fleischer Drift analysis, the 2.20" value for the N-S direction is very similar to the 2.28" found under Seismic Total Drift. Perhaps error in diaphragm weight or frame stiffness calculation affected these values.

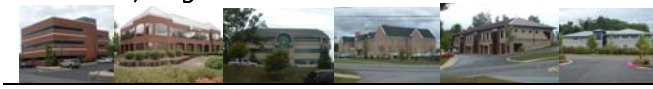
CONCLUSION

Two methods were used to assess lateral force resistance in the two perimeter moment frame systems in the Signal Hill Professional Center. The first employs a STAAD analysis of each moment frame to determine stiffness for lateral force distribution, and then combines lateral forces and gravity loads from a previous RAMSteel model to determine maximum moments and shears. The second alters the original RAMSteel model to incorporate the two moment frames and lateral loads.

These two analyses revealed that:

- Generally, members in both moment frames were sufficient for the given lateral loadings. Where one beam was at over capacity in the STAAD Analysis, the same beam was well within its strength in the RAM model. Most members were sufficiently over-designed.
- More attention needs to be brought to the connection between the moment frame columns and the first floor / parking diaphragm. After modeling only the top four floors in RAM with approximated pin connections at the first floor diaphragm, all columns at the first floor were at over-capacity. Possibly, moment distribution to moment frames in the underground parking structure would help reduce moments in these members.
- The large length and thickness of the basement walls proves that shear forces will not control basement wall reinforcement design. Rather, lateral earth pressures will direct design. The piers adjoining the basement walls, assumed to take all vertical loads from the moment frames, were determined sufficient for both the maximum compressive and tensile forces. Like in the basement walls, lateral earth pressures will again play a larger role.
- Due to the relatively low height, small lateral forces, and large building weight, overturning moment will not be a concern in structural design.
- Drifts were calculated both on RAMSteel and by hand using a Fleischer Drift Analysis. Both determined maximum drift to be around 2.3", which is greater than the accepted maximum drift of H/400 (1.6"). This discrepancy in drift control is most likely due to improper modeling of support conditions; as the supports range from pinned to fixed, drift should reduce.

See the Appendix for hand calculations.



APPENDIX A. LOADING CALCULATIONS.

WIND LOAD CALCULATIONS — ASCE 7 02

BASIC WIND SPEED: 90MPH = V (MANASSAS, VA) FIG 6-1

WIND DIR. FACTOR: $K_D = 0.85$ FOR MWFRS

IMPORTANCE: $I = 1.0$ (SPECS)

EXPOSURE CATEGORY: B (SPECS)
$z_g = 1200$ $\alpha = 7.0$ TBL 6-2 $z = 53.33'$ MAX
$K_z = 2.01(z/z_g)^{2/\alpha} = 0.8257 = q_H$
$K_H = 0.83$ (TBL 6-3)

TOPOGRAPHIC FACTOR: $K_{zt} \sim 1.0$ (SLOPING SITE, NO IRREGULARITIES)

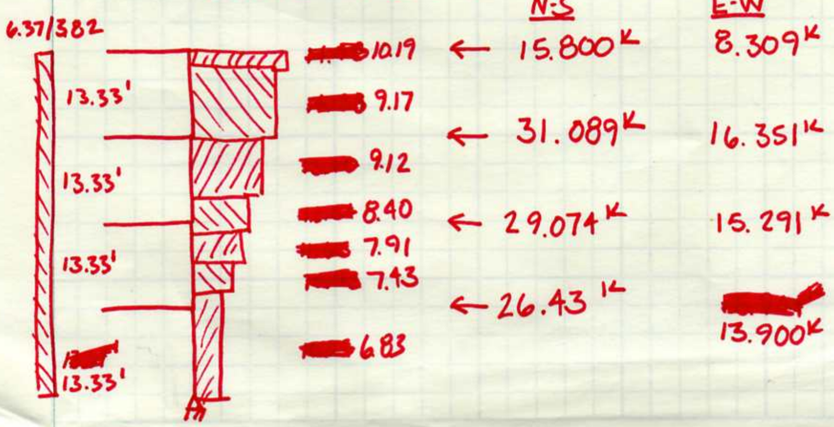
GUST EFFECTS: $G = 0.85$ (RIGID STRUCTURE)

WALL PRESSURE COEFF: (COMPLETELY ENCLOSED)
$C_p = 0.8$ (q_z) WINDWARD
$= -0.5$ (q_h) LEEWARD (N-S DIR)
$= -0.3$ (q_h) LEEWARD (E-W DIR)

VELOCITY PRESSURE: $q_z = 0.00256 K_z K_{zt} K_D V^2 I$

HEIGHT (FT)	K_z	q_z (PSF)	GC_pq (WINDWARD)	GC_pq (LEEWARD)
0-15	0.57	10.05	6.83	-6.37 / -3.82
15-20	0.62	10.93	7.43	
20-25	0.66	11.63	7.91	
25-30	0.70	12.34	8.40	
30-40	0.76	13.40	9.12	
40-50	0.81	14.28	9.71	
50-53.33	0.85	14.98	10.19	N-S / E-W

N-S DIR: 147'-8" WIDE
E-W DIR: 77'-8" WIDE



APPENDIX A, CONTD.

SEISMIC LOADING. EQUIVALENT LATERAL FORCE PROCEDURE
IBC 2003

SEISMIC USE GROUP I
$I = 1.0$
$S_{DS} = 0.186$
$S_{D1} = 0.065$
SITE CLASS "D"
$R = 3.0$

$V = C_s W$

STRUCTURE WT: DL + 10PSF FOR PARTITIONS
10875 [redacted] SQ. FT. ROUGH FLOOR AREA, 440FT PERIMETER

ROOF: 40 PSF (10875) + 440 PLF (440) = 629 K
FLOORS 2-4 80PSF(10875) + 440 PLF (440) = 1064 K
Σ 4883 K

$$C_s = \frac{S_{DS}}{R/I} = \frac{0.186}{3} = 0.062 \leftarrow$$

$$\frac{S_{D1}}{T(R/I)} = \frac{0.065}{0.67(3)} = 0.032$$

$$0.044 S_{DS} I = 0.044(0.186) = 0.008$$

$T = C_t h_n^x$ $C_t = 0.028$, $x = 0.80$ [MOMENT RESISTING STEEL]
$h_N = 53'\text{-}0$
$T = 0.67s.$

$V = C_s W = 0.062(4883) = 302^K$ BASE SHEAR

$F_x = C_{vx} V$, USING 170 K BASE SHEAR IN SPECS.

$$C_{vx} = \frac{w_x h_x}{\Sigma w_x h_x}$$

FLOOR	$w_x h_x$
R	33337
4	42206
3	28019
2	13832
Σ	117394

$F_R = \cancel{85} \; 48.3^K$
$F_4 = 61.1^K$
$F_3 = 40.6^K$
$F_2 = 20.1^K$

APPENDIX B. LATERAL FORCE RESISTING SYSTEM CALCULATIONS.

MOMENT FRAME CALCULATIONS.

ANALYSIS 1, NS FRAME.

④ $P_u/\phi P_n < 0.2$ $\quad \frac{18.92}{2(310)} + \left(\frac{29}{186}\right) = 0.187 < 1$ ✓

⑤ $P_u/\phi P_n < 0.2$ $\quad \frac{23.5}{2(275.8)} + \left(\frac{40.8}{123}\right) = 0.375 < 1$ ✓

⑬ $P_u/\phi P_n > 0.2$ $\quad \frac{330}{995} + \frac{8}{9}\left(\frac{279}{541}\right) = 0.790 < 1$ ✓

⑰ $P_u/\phi P_n < 0.2$ $\quad \frac{99}{2(552.5)} + \left(\frac{340}{359.0}\right) = 0.956 < 1$ ✓

ANALYSIS 1, EW FRAME

④ $P_u/\phi P_n < 0.2$ $\quad \frac{18.9}{2(281)} + \left(\frac{47.0}{188.0}\right) = 0.283 < 1$ ✓

⑤ $P_u/\phi P_n < 0.2$ $\quad \frac{104}{2(326.4)} + \left(\frac{69.4}{166.0}\right) = 0.577 < 1$ ✓

⑭ $P_u/\phi P_n < 0.2$ $\quad \frac{5.9}{2(552.5)} + \left(\frac{319.2}{359}\right) = 0.894 < 1$ ✓

㉚ $P_u/\phi P_n > 0.2$ $\quad \frac{189.5}{625} + \frac{8}{9}\left(\frac{230.5}{302.0}\right) = 0.982 < 1$ ✓

ANALYSIS 2, NS FRAME

㊹ $P_u/\phi P_n < 0.2$ $\quad \frac{1}{2(501)} + \left(\frac{110.3}{294}\right) = 0.376 < 1$ ✓

㊺ $P_u/\phi P_n > 0.2$ $\quad \frac{330}{966} + \frac{8}{9}\left(\frac{144.5}{542}\right) = 0.579 < 1$ ✓

ANALYSIS 2, EW FRAME.

㊳ $P_u/\phi P_n < 0.2$ $\quad \frac{8.7}{2(77.8)} + \left(\frac{64.8}{578}\right) = 1.127 > 1$ X

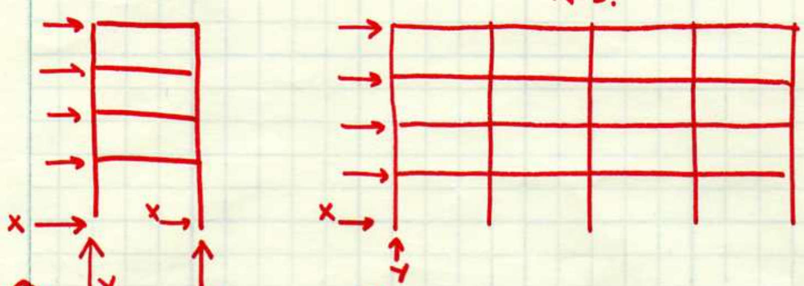
㊴ $P_u/\phi P_n > 0.2$ $\quad \frac{330}{817} + \frac{8}{9}\left(\frac{220}{408}\right) = 0.883 < 1$ ✓

APPENDIX B, CONTD.

SHEAR WALL ANALYSIS

REACTIONS FROM FRAMES (K)

E-W. N-S.



E-W:

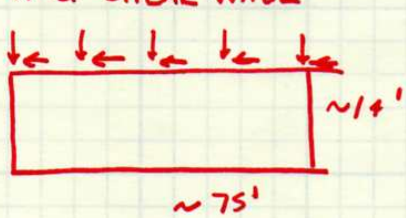
	$\boxed{A1, B1, E1, F1}$	
X	1.7D, 1.4L, -12.2W, -23.8E	-1.7D, -1.4L, -9.4W, -18.0E
Y	41.2D, 33.1L, -22.6W, -53.1E	41.2D, 33.1L, 22.6W, 53.1E

N-S:

	A1	A2	A3	A4	A5
X	1.1D, 0.6L, -16.7, -17.0	-0.2D, -0.1L, -18.6W, -19.3E	0.1D, 0.1L, -13.7W, -14.3E	0.3D, 0.2L, -14.8W, -16.4E	-1.2D, -0.7L, -18.7W, -19.2E
Y	42.0D, 23.2L, -39.9W, -48.9E	83.5D, 46.1L, 9.7W, 11.3E	74.1D, 40.9L, 2.4W, 2.8E	83.1D, 45.9L, -10.6W, -12.0E	42.2D, 23.3L, 38.5W, 46.9E

E-W. SHEAR WALL

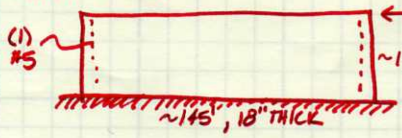
N-S. SHEAR WALL



APPENDIX B, CONTD.

LATERAL LOAD TO BASEMENT STRUCTURE

NORTH-SOUTH



CANTILEVER BEAM. SHEAR $V_c = 2\sqrt{f'_c}\,bh = 2\sqrt{3000}\,(12)(145)(18) = 3431^K$

$V_u = 132^K < V_c$ ✓

FLEXURE $M_u = (132)(13.75) = 1815$ FT-K

$A_s = \frac{M_u}{\phi F_y (d - \frac{a}{2})}$ $a \sim 0.2d$, $d \sim 145' - 6' = 139'$

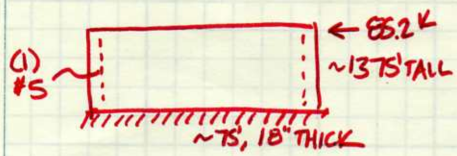
$= \frac{1815(12)}{0.9(60)(0.9)(139)(12)} = 0.269\ IN^2$

USE #5 $A_s = 0.31\ IN^2$

$a = \frac{(0.31)(60)}{0.85(3)(18)} = 0.405228'' < 27.8'$ ✓

BY INSPECTION, #4 WOULD WORK (MINIMUM SHRINKAGE/TEMPERATURE REINFORCEMENT WILL CONTROL)

EAST-WEST



SHEAR $V_c = 2\sqrt{f'_c}\,bh = 2\sqrt{3000}\,(12)(75)(18) = 1774.6^K$

$V_u = 85.2^K < V_c$ ✓

FLEXURE $M_u = (85.2)(13.75) = 1172$ FT-K

$A_s = \frac{1172(12)}{0.9(60)(69)(12)}$ $0.31\ IN^2$ $d \sim 75 - 6 = 69'$, $a = 0$

USE #5 $A_s = 0.31\ IN^2$

BY INSPECTION, MAXIMUM SHRINKAGE/TEMPERATURE REINFORCEMENT WILL CONTROL

APPENDIX B, CONTD.

LATERAL LOAD TO BASEMENT STRUCTURE, CONTD

CRITICAL PIERS (24" SQUARE)

MAX UPLIFT FORCE, PIER #A1, 1.2D+0.5L+1.6W CONDITION

$F = -10.24^K$

$F_{TCE} = 7.5\sqrt{f'c} = 7.5\sqrt{3000} = 410\ psi$

$F_T = (10.24)(1000)/(24^2) = 17.8\ psi$ ✓

MAX COMPRESSIVE FORCE, PIER #A2, 1.2D+1.6L CONDITION

$F = 174^K$

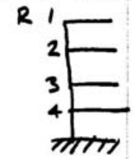
F_{cr} = (ASSUME FULLY BRACED INTEGRALLY WITH WALL)

$= 0.85(3000)(24^2) = 1469^K$

$1469 > 174$ ✓

APPENDIX C. FLEISCHER SIMPLIFIED DRIFT CALCULATIONS.

FLEISCHER SIMPLIFIED DRIFT ANALYSIS



H = 53.3 FT
n STORIES = 4
$\bar{D}_T$ = 53.3(12)/400 = 1.6" ← MAXIMUM ALLOWABLE
ω_1 (@ ROOF) = 147.67 (WIDTH)/2 [0.01019 PSF] = 0.752 KLF
ω_{30} (@ 30') = 147.67 (WIDTH)/2 [0.00912 PSF] = 0.674 KLF

$$a = \frac{\log(0.752/0.674)}{\log(53.3/30)} = \frac{0.04756}{0.2496} = 0.19$$

C = COLUMN BELOW
G = GIRDER

I_G
(1): 2(199/20) + 2(199/17.5) = 42.6
(3): 2(612/20) + 2(510/17.5) = 119.5 IN⁴/FT
(4): 2(843/20) + 2(612/17.5) = 154.3

I_C
(1): (4/53.3) [2(307) + 3(171)] = 84.6
(3): (4/53.3) [533 + 394 + 303 + 341 + 475] = 153.6 IN⁴/FT
(4): (4/53.3) [662 + 534 + 2(394) + 833] = 211.4

I_B $\Sigma A c e_x^2$
(1): (2/144) [(11.7)(37.5²) + (9.71)(17.5²)] = 270
(3): (1/144) [(19.1)(37.5²) + (20)(17.5²) + (17.6)(17.5²) + (17)(37.5²)] = 433
(4): (1/144) [(23.2)(37.5²) + (25.9)(17.5)² + (20)(17.5²) + (28.2)(37.5²)] = 600 FT⁴

MATERIAL DISTRIBUTION FACTORS
b_G = $n(I_{G1})/I_{G30}$ = (4)(42.6)/154.3 = 1.10
b_C' = $n I_{C1}/I_{C30}$ = (4)(84.57)/211.4 = 1.60
b_C'' = $n I_C''_1/I_C''_{30}$ = (4)(270)/600 = 1.80

$$C_G = \frac{\log(119.5-42.6 / 154.3-42.6)}{\log(3-1/4-1)} = 0.921$$

$$C_C' = \frac{\log(153.55-84.57 / 211.4-84.57)}{\log(3-1/4-1)} = 1.502$$

$$C_C'' = \frac{\log(433-270/600-270)}{\log(3-1/4-1)} = 1.7396$$

FROM TABLES, 3 4 5 :
$\phi_G \sim 0.594$
$\phi_C' \sim 0.753$
$\phi_C'' \sim 0.846$

APPENDIX C, CONTD.

FLEISCHER SIMPLIFIED DRIFT ANALYSIS, CONT'D

USING SEISMIC LOADS.

BASE SHEAR. 170^K $\quad H/L = 53.33/75 = 0.711$

$\phi_G \sim 0.704$

$\phi_C' \sim 0.916$

$\phi_C'' \sim 1.026$

GIRDER MOMENT DRIFT

$$D_G = \frac{VH^2}{134\,n\,I_{G(N)}}\phi_G = \frac{(\overset{48.3}{[redacted]})(53.33^2)(0.704)}{134(4)(154.3)} = [redacted]\ 1.17''$$

COLUMN MOMENT DRIFT

$$D_C' = \frac{VH^2}{134\,n\,I_{G(N)}}\phi_C = \frac{(\overset{48.3}{[redacted]})(53.33^2)(0.916)}{134(4)(211.4)} = [redacted]\ 1.11''$$

COLUMN AXIAL FORCES

$$D_C'' = \frac{V}{I_{B(N)}}\left(\frac{H}{112}\right)^3\phi_C'' = \frac{\overset{48.3}{[redacted]}}{600}\left(\frac{53.3}{112}\right)^3(1.026) = [redacted] = 0.0089$$

TOTAL DRIFT = ΣD = [redacted]
2.28''

APPENDIX C, CONTD.

FLEISCHER SIMPLIFIED DRIFT ANALYSIS, CONTD

WEIGHTS:

$q_{L(30)} = (1333)[79+88+68+68+96] = 5320 \text{ lb}$

$q_{G(30)} =$ [redacted] $2(20)(44)+2(17.5)(40) = 3160 \text{ lb}$

GIRDER MOMENT DRIFT

$$D_G = \frac{w_1 H^3}{201 n I_{G(n)}}(\phi_G) = \frac{0.752(5333^3)}{201(4)(154.3)}[0.597] = 0.546'$$

COLUMN MOMENT DRIFT

$$D_C' = \frac{w_1 H^3}{201 n I_{C(n)}}(\phi_C') = \frac{0.752(5333^3)}{201(4)(211.4)}[0.753] = 0.505''$$

COLUMN CHORD DRIFT

$$D_C'' = \frac{w_1(H/38)^4}{I_{B(n)}}[\phi_C''] = \frac{0.752(5333/38)^4}{600}[0.846] = 0.004''$$

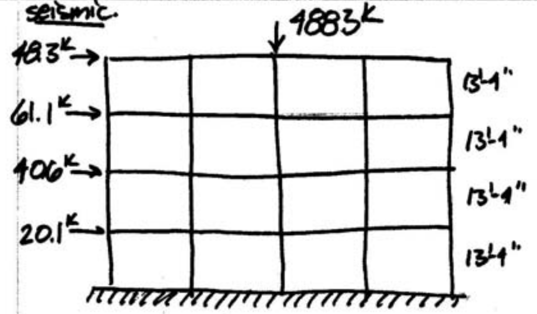
TOTAL DRIFT $= \Sigma D = 0.546 + 0.505 + 0.004 = 1.055'' < \bar{D}_T = 1.6$

✓OK

APPENDIX D. OVERTURNING MOMENT CALCULATIONS.

CALCULATION OF OVERTURNING MOMENT

seismic



LATERAL MOMENT = (48.3)(53.33) + (61.1)(40) + (40.6)(26.67) + (20.1)(13.33) = 6370.6 k

RESISTANCE OF DEAD LOAD: (48.83)(37.5) = 183112.5 FT-K

UPLIFT = (30PSF)(77.67)(147.67)(37.5) = 12903 FT-K

6370.6 + 12903 = 19274 FT-K << 183112 FT-K ✓ OVERTURNING NOT A PROBLEM