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Technical Report #3

Lateral System

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Executive Summary

Technical Report #3 covers the lateral system of the Arizona State University Southwest Student Housing building in greater depth. An ETABS model was created for this report, using several assumptions, such as rigid diaphragm floor systems and a 50% reduction of gross-section properties to account for concrete cracking. The Southwest Housing building consists of only one lateral system—the concrete cores that are also the gravity system for this building. There is bi-axial symmetry in the building plan, so both the center of rigidity and the center of mass are located on the centerlines of the building, assuming minimal errors or imperfections in construction.

It was ultimately determined that Case 2 of the wind load chapter of ASCE 7-05 (chapter 6) controlled design drift, shear and moment. The overall drift of the building was compared to serviceability limits and found adequate, with the maximum drift reaching approximately 0.03”, well under the 6.24” limit. The potential for overturning due to lateral loads was also examined and found to be negligible, due to the self-weight of the building counteracting the overturning moment. The maximum base shear was calculated to be 424 kips for Case 2, which was found to be well under the 2060 kip shear capacity (see Appendix G for concrete core capacity calculations).

Appendices E and F provide all of the necessary loads, tables and factors to follow the calculations and results.

Introduction

The Southwest (SW) Student Housing building is a 20-story high-rise for students attending Arizona State University. The building site is located in a downtown area, at 1000 Apache Blvd. East in Tempe, Arizona (see Figure 1, the site is highlighted in red[1]). The building plans are designed to accommodate 528 beds in 268 units, with an emphasis on modularity for ease and economy of construction. There is additional potential to include an automated parking facility on the first level, which can be accounted for in the initial building design. A rendering of the potential building design can be observed on the front cover of this report.

Figure 1: Site Location, 1000 Apache Blvd. East, Tempe, AZ

This particular building has a unique structure designed for easy assembly on site to enable extremely fast and efficient construction. The building's gravity and lateral systems are one and the same: a series of three 8-inch thick concrete cores, 25' wide and 25' long. These cores are constructed first using slip-forms to within a 1/8" tolerance. The roof of the building is then assembled on the ground around the cores in two parts and lifted into place using six 75-ton strand jacks. Each subsequent floor is then assembled on the ground, half the floor area at a time (with the joint located at the precise halfway point of the floor plan, as indicated in figure 2), and lifted into place. The building is essentially constructed from the top, down.

The floors are constructed using metal deck with lightweight concrete and structural steel beams. Each floor has a similar and regular floor plan (and thus, loading), with residential areas for 23' on each side of a 6'-wide corridor running through the center of the building, lengthwise (see Figure 2 below).

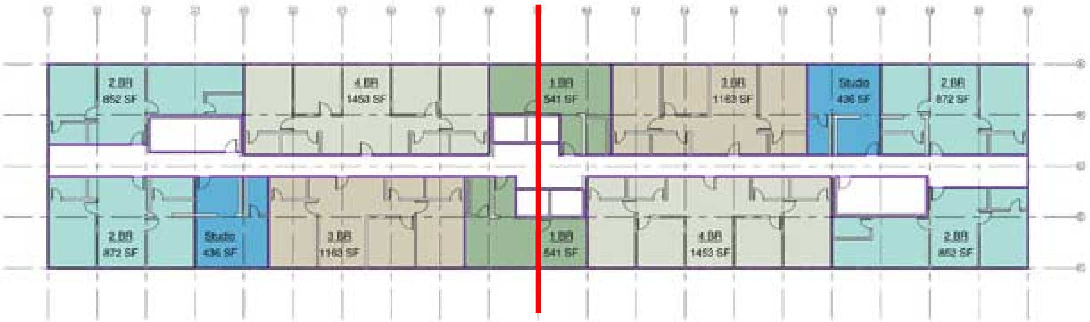

Figure 2: Typical Building Floor Plan

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[1] Taken from http://maps.google.com

Structural Systems

Foundation

The SW Student Housing building will exert significant loads to the foundation elements, according to the geotechnical report for the area. As a result, this building will require a deep foundation system that penetrates through to the second layer of soil on the site to limit settlement. The first layer of the site is Silty Sand and Poorly Graded Sand for a depth range from 10’ to 35’. The second layer of soil on the site is Sand Gravel Cobble, from a depth of 35’ to 100’.

The geotech report recommends drilled piers, with no pier shaft sized to a diameter of less than 12”. Each pier should penetrate at least twice the shaft diameter into the second layer of soil. The predicted settlement for this pier configuration is less than one inch for an isolated pier shaft with a diameter of less than 60”. A potential foundation layout is shown in Appendix I, with relevant calculations.

Floor System

The floor system is the same on all floors. This system consists of 3-1/4” lightweight concrete on 3” metal deck, with a minimum gage of 20. The composite deck is supported by a structural steel frame, with wide-flange sizes ranging from W14x22 infill beams to W24x176 interior girders, as prescribed by the typical framing shown in Figure 3, and reiterated in the notes included in Appendix A. All four girders span the length of the building (250’), and all typical load beams span the width of the building (52’). Infill beams span either 12’-6” or 24’, depending on their location within the building. The typical members are labeled in Figure 3. Every structural steel element in the typical frame is cambered. Some members are cambered up to 4 inches at the cantilevered ends (See Appendix A for the project structural engineer’s camber diagrams).

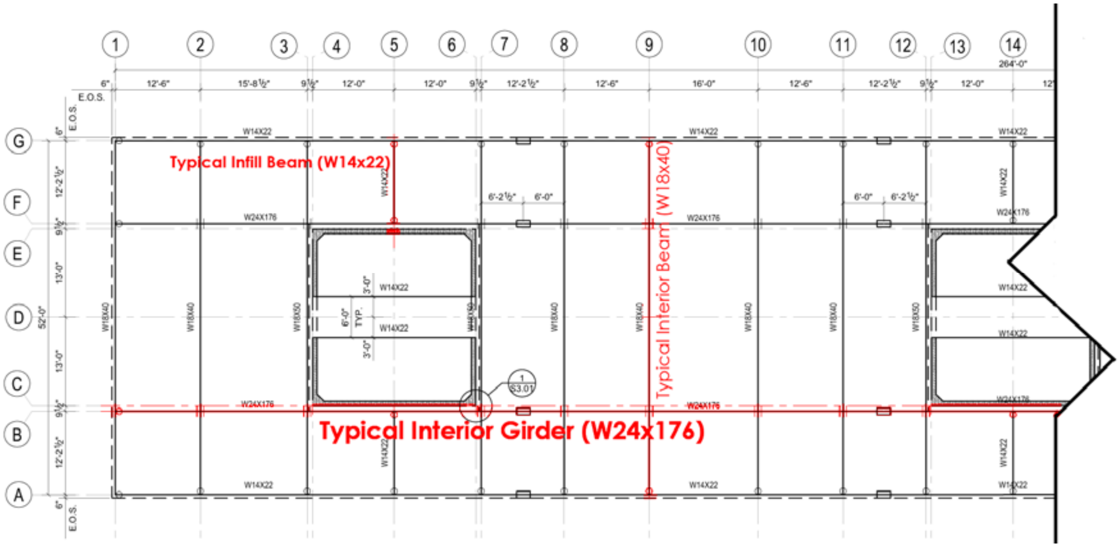
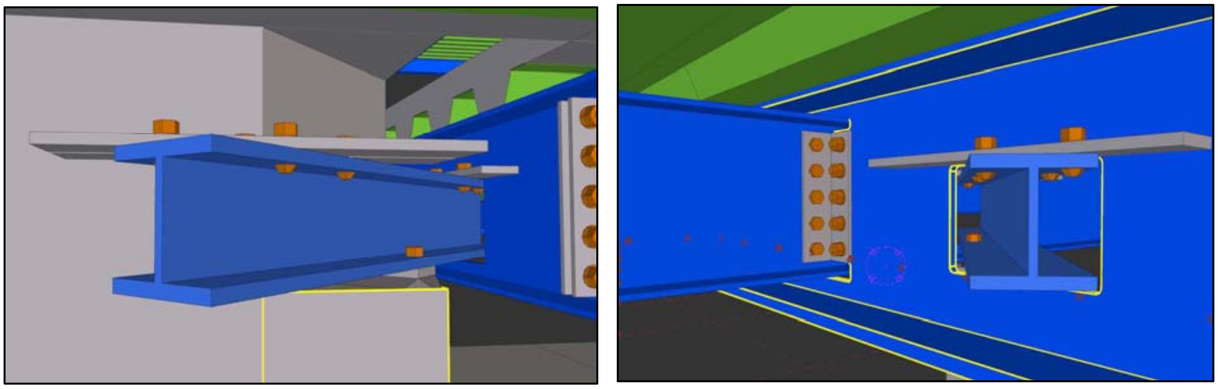


Figure 3: Typical Framing Plan (building is symmetric about line 14)

Gravity and Lateral System

Unlike some conventional construction, this building has no columns. The three 8-inch thick, 25'x25' (at the centerline) concrete cores carry all of the gravity weight of each floor. As a result, the floors are cantilevered off of the cores (spaced at 62'-6" on center), which support the structural steel floor framing via a wide-flange beam inserted through each of the four corners in every core, as illustrated in Figure 4. During construction, half of a floor is lifted via the 75-ton strand jacks and then fitted into place using the aforementioned corner details. The cores are designed as walls using ACI 318-05. As a result, each core has a minimal amount of reinforcement through the center (one layer of the smallest permitted rebar size by code).



Figures 5.1 and 4.2: Corner detail at every floor, framing into the interior girder to support each level

The concrete cores are also the building's sole lateral system, and provide lateral bracing in both directions in the form of shear walls. For clarity, the cores are highlighted with red in the typical building floor plan below in Figure 5, with boundaries at openings selected. It can be observed in Figure 6 on the next page that the openings are only present for a minimal height on each floor so that the shear walls can be reunited via large coupling beams for added rigidity and support.

Figure 4: Typical Building Floor Plan (Core areas are highlighted in red, core walls are highlighted in green)

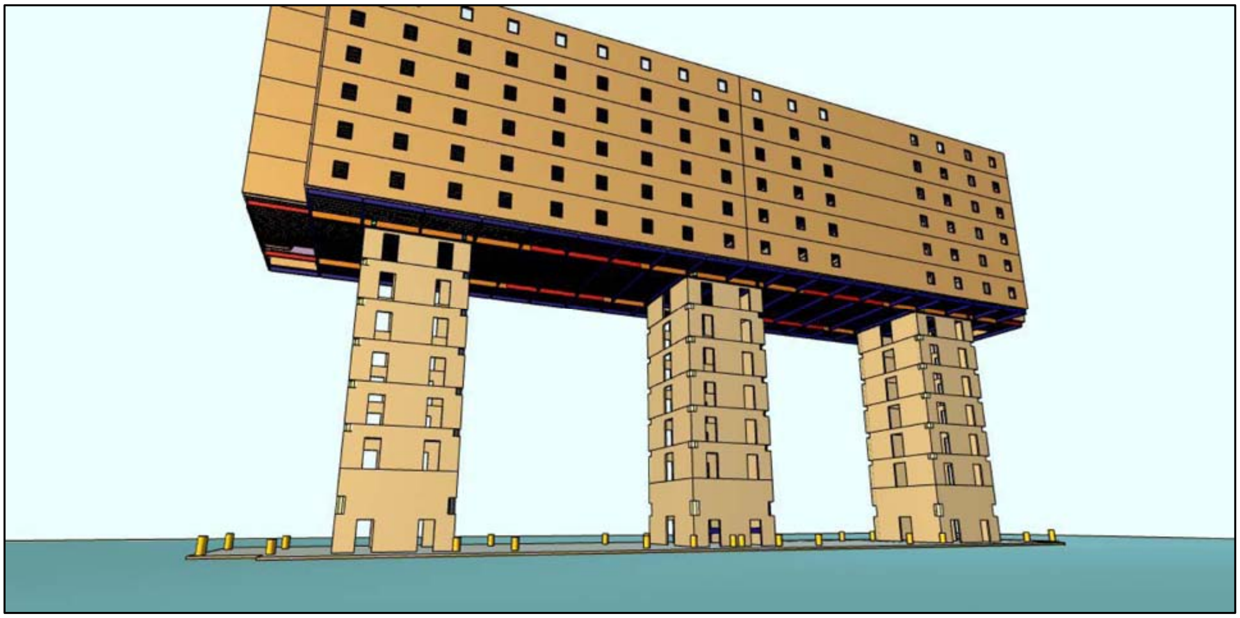


Figure 6: Rendering of visible openings in concrete cores

The theory behind this building design seems to be simplicity: a single set of structural elements to resist all loading. The sizing of these elements was carried out using a combination of hand calculations employing ASD, and computer modeling for more precise answers. ASD hand calculations were found to be generally with 10% of the computer modeling outputs, which used the LRFD method of design.

Roof System

The roof system is a simple, long-lasting construction of the typical floor framing (3-1/4” lightweight concrete with 3” metal deck, minimum 20 gage), 3” of rigid insulation and an Ethylene Propylene Diene Terpolymer (EPDM) membrane on top. There is no mechanical equipment on the roof- the major mechanical elements will be located on the ground floor, and will serve each unit in the building via a 2-pipe system.

Codes, References and Standards

Building Design Codes:

Model Code:

International Building Code, 2006 Edition, as amended by the city of Tempe, AZ

Design Codes:

American Institute of Steel Construction "Specifications for Structural Steel Buildings", AISC 360-05

American Concrete Institute "Building Code Requirements for Structural Concrete", ACI 318-05

Structural Standards:

American Society of Civil Engineers "Minimum Design Loads for Buildings and other Structures", ASCE7-05

Thesis Codes:

Model Code:

International Building Code, 2006 Edition

Design Codes:

American Institute of Steel Construction "Specifications for Structural Steel Buildings", AISC 360-05 (13th ed.) and AISC 360-10 (14th ed.)

American Concrete Institute "Building Code Requirements for Structural Concrete", ACI 318-05

Structural Standards:

American Society of Civil Engineers "Minimum Design Loads for Buildings and other Structures", ASCE7-05

Deflection Criteria:

Limit Unfactored Live Load deflections to L/360 or less

Limit Total (Service) Load deflections to L/240 or less

Limit building drift to h/400 or less for wind, and adhere to ASCE 7-05 for seismic limits

Materials

Structural Steel:

- All Rolled Shapes – ASTM A992 Grade 50
- All Plates and Connection Material – ASTM A36
- All Tubular Sections – ASTM A500 Grade B
- All Pipe Sections – ASTM A53 Grade B
- Anchor Rods – ASTM F1554

Cast-in-Place Concrete:

- Foundations – 4000 psi normal weight
- Slab on Grade – 4000 psi normal weight
- Structural Slab on Grade – 5000 psi normal weight
- Lightweight Concrete – 4000 psi
- Walls (core) – 4000 – 5000 psi

Reinforcement:

- Deformed Bars – ASTM A615 Grade 60 typ.
- Welded Wire Fabric – ASTM A195

Welding Electrodes:

- E70xx Low Hydrogen

Bolting Materials:

- ASTM 325 or A490

Load Calculations

Gravity Loads

See Appendix B for all calculations, including confirmation of structural steel allowance from typical framing plan and citations for calculating snow load.

Construction Dead Load:

3" Metal Deck (20 gage)	2.14	psf
3-1/4" Lightweight Concrete (110 PCF)	46	psf
Structural Steel Allowance	11	psf
Sum (CDL)	**59.14**	**psf**

Superimposed Dead Load:

Assumed, according to structural engineers	15	psf
Sum (SDL)	**15**	**psf**

Live Loads:

Building uses

Residential	40	psf
Parking	40	psf
Corridors	80	psf
Live Load (LL)	**80**	**psf**

Wall Loads:

Curtain Wall	15	psf
Sum	**15**	**psf**

Snow Loads:

Ground snow load for region	0	psf
Sum	**0**	**psf**

Lateral Loads

Wind Loads

Due to this building not meeting criteria for the simplified method of analysis (Method 1 – Simplified Procedure), wind loads for this structure were analyzed using Method 2 – Analytical Procedure, which can be found in Chapter 6, section 6.5 of ASCE7-05. Supplemental calculations to justify values in the following tables can be found in Appendix C.

The regularity and simple form of this building allowed for ease in calculating maximum wind pressures (in the East-West direction of the building, along the longer axis). The wind pressures were found to be greatest on the East-West side because of the large exposure of the façade. The length of the building is 250', so a total area of 208'x250'=52,000 square feet of façade is exposed on the E-W side to wind. On the N-S side, only 208'x52'=10,816 square feet of façade is exposed to wind (approximately one fifth of the E-W façade). As a result of the greater wind pressures, the base shear controlled in the N-S direction. Tables 1 and 2 below show the pressures and forces acting on the building due to wind pressure in both the E-W and N-S directions:

Table 1: Coefficients for wind analysis and wind pressures

C_p	N-S	E-W
Windward	0.8	0.8
Leeward	-0.5	-0.2

	Story	Height h_x (ft)	K_z	q_z	Wind Pressures (psf)	
					N-S	E-W
Windward	**Roof**	208	1.218	21.47	**14.60**	**14.60**
	20	198	1.201	21.17	**14.40**	**14.40**
	19	188	1.184	20.86	**14.19**	**14.19**
	18	178	1.165	20.54	**13.97**	**13.97**
	17	168	1.146	20.20	**13.74**	**13.74**
	16	158	1.126	19.85	**13.50**	**13.50**
	15	148	1.105	19.48	**13.25**	**13.25**
	14	138	1.083	19.10	**12.99**	**12.99**
	13	128	1.060	18.69	**12.71**	**12.71**
	12	118	1.036	18.26	**12.42**	**12.42**
	11	108	1.010	17.81	**12.11**	**12.11**
	10	98	0.983	17.32	**11.78**	**11.78**
	9	88	0.953	16.79	**11.42**	**11.42**
	8	78	0.921	16.22	**11.03**	**11.03**
	7	68	0.885	15.60	**10.61**	**10.61**
	6	58	0.846	14.91	**10.14**	**10.14**
	5	48	0.801	14.12	**9.60**	**9.60**
	4	38	0.750	13.21	**8.98**	**8.98**
	3	28	0.687	12.11	**8.23**	**8.23**
	2	18	0.605	10.67	**7.26**	**7.26**
Leeward	All	**All**	1.218	21.47	**-9.13**	**-3.65**

Table 2: Lateral forces, story shear and moment from wind analysis

E-W Width	250	ft
N-S Width	52	ft

Story	Height h_x (ft)	Lateral Force F_x (k)		Story Shear V_x (k)		Moment M_x (ft-k)	
		E-W	**N-S**	**E-W**	**N-S**	**E-W**	**N-S**
Roof	208	**6.17**	**18.25**	0.00	0.00	1283	**3796**
20	198	**7.49**	**35.99**	6.17	18.25	1482	**7127**
19	188	**7.38**	**35.46**	13.66	54.24	1387	**6667**
18	178	**7.26**	**34.91**	21.03	89.71	1293	**6215**
17	168	**7.14**	**34.34**	28.29	124.62	1200	**5769**
16	158	**7.02**	**33.75**	35.44	158.96	1109	**5332**
15	148	**6.89**	**33.12**	42.46	192.71	1020	**4902**
14	138	**6.75**	**32.47**	49.35	225.83	932	**4480**
13	128	**6.61**	**31.77**	56.10	258.29	846	**4067**
12	118	**6.46**	**31.04**	62.71	290.07	762	**3663**
11	108	**6.30**	**30.27**	69.16	321.11	680	**3269**
10	98	**6.12**	**29.44**	75.46	351.38	600	**2885**
9	88	**5.94**	**28.55**	81.58	380.82	523	**2512**
8	78	**5.74**	**27.58**	87.52	409.37	447	**2151**
7	68	**5.52**	**26.52**	93.26	436.95	375	**1803**
6	58	**5.27**	**25.34**	98.78	463.48	306	**1470**
5	48	**4.99**	**24.01**	104.05	488.82	240	**1152**
4	38	**4.67**	**22.46**	109.04	512.83	178	**853**
3	28	**4.28**	**20.58**	113.71	535.29	120	**576**
2	18	**1.89**	**9.07**	117.99	555.87	34	**163**
	Sum	**120**	**565**	**120**	**565**	**14815**	**68855**

The final values in Table 2 provide confirmation that the N-S direction has higher base shear, which is the result of the considerably larger façade area in that direction when compared to the E-W direction. The base shear in the N-S direction is almost 5 times as large as the base shear in the E-W direction. The following figures show the summary diagram of the final calculated wind loads on the building in each direction.

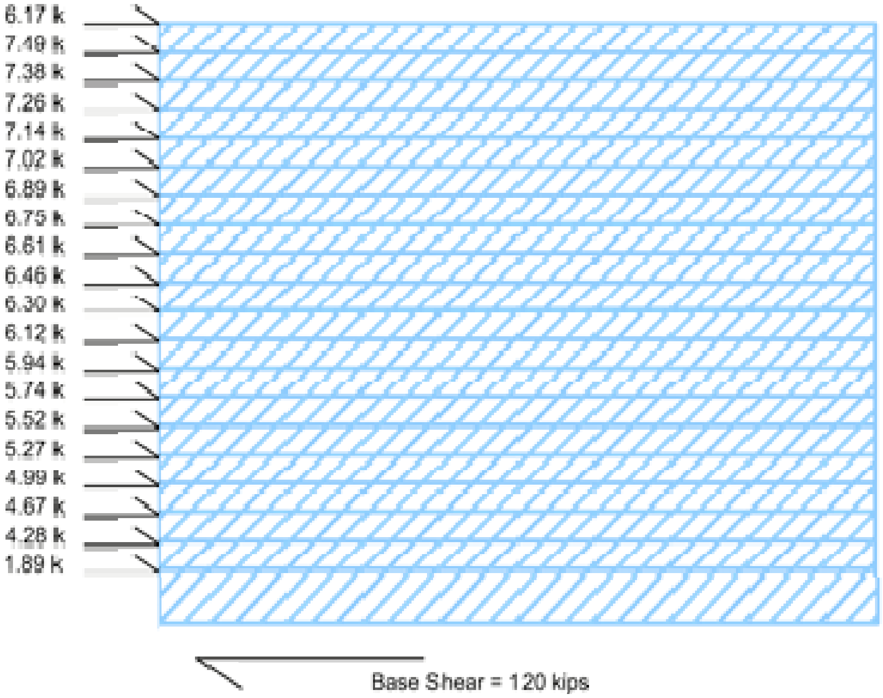


Figure 7: East-West direction wind forces at each story

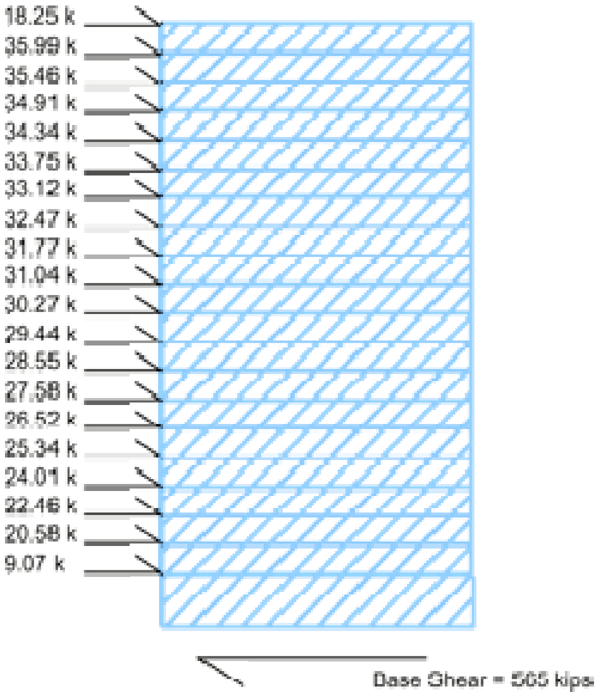


Figure 8: North-South direction wind forces at each story

Seismic Loads

The engineers that designed this building used the equivalent lateral force method to analyze seismic loads. As a result, this thesis also uses equivalent lateral force method for analysis. All loads were calculated using provisions from Chapters 11 and 12 of ASCE7-05. All coefficient calculations and sample load calculations can be found in Appendix D. Table 3 shows the load distribution under seismic loading, as well as several essential coefficients for the calculations.

Ultimately, it can be seen that wind loads govern this building design. The seismic base shear is 235.5 kips, as opposed to the maximum wind base shear of 565 kips (a little under twice as much). Wind loading also produces significantly higher moments, as well as higher story forces. Seismic story forces only control on the roof level, where seismic loading produces a 29k load while wind loading produces a maximum of 18.25k

One thing to note is that this building, though made of concrete, is not as heavy as a conventional concrete building would be. According to the engineers that designed this system, if this building were made of conventional concrete, it would be almost twice as heavy. The result of this increase in mass and weight would be a drastic increase in the seismic base shear, which currently does not govern building design.

Table 3: Essential coefficients and calculated seismic loads on each story

T=	1.100	s
k=	2.000	
V_b=	235.5	kips

Story	Height h_x (ft)	Weight w_x (k)	$w_xh_x^k$	C_{vx}	Lateral Force F_x (k)	Story Shear V_x (k)	Moment M_x (ft-k)
Roof	208	962	41619968	0.124	**29**	0	**6087**
20	198	1052.6	41266130	0.123	**29**	29	**5745**
19	188	1052.6	37203094	0.111	**26**	58	**4918**
18	178	1052.6	33350578	0.100	**23**	84	**4174**
17	168	1052.6	29708582	0.089	**21**	108	**3509**
16	158	1052.6	26277106	0.078	**18**	129	**2919**
15	148	1052.6	23056150	0.069	**16**	147	**2399**
14	138	1052.6	20045714	0.060	**14**	163	**1945**
13	128	1052.6	17245798	0.051	**12**	178	**1552**
12	118	1052.6	14656402	0.044	**10**	190	**1216**
11	108	1052.6	12277526	0.037	**9**	200	**932**
10	98	1052.6	10109170	0.030	**7**	209	**697**
9	88	1052.6	8151334	0.024	**6**	216	**504**
8	78	1052.6	6404018	0.019	**5**	221	**351**
7	68	1052.6	4867222	0.015	**3**	226	**233**
6	58	1052.6	3540946	0.011	**2**	229	**144**
5	48	1052.6	2425190	0.007	**2**	232	**82**
4	38	1052.6	1519954	0.005	**1**	234	**41**
3	28	1052.6	825238	0.002	**1**	235	**16**
2	18	1215.68	393880	0.001	**0**	235	**5**
	S	**21124.48**	**334944008**	**1.000**	**236**	**236**	

Computer Model

The lateral system was analyzed in detail using ETABS. Figure 9 shows a rendering of the 3D model used in analysis. The load cases considered are listed below. See Appendix E and Appendix F for specific loads and explanation of cases for Seismic and Wind, respectively:

- Seismic loads in the North-South direction with 5% accidental torsion
- Seismic loads in the East-West direction with 5% accidental torsion
- ASCE 7 Chapter 6 Case 1 wind forces in each direction
- ASCE 7 Chapter 6 Case 2 wind forces in each direction
- ASCE 7 Chapter 6 Case 3 wind forces
- ASCE 7 Chapter 6 Case 4 wind forces

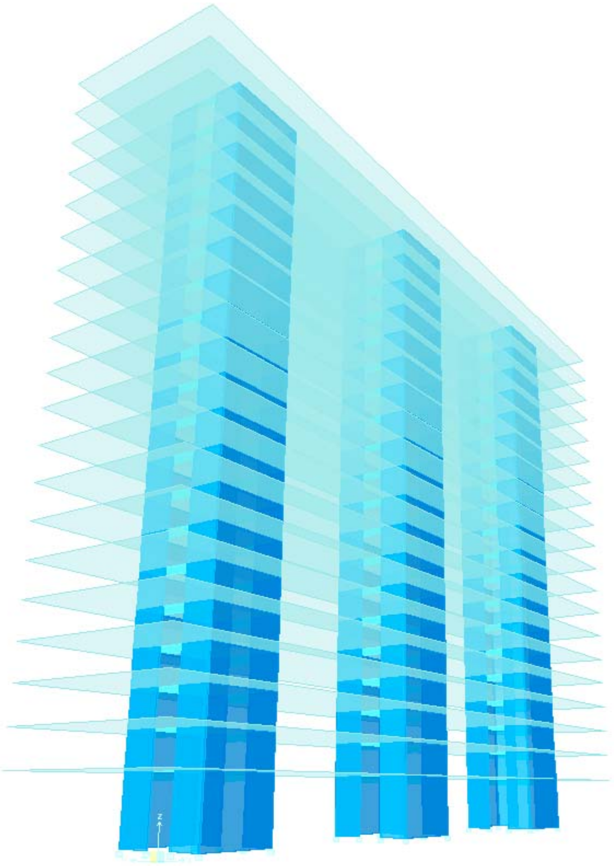


Figure 9: ETABS model used in analysis

Model Parameters for Analysis

By observation of symmetry in both the North-South and East-West directions, the center of mass and center of rigidity are both directly in the center of the building. Each concrete core was modeled as 2 C-shaped 8" shear walls connected with 2' thick coupling beams, located 6' apart to account for the corridor running East-West through the building (see Figure 10). The modeled material properties of the concrete correspond to the material properties stated in the Materials section of this document. Additionally, the concrete gross-section properties were reduced 50% to account for concrete cracking. Each floor level was modeled as a rigid diaphragm.

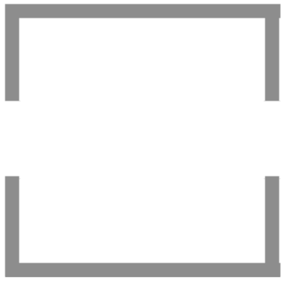


Figure 10: C-shaped portions of concrete cores (plan view)

Lateral Distribution and Modeling Analysis

For hand calculated seismic force distribution, it was observed that the distribution of forces to each shear wall in the lateral force resisting system is approximately even: every concrete core is symmetric, so it can be assumed that the loading the building experiences is symmetric. Tables 4 and 5 compare the hand-calculated lateral force distribution of loads to the ETABS model output for Story 10 (under seismic loading). Figures 11 and 12 provide keys for the labeling of the lateral force resisting system components for Tables 4 and 5. Story 10 was picked as an arbitrary middle point to confirm hand-calculated values via computer modeling procedures.

There are some differences between the hand-calculated loads and the ETABS-calculated loads. Effects such as torsion might have been estimated differently in each method, which could explain the differences. Appendix E and Appendix F contain tables of the overall story shear comparison for hand calculations and ETABS output.

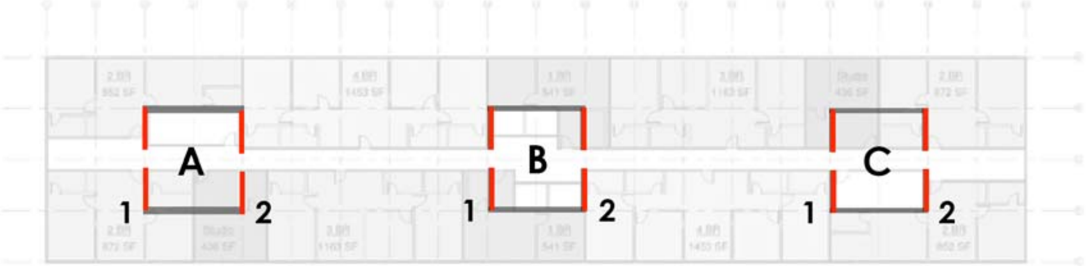


Figure 11: Labels for North-South lateral force resisting system components (highlighted in red)

Table 4: North-South direction shear in story 10 under the calculated seismic loading

North-South Shear - Story 10, Lateral Distribution

Method	A		B		C	
	1	2	1	2	1	2
ETABS	25.10	32.80	30.65	38.35	36.20	43.90
Hand	34.92	34.94	35.95	35.95	34.94	34.92

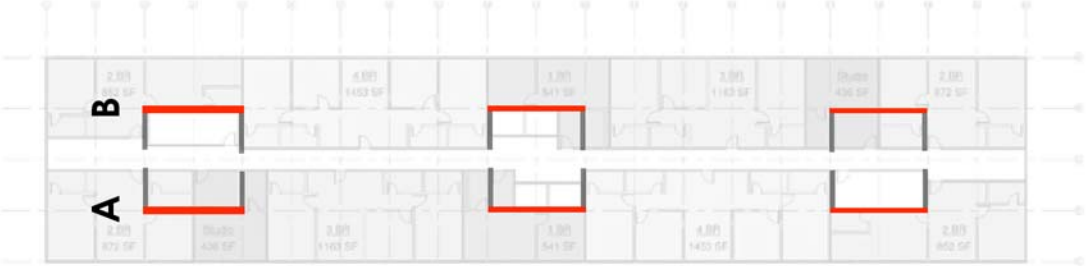


Figure 12: Labels for East-West lateral force resisting system components (highlighted in red)

Table 5: East-West direction shear in story 10 under the calculated seismic loading

East-West shear (ETABs) - Story 10, Lateral Distribution

Method	A	B
ETABS	57.27	60.73
Hand	60.33	60.33

Drift Analysis Results

Drifts were analyzed for each aforementioned load case, and can be viewed in Appendix E (for seismic) and Appendix F (for wind). Table 6 displays the building drifts for the governing load case- Wind Load Case 2 from ASCE 7-05 Chapter 6. Table 7 shows the building drifts under seismic loading in the North-South direction (the direction with the greatest drifts).

Table 6: Wind load case 2 drifts in the North-South direction (governing load case)

CASE 2 LOADING

North-South Drifts

Story	Drift (in.)	Drift Limit (in.) = H/400	Acceptable?	Total Drift (in.)	Drift Limit (in.) = H/400	Acceptable?
Roof	0.00123	0.3	Y	0.03016	6.24	Y
20	0.00127	0.3	Y	0.02893	5.94	Y
19	0.00132	0.3	Y	0.02766	5.64	Y
18	0.00138	0.3	Y	0.02634	5.34	Y
17	0.00144	0.3	Y	0.02496	5.04	Y
16	0.00151	0.3	Y	0.02352	4.74	Y
15	0.00158	0.3	Y	0.02201	4.44	Y
14	0.00164	0.3	Y	0.02043	4.14	Y
13	0.00169	0.3	Y	0.01879	3.84	Y
12	0.00174	0.3	Y	0.01710	3.54	Y
11	0.00177	0.3	Y	0.01537	3.24	Y
10	0.00180	0.3	Y	0.01359	2.94	Y
9	0.00180	0.3	Y	0.01180	2.64	Y
8	0.00179	0.3	Y	0.00999	2.34	Y
7	0.00175	0.3	Y	0.00820	2.04	Y
6	0.00169	0.3	Y	0.00645	1.74	Y
5	0.00158	0.3	Y	0.00476	1.44	Y
4	0.00142	0.3	Y	0.00319	1.14	Y
3	0.00118	0.3	Y	0.00177	0.84	Y
2	0.00059	0.54	Y	0.00059	0.54	Y

Table 7: Building drifts under seismic loading in the North-South direction

SEISMIC LOADING

North-South Drifts

Story	Drift (in.)	Drift Limit (in.) = 0.02h	Acceptable?	Total Drift (in.)	Drift Limit (in.) = 0.02h	Acceptable?
Roof	0.00082	0.2	Y	0.01745	4.16	Y
20	0.00085	0.2	Y	0.01663	3.96	Y
19	0.00088	0.2	Y	0.01578	3.76	Y
18	0.00092	0.2	Y	0.01489	3.56	Y
17	0.00095	0.2	Y	0.01398	3.36	Y
16	0.00098	0.2	Y	0.01302	3.16	Y
15	0.00101	0.2	Y	0.01204	2.96	Y
14	0.00102	0.2	Y	0.01104	2.76	Y
13	0.00103	0.2	Y	0.01001	2.56	Y
12	0.00104	0.2	Y	0.00898	2.36	Y
11	0.00103	0.2	Y	0.00794	2.16	Y
10	0.00101	0.2	Y	0.00691	1.96	Y
9	0.00099	0.2	Y	0.00590	1.76	Y
8	0.00095	0.2	Y	0.00491	1.56	Y
7	0.00090	0.2	Y	0.00396	1.36	Y
6	0.00084	0.2	Y	0.00306	1.16	Y
5	0.00076	0.2	Y	0.00222	0.96	Y
4	0.00066	0.2	Y	0.00146	0.76	Y
3	0.00053	0.2	Y	0.00080	0.56	Y
2	0.00027	0.36	Y	0.00027	0.36	Y

The drifts found from the ETABS analysis of a 3D model of the building with 50% gross-section properties were compared to the deflection criteria established in the Codes, References and Standards section of this document. It can be observed in Tables 6 and 7, as well as the tables in Appendices E and F, that the calculated building drifts are within the acceptable serviceability limits on all stories, for all load cases.

Overturning Analysis on Foundations

The potential for overturning in the Southwest Student Housing building is low due to the high self-weight and relatively low lateral design loads. Table 8 shows a comparison of the moment applied by the wind loads from Case 2 in the North-South direction to the moment applied on the shear walls by the self-weight of the building. The self-weight moment was found by assuming that half of the building weight would apply a load at the edge of the shear wall of the concrete core, as shown in Figure 13.

Table 8: Moment comparisons at the building base

OVERTURNING - Moment check (ft-k)

Case 2 applied moment (N-S direction)	17524.91
Opposing moment from building self-weight	132031.25

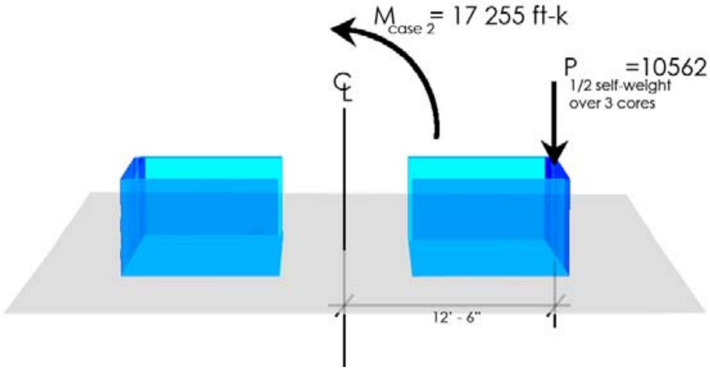


Figure 13: Diagram of loads applied at a section cut of a concrete core.

Spot Check of Concrete Cores

The concrete cores in this building are 8" thick, 25' wide and 25' long on center. The total concrete area resisting moment and axial load is 28800 in^2. Analysis for the concrete cores involved finding the total weight of the building that is applied to the total core area, as well as the maximum moment due to wind and seismic loading. Calculations for shear capacity were also examined. The cores were found to be adequate for the governing lateral loads in the design with regards to both shear and moment. Diagrams and hand calculations for the concrete cores can be found in Appendix H.

Conclusion

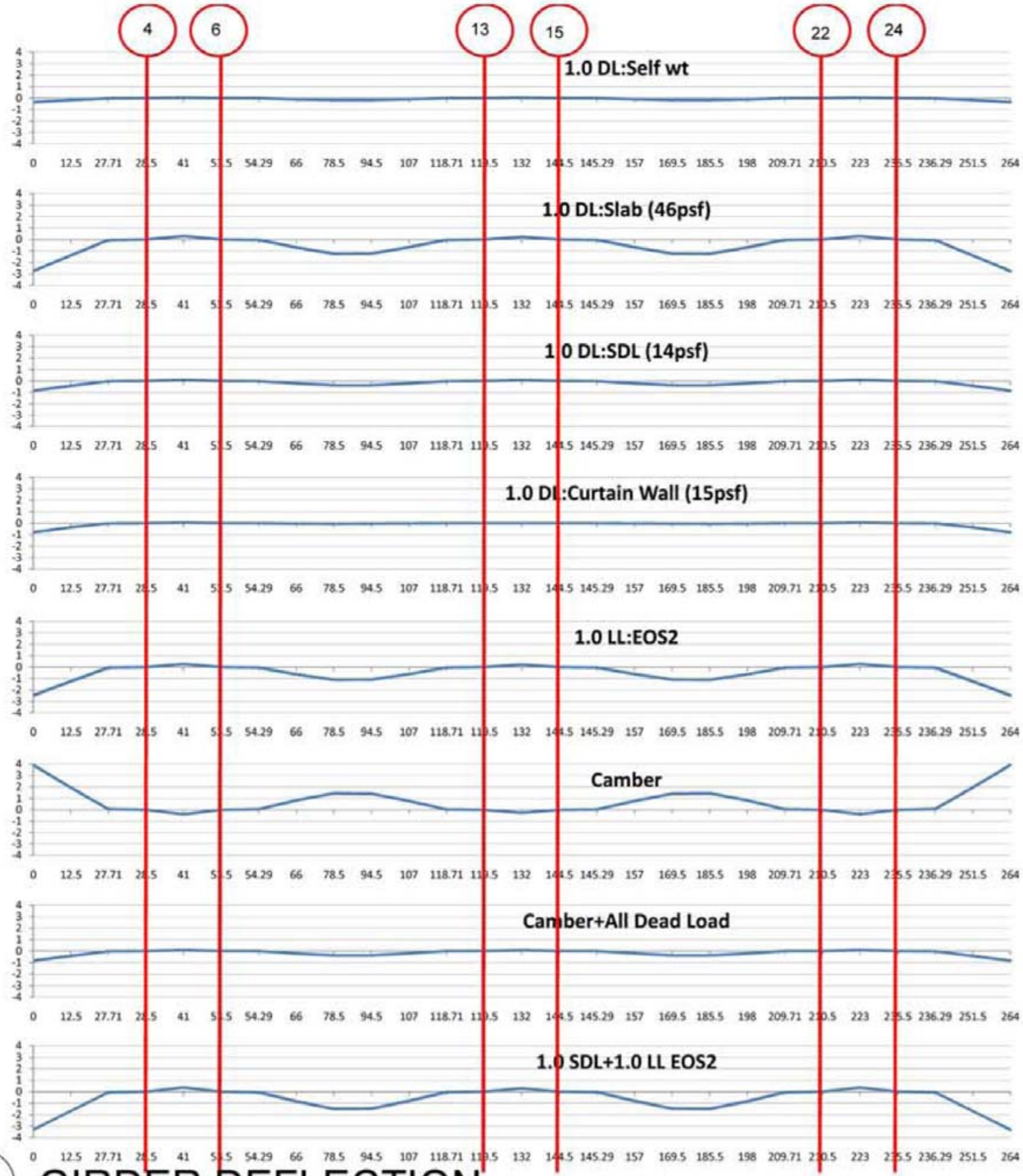
This document, Technical Report #3, has been compiled to specifically discuss the lateral system of the Arizona State University Southwest Student Housing building in Tempe, Arizona. A 3D ETABS model was created to examine lateral load distribution and drift effects under 6 different load cases (2 seismic cases, and 4 wind cases). It was found that, when comparing overall forces and drifts, wind load case 2 from ASCE 7-05 Chapter 6 produced the controlling forces in the building, except at the roof level, where seismic loading in the North-South direction controls. The impact of the lateral load on the foundations was found to be negligible, and completely counteracted by the building self-weight. The ETABS-computed drifts were observed to be well within the allowable service drifts mentioned in previous technical reports.

From the calculations and models run in this report, it is clear that the current lateral system is more than adequate to withstand the design lateral forces for this building.

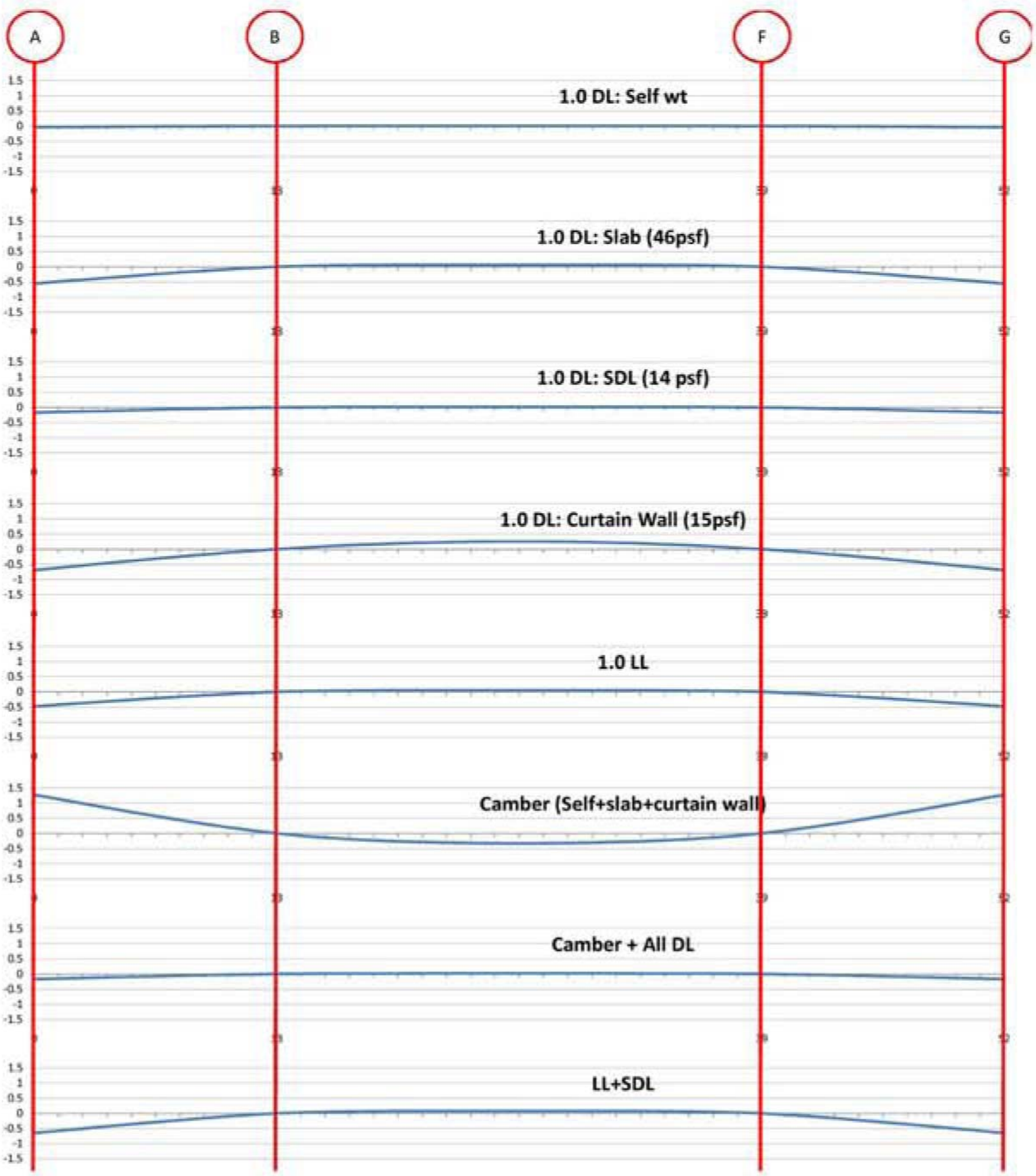
There are several key things to note about the results yielded by the 3D ETABS model: first, the openings in the concrete cores were not modeled in order to simplify the modeling procedure. Additionally, the concrete gross-section properties were taken to be reduced by 50%. Each floor was modeled as a rigid diaphragm. The shear walls were modeled as area elements with a maximum size of 48” square.

Appendices

Appendix A – Building Information Notes



01 GIRDER DEFLECTION



02 BEAM DEFLECTION

APPENDICES - APPENDIX A

TECH 1 | SOUTHWEST STUDENT HOUSING | 2

BUILDING INFORMATION

- MODEL CODE: IBC 2006 AS AMENDED BY THE CITY OF TEMPE, ARIZONA
- DESIGN CODES: AISC "SPEC FOR STRUCTURAL STL BLDGS" AISC 360-05
  ACI "BUILDING CODE REQUIREMENTS FOR STRUCTURAL CONC" ACI 318-05
- STRUCTURAL STANDARDS: $\boxed{\text{ASCE 7-05}}$

I WILL USE ASCE 7-05, AISC 13TH ED, ACI 318-05, IBC 06

- DEFLECTION CRITERIA
  ↳ STRUCTURAL STEEL IS ALL CAMBERED TO DEAL W/ HIGH DEFLECTIONS.

OUT OF CURIOSITY - HOW DOES ONE ASSEMBLE A CAMBERED STRUCTURAL STEEL FLOOR IF THE METHOD IS TO SLIDE BEAMS THROUGH PLASMA-CUT HOLES ON HYDRAULIC ROLLERS?

CHARLIE'S NOTES: $\frac{L}{360}$ FOR LIVE $\frac{L}{240}$ FOR TOTAL $\frac{H}{400}$ FOR DRIFT

- STRUCTURAL OVERVIEW:
  - FOUNDATIONS - SPREAD FOOTING ACCORDING TO CHARLIE ←

ACCORDING TO SOILS REPORT RECOMMENDATIONS -
BLDG WILL EXERT SIGNIFICANT LOADS TO FOUNDATION ELEMENTS.
↳ EMPLOY DEEP FOUNDATION SYSTEM TO LIMIT SETTLEMENTS

ACCORDING TO TTG PACKAGE: (5) 350 YD3 CORE MATS, PERIMETER GRADE BEAMS, SLAB ON GRADE ON TOP

- DRILLED PIERS + MAT

AXIAL CAPACITY = SKIN FRICTION OF SHAFT + END BEARING AT TIP

SOILS:	DEPTH	FRICTION	END BEARING
I SILTY SAND + POORLY GRADED SAND	10-35'	0.4 KSF	—
II SAND GRAVEL COBBLE	35-100'	2.5 KSF	30 KSF

PIER SHAFTS SHOULD PENETRATE AT LEAST 2.5 × PIER ϕ INTO II LAYER & NO PIER ϕ < 12"

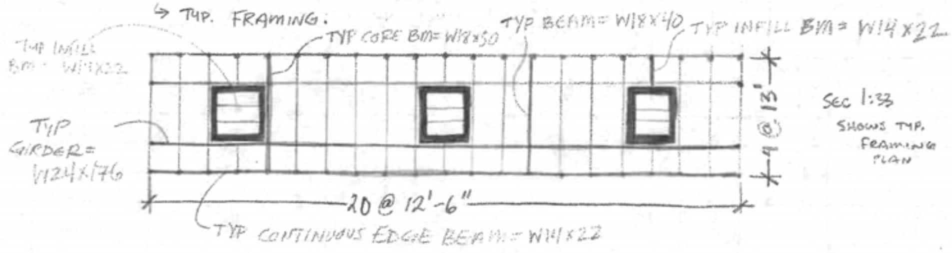
MIN CLEAR SPACING = 3 × BIGGEST ADJACENT PIER ϕ

PREDICTION: FOR ISOLATED PIER, ϕ < 60", SETTLEMENT ≤ 1"

CHARLIE'S NOTES: BLDG DOESN'T HAVE LOCAL? SETTLING B/C SPANS ARE LONGER → NO DIFFERENTIAL SETTLING

- FLOOR SYSTEM - TYP. FOR ALL FLOORS - 3.25" LIGHTWEIGHT CONCRETE ON 5" DECK
  FOR HAND CALCS, CHARLIE ASSUMED 4.5" LIGHTWEIGHT CONCRETE
  ~~WHEELING CORRIGATED 3.0 SB LW.C~~
  (P 14 IN PDF) 20 GAGE
  GO W/ VULCRAFT

SUPPORTED BY STRUCTURAL STEEL FRAME
↳ TYP. FRAMING:



- FRAMING SYSTEM - SEE ABOVE FOR FLOOR FRAMING SYSTEM
  SEE BELOW FOR GRAVITY FRAMING SYSTEM
- LATERAL SYSTEM - (3) CONCRETE CORES: 8" THICK, 25' × 25' ON CENTER (KIND OF)
  SPACED 62'-6" APART (STARTING AT CENTER)

Appendix B – Gravity Load Calculations

CALCULATED LOADS

★GRAVITY

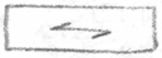
- CONSTRUCTION DEAD LOAD

  - DECK - 3.0 VLI (VULCRAFT COMPOSITE DECK) 20 GAGE
    3.25" LIGHTWEIGHT CONCRETE
    DECK WEIGHT = 2.14 PSF
    CONCRETE WEIGHT = 46 PSF
    TOTAL = 48.14 PSF

  - STRUCTURAL STEEL - ASSUME ≈ ~~8 PSF~~
    CHECK CURRENT SIZES:

SIZE	LENGTH	#	WT (K)
W18X50	52	6	15.6 K
W18X40	52	12	25 K
W14X22	13	6	1.7 K
W24X176	262.5	2	42.4 K
W14X22	262.5	2	11.6 K
			146.3 K

ADDITIONAL NOTES FOR LATER
MAX UNSHORED SPAN
1 SPAN = 10'-6"
2 SPAN = 12'-10"
3 SPAN = 13'-3"
MAX SPAN ON PLAN? 12'-6"
(ASSUME BELOW DECK ORIENTATION)



MAX SUPERIMPOSED LIVE LOAD ON MAX SPAN? 73 PSF
← VERIFY THAT THIS IS LL ONLY
(P55 OF NUCOR DECK MAN.)

TYP FLOOR DIMENSIONS:
250' × 52' = 13 000 SF

APPROX. WEIGHT OF STRUCTURAL STL:

$$\frac{146.3 \times 10^3}{1300} = 11.25 \text{ PSF}$$

STRUCTURAL STEEL - ASSUME $\boxed{11 \text{ PSF}}$ ✓

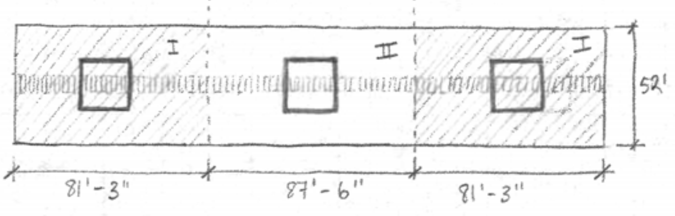
→TOTAL CONSTRUCTION DEAD LOAD = 48.14 + 11 = 59.14 → 59 PSF

- SUPERIMPOSED DEAD LOAD
  →ASSUME SDL OF 15 PSF ← ASSUMPTION BY ENGINEERS. SHOULD HAVE CONFIRMED - PARTITIONS NOT INCLUDED.

- LIVE LOAD
  - RESIDENTIAL = 40 PSF
  - PARKING = 40 PSF
  - CORRIDORS = $\boxed{80}$ ON FLOORS ABOVE GROUND (PSF)
    = 100 ON GROUND FLOOR (PSF)

(THERE IS A 6' WIDE CORRIDOR RUNNING THRU THE CENTER OF THE BUILDING IN THE LONG DIRECTION)
*ENGINEERS JUST TOOK LL TO BE 40 PSF

LIVE LOAD REDUCTION:



TECH 1

APPENDICES-APPENDIX B
SOUTHWEST STUDENT HOUSING

26

CALCULATED LOADS (CONTINUED)

REDUCTION FACTORS:

SECTION	AREA/FLOOR	# FLOORS	FACTOR
I	~4390 SF	1	0.5
		>1	@2, 0.41 @>2, 0.4
II	4725 SF	1	0.5
		>1	0.4

} WILL JUST ROUND TO 0.4

$$L = L_0 \underbrace{\left(0.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right)}_{\substack{\geq 0.5 \ (1 \text{ FLR}) \\ \geq 0.4 \ (>1 \text{ FLR})}} \qquad K_{LL} = 1$$

SAMPLE CALCULATIONS

SECTION I, 1 FLOOR:

$$0.25 + \frac{15}{\sqrt{1 \times 4390}} = 0.476 \rightarrow 0.5$$

>1 FLOOR:

$$2? \quad 0.25 + \frac{15}{\sqrt{1 \times 4390 \times 2}} = 0.41$$

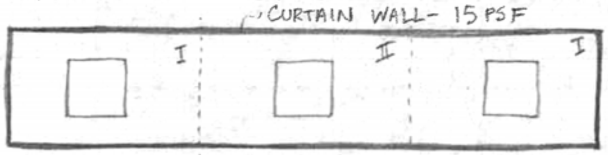
$$3? \quad 0.25 + \frac{15}{\sqrt{1 + 4390 \times 3}} = 0.38 \rightarrow 0.4$$

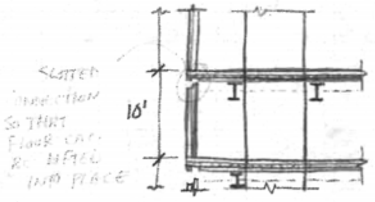
$\boxed{80 \text{ PSF}} \times 0.5 = \underline{\underline{40 \text{ PSF}}}$ (FOR CORE CALCULATIONS)

— MECHANICAL EQUIPMENT

ON GROUND FLOOR, W/ 2-PIPE SYSTEM FOR HEATING/COOLING ROOMS
CHILLER, PUMPS, ETC. ON GROUND FLOOR

— WALL LOADS





EACH FLOOR SUPPORTS ITS RESPECTIVE "CURTAIN WALL"
∴ FLOORS 2-19 HAVE 10' PERIMETER CURTAIN WALLS @ 15 PSF

— SNOW LOADS

$P_f = 0.7\, C_e C_t I p_g$ >0
(FLAT ROOF)

$P_f = 0$

↙ FIGURE 7-1 (ASCE7-05)

$P_g =$ (5000) 10
(4600) 5
(3500) ZERO

TEMPE, AZ IS @ AN ELEVATION OF... 1140-1495 FEET

$1495 < 3500 \rightarrow P_g = 0$

4

Appendix C – Lateral Load Calculations: Wind

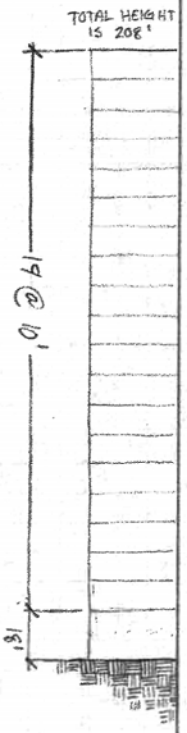
CALCULATED LOADS (CONTINUED)

*LATERAL

-WIND LOADS

ASSUME ENCLOSED BUILDING

FIGURE 6-1: 90 MPH NOMINAL 3-SEC GUST WIND SPEED
TABLE 6-1: IMPORTANCE CATEGORY II, FACTOR = 1.0
EXPOSURE CATEGORY B
(URBAN & SUBURBAN AREAS w/ NUMEROUS CLOSELY SPACED OBSTRUCTION ≥ SINGLE FAMILY DWELLINGS FOR 20 X BUILDING HEIGHT = 4160 FT)
⇒ JUSTIFIED BY SITE LOCATION IN CENTRAL TEMPE, AZ
(SHOW PHOTO OF SITE?)



$GC_{pi} = \pm 0.18$ (ENCLOSED)

$L/B = \frac{(12.5' \times 20)}{52'} = 4.81 > 4$ (E-W) $= \frac{52}{250} = 0.208$ (N-S)

C_P VALUES	E-W	N-S
WW WALL	0.8	0.8
LW WALL	-0.2	-0.5
SIDE WALL	-0.7	-0.7

ASSUME BUILDING IS RIGID (TO BE VERIFIED IN LATER REPORT)

TAKE G = 0.85

$$q_z = 0.00256 \underbrace{(K_z)}_{} \underbrace{(K_{zt})}_{1.0} \underbrace{(K_d)}_{0.85} V^2 \underbrace{(I)}_{1.0} \text{ (PSF)}$$

$15' \leq z \leq 1200'$

$K_z = 2.01\left(\frac{z}{1200}\right)^{2/7}$

0.85 FOR MWFRS & COMPONENTS/CLADDING

$$q_z = 0.00256\left[2.01\left(\frac{z}{1200}\right)^{2/7}\right](0.85)(90)^2(1.0)(1.0) \text{ PSF}$$

FOR A FLOOR HEIGHT z OFF THE GROUND

MWFRS: $P = qGC_p \boxed{\cancel{-q_i(GC_{pi})}}$ (PSF)

$$P = \left[0.00256\left\{2.01\left(\frac{z}{1200}\right)^{2/7}\right\}0.85(90^2)\right](0.85)\,C_P$$

VARIES DEPENDING ON WALL & DIRECTION!

SO, 2ND FLOOR, WINDWARD WALL, EAST : WEST DIRECTION:

$z = \frac{18}{2} = 9'$

$$P_2 = \left[0.00256\left\{2.01\left(\frac{9}{1200}\right)^{2/7}\right\}0.85(90^2)\right]0.85\,(0.8) \cong 6 \text{ PSF}$$

(SEE EXCEL TABLE FOR ALL CALCULATED VALUES, AS WELL AS TOTAL PRESSURE)

TOTAL PRESSURE = |WW| + |LW|

Appendix D – Lateral Load Calculations: Seismic

TECH 1 (REVISION 10.11.2011)

APPENDICES – APPENDIX D
SOUTHWEST STUDENT HOUSING

CALCULATED LOADS (CONTINUED)

– SEISMIC LOADS

FRAME SYSTEM: REINFORCED CONCRETE SHEAR WALLS

$R = 4.0$
$I = 1.0$

$C_S = \frac{S_{DS}}{(R/I)} \geq 0.01$ $\quad C_S \leq \frac{S_{D1}}{C_u T_a (R/I)}$ FOR $T \leq T_L$

$T_a = \underset{0.02}{C_T}\, \underset{208}{h_n}^{\overset{0.75}{x}} \approx 1.1$ SEC

$S_S = 16\%g$
$S_1 = 8\%g$
$T_L = 6$ SEC

} FIGURES FROM ASCE 7-05, CH 22

$F_x = C_{vx} V \qquad C_{vx} = \frac{w_x h_x^{\overset{2}{k}}}{\sum_{i=1}^{n} w_i h_i^k}$

SITE CLASS C (FROM GEOTECH REPORT)

$F_a = 1.2 \quad F_v = 1.7$

$S_{MS} = F_a S_S = 1.2(0.16) = 0.192 \qquad S_{DS} = \frac{2}{3} S_{MS} = 0.128$

$S_{M1} = F_v S_1 = 1.7(0.08) = 0.136 \qquad S_{D1} = \frac{2}{3} S_{M1} = 0.0907 < 0.1 \Downarrow C_u = 1.7$

$T_0 = 0.2 \frac{S_{D1}}{S_{DS}} = 0.2 \frac{0.0907}{0.128} = 0.142$

$T_S = \frac{S_{D1}}{S_{DS}} = \frac{0.0907}{0.128} = 0.708$

$C_S = \frac{0.128}{4/1.0} = 0.032 \leq \frac{0.0907}{1.7(1.1)(4/1.0)} = 0.0121 \Rightarrow C_S = 0.0121$

$W_{TOT} = \sum_{i=1}^{20} w_i$

$W_i = (C_{DL} + S_{DL}) \overset{250 \times 52}{A_{FLOOR}} +$ CURTAIN WALL (PERIMETER) $= (59+15)13000 + 15(604) = 971.1^k$
↳ PER FLOOR

$V = 0.0121\,(971.1 \times 20) = 235.5$ KIPS (SEE EXCEL TABLE FOR VERTICAL DISTRIBUTION)

N-S -Direction Seismic Loads

T=	2.708	s
k=	2.000	
V_b=	235.5	kips

Story	Height h_x (ft)	Weight w_x (k)	$w_x h_x^k$	C_{vx}	Lateral Force F_x (k)	Story Shear V_x (k)	Width, X-dir (ft)	5% width (ft)	A_x	Moment M_z (ft-k)
Roof	208	962	41619968	0.124	**29**	0	250	13	1	**366**
20	198	1052.6	41266130	0.123	**29**	29	250	13	1	**363**
19	188	1052.6	37203094	0.111	**26**	58	250	13	1	**327**
18	178	1052.6	33350578	0.100	**23**	84	250	13	1	**293**
17	168	1052.6	29708582	0.089	**21**	108	250	13	1	**261**
16	158	1052.6	26277106	0.078	**18**	129	250	13	1	**231**
15	148	1052.6	23056150	0.069	**16**	147	250	13	1	**203**
14	138	1052.6	20045714	0.060	**14**	163	250	13	1	**176**
13	128	1052.6	17245798	0.051	**12**	178	250	13	1	**152**
12	118	1052.6	14656402	0.044	**10**	190	250	13	1	**129**
11	108	1052.6	12277526	0.037	**9**	200	250	13	1	**108**
10	98	1052.6	10109170	0.030	**7**	209	250	13	1	**89**
9	88	1052.6	8151334	0.024	**6**	216	250	13	1	**72**
8	78	1052.6	6404018	0.019	**5**	221	250	13	1	**56**
7	68	1052.6	4867222	0.015	**3**	226	250	13	1	**43**
6	58	1052.6	3540946	0.011	**2**	229	250	13	1	**31**
5	48	1052.6	2425190	0.007	**2**	232	250	13	1	**21**
4	38	1052.6	1519954	0.005	**1**	234	250	13	1	**13**
3	28	1052.6	825238	0.002	**1**	235	250	13	1	**7**
2	18	1215.68	393880	0.001	**0**	235	250	13	1	**3**
	S	**21124.48**	**334944008**	**1.000**	**236**	**236**				**2944**

N-S shear in each lateral force resisting component (k)

Distances from center (ft)

A		B		C	
1	2	1	2	1	2
100	87.5	12.5	12.5	87.5	100

Story	A		B		C	
	1	2	1	2	1	2
Roof	0.61	0.70	4.88	4.88	0.70	0.61
20	5.48	5.57	9.71	9.71	5.57	5.48
19	10.26	10.34	14.07	14.07	10.34	10.26
18	14.56	14.63	17.98	17.98	14.63	14.56
17	18.42	18.48	21.46	21.46	18.48	18.42
16	21.85	21.90	24.54	24.54	21.90	21.85
15	24.88	24.93	27.24	27.24	24.93	24.88
14	27.54	27.58	29.59	29.59	27.58	27.54
13	29.84	29.88	31.61	31.61	29.88	29.84
12	31.83	31.86	33.33	33.33	31.86	31.83
11	33.51	33.54	34.77	34.77	33.54	33.51
10	34.92	34.94	35.95	35.95	34.94	34.92
9	36.07	36.09	36.91	36.91	36.09	36.07
8	37.00	37.02	37.66	37.66	37.02	37.00
7	37.73	37.74	38.23	38.23	37.74	37.73
6	38.28	38.29	38.64	38.64	38.29	38.28
5	38.68	38.69	38.93	38.93	38.69	38.68
4	38.95	38.95	39.11	39.11	38.95	38.95
3	39.12	39.12	39.20	39.20	39.12	39.12
2	39.21	39.21	39.25	39.25	39.21	39.21

E-W -Direction Seismic Loads

T=	2.283	s
k=	1.892	
V_b=	235.5	kips

Story	Height h_x (ft)	Weight w_x (k)	$w_x h_x^k$	C_{vx}	Lateral Force F_x (k)	Story Shear V_x (k)	Width, X-dir (ft)	5% width (ft)	A_x	Moment M_z (ft-k)
Roof	208	962	23323401	0.070	**16**	0	52	3	1	**43**
20	198	1052.6	23249069	0.069	**16**	16	52	3	1	**43**
19	188	1052.6	21078172	0.063	**15**	33	52	3	1	**39**
18	178	1052.6	19007841	0.057	**13**	48	52	3	1	**35**
17	168	1052.6	17038677	0.051	**12**	61	52	3	1	**31**
16	158	1052.6	15171316	0.045	**11**	73	52	3	1	**28**
15	148	1052.6	13406438	0.040	**9**	84	52	3	1	**25**
14	138	1052.6	11744774	0.035	**8**	93	52	3	1	**21**
13	128	1052.6	10187111	0.030	**7**	101	52	3	1	**19**
12	118	1052.6	8734302	0.026	**6**	108	52	3	1	**16**
11	108	1052.6	7387277	0.022	**5**	115	52	3	1	**14**
10	98	1052.6	6147061	0.018	**4**	120	52	3	1	**11**
9	88	1052.6	5014785	0.015	**4**	124	52	3	1	**9**
8	78	1052.6	3991722	0.012	**3**	128	52	3	1	**7**
7	68	1052.6	3079314	0.009	**2**	130	52	3	1	**6**
6	58	1052.6	2279226	0.007	**2**	133	52	3	1	**4**
5	48	1052.6	1593423	0.005	**1**	134	52	3	1	**3**
4	38	1052.6	1024293	0.003	**1**	135	52	3	1	**2**
3	28	1052.6	574861	0.002	**0**	136	52	3	1	**1**
2	18	1215.68	287851	0.001	**0**	136	52	3	1	**1**
	S	**21124.48**	**194320912**	**0.580**	**137**	**137**				**355**

E-W shear in each lateral force resisting component (k)

Distances from center (ft)

A	B
12.5	12.5

Story	A	B
Roof	1.71	1.71
20	9.90	9.90
19	17.91	17.91
18	25.17	25.17
17	31.71	31.71
16	37.56	37.56
15	42.77	42.77
14	47.36	47.36
13	51.38	51.38
12	54.85	54.85
11	57.82	57.82
10	60.33	60.33
9	62.41	62.41
8	64.10	64.10
7	65.43	65.43
6	66.46	66.46
5	67.21	67.21
4	67.73	67.73
3	68.05	68.05
2	68.23	68.23

Appendix E – Modeled Seismic Loads

N-S Seismic Shear Comparison

Story	Calculated	ETABS Output
Roof	12.37	29.00
20	41.53	58.00
19	69.33	84.00
18	94.34	107.00
17	116.71	128.00
16	136.58	146.00
15	154.10	162.00
14	169.41	176.00
13	182.68	188.00
12	194.03	198.00
11	203.63	207.00
10	211.62	214.00
9	218.15	220.00
8	223.36	225.00
7	227.40	228.00
6	230.43	230.00
5	232.59	232.00
4	234.03	233.00
3	234.89	234.00
2	235.34	234.00

E-W Seismic Shear Comparison

Story	Calculated	ETABS Output
Roof	3.41	16.00
20	19.80	32.00
19	35.83	47.00
18	50.35	60.00
17	63.42	72.00
16	75.13	83.00
15	85.54	92.00
14	94.72	100.00
13	102.75	107.00
12	109.70	113.00
11	115.64	118.00
10	120.66	122.00
9	124.81	126.00
8	128.19	129.00
7	130.86	131.00
6	132.91	133.00
5	134.41	134.00
4	135.45	135.00
3	136.11	135.00
2	136.47	135.00

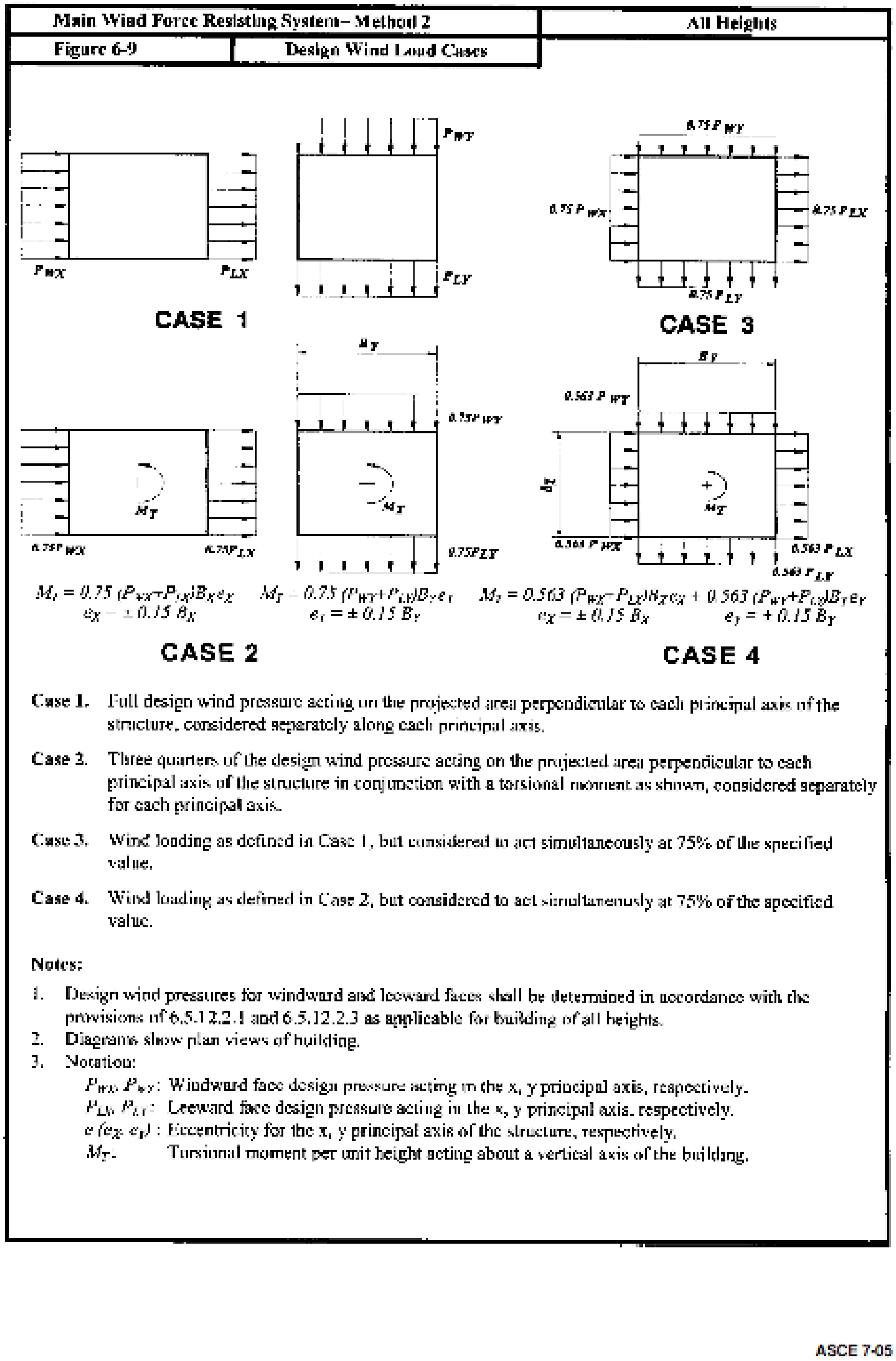
East-West Drifts

Story	Drift (in.)	Drift Limit (in.) = 0.02h	Acceptable?	Total Drift (in.)	Drift Limit (in.) = 0.02h	Acceptable?
Roof	0.00042	0.2	Y	0.00648	4.16	Y
20	0.00042	0.2	Y	0.00606	3.96	Y
19	0.00042	0.2	Y	0.00564	3.76	Y
18	0.00042	0.2	Y	0.00521	3.56	Y
17	0.00042	0.2	Y	0.00479	3.36	Y
16	0.00042	0.2	Y	0.00437	3.16	Y
15	0.00041	0.2	Y	0.00395	2.96	Y
14	0.00040	0.2	Y	0.00354	2.76	Y
13	0.00039	0.2	Y	0.00314	2.56	Y
12	0.00038	0.2	Y	0.00275	2.36	Y
11	0.00036	0.2	Y	0.00237	2.16	Y
10	0.00034	0.2	Y	0.00201	1.96	Y
9	0.00032	0.2	Y	0.00166	1.76	Y
8	0.00030	0.2	Y	0.00134	1.56	Y
7	0.00027	0.2	Y	0.00105	1.36	Y
6	0.00024	0.2	Y	0.00078	1.16	Y
5	0.00020	0.2	Y	0.00054	0.96	Y
4	0.00016	0.2	Y	0.00034	0.76	Y
3	0.00012	0.2	Y	0.00018	0.56	Y
2	0.00006	0.36	Y	0.00006	0.36	Y

North-South Drifts

Story	Drift (in.)	Drift Limit (in.) = 0.02h	Acceptable?	Total Drift (in.)	Drift Limit (in.) = 0.02h	Acceptable?
Roof	0.00082	0.2	Y	0.01745	4.16	Y
20	0.00085	0.2	Y	0.01663	3.96	Y
19	0.00088	0.2	Y	0.01578	3.76	Y
18	0.00092	0.2	Y	0.01489	3.56	Y
17	0.00095	0.2	Y	0.01398	3.36	Y
16	0.00098	0.2	Y	0.01302	3.16	Y
15	0.00101	0.2	Y	0.01204	2.96	Y
14	0.00102	0.2	Y	0.01104	2.76	Y
13	0.00103	0.2	Y	0.01001	2.56	Y
12	0.00104	0.2	Y	0.00898	2.36	Y
11	0.00103	0.2	Y	0.00794	2.16	Y
10	0.00101	0.2	Y	0.00691	1.96	Y
9	0.00099	0.2	Y	0.00590	1.76	Y
8	0.00095	0.2	Y	0.00491	1.56	Y
7	0.00090	0.2	Y	0.00396	1.36	Y
6	0.00084	0.2	Y	0.00306	1.16	Y
5	0.00076	0.2	Y	0.00222	0.96	Y
4	0.00066	0.2	Y	0.00146	0.76	Y
3	0.00053	0.2	Y	0.00080	0.56	Y
2	0.00027	0.36	Y	0.00027	0.36	Y

Appendix F – Modeled Wind Loads



Main Wind Force Resisting System – Method 2		**All Heights**
Figure 6-9	**Design Wind Load Cases**	

$M_T = 0.75\ (P_{WX}+P_{LX})B_X e_X$
$e_X = \pm\ 0.15\ B_X$

$M_T = 0.75\ (P_{WY}+P_{LY})B_Y e_Y$
$e_Y = \pm\ 0.15\ B_Y$

CASE 2

$M_T = 0.563\ (P_{WX}+P_{LX})B_X e_X + 0.563\ (P_{WY}+P_{LY})B_Y e_Y$
$e_X = \pm\ 0.15\ B_X \qquad e_Y = \pm\ 0.15\ B_Y$

CASE 4

Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered seperately along each principal axis.

Case 2. Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes:

1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of 6.5.12.2.1 and 6.5.12.2.3 as applicable for building of all heights.
2. Diagrams show plan views of building.
3. Notation:

P_{WX}, P_{WY}: Windward face design pressure acting in the x, y principal axis, respectively.
P_{LX}, P_{LY}: Leeward face design pressure acting in the x, y principal axis, respectively.
$e\ (e_X, e_Y)$: Eccentricity for the x, y principal axis of the structure, respectively.
M_T: Torsional moment per unit height acting about a vertical axis of the building.

CASE 1 LOADING

East-West Wind Loads

Story	Force (k)	Moment (ft-k)	Moment (in-k)
Roof	6.17	0.00	0.00
20	7.49	0.00	0.00
19	7.38	0.00	0.00
18	7.26	0.00	0.00
17	7.14	0.00	0.00
16	7.02	0.00	0.00
15	6.89	0.00	0.00
14	6.75	0.00	0.00
13	6.61	0.00	0.00
12	6.46	0.00	0.00
11	6.30	0.00	0.00
10	6.12	0.00	0.00
9	5.94	0.00	0.00
8	5.74	0.00	0.00
7	5.52	0.00	0.00
6	5.27	0.00	0.00
5	4.99	0.00	0.00
4	4.67	0.00	0.00
3	4.28	0.00	0.00
2	1.89	0.00	0.00

North-South Wind Loads

Story	Force (k)	Moment (ft-k)	Moment (in-k)
Roof	18.25	0.00	0.00
20	35.99	0.00	0.00
19	35.46	0.00	0.00
18	34.91	0.00	0.00
17	34.34	0.00	0.00
16	33.75	0.00	0.00
15	33.12	0.00	0.00
14	32.47	0.00	0.00
13	31.77	0.00	0.00
12	31.04	0.00	0.00
11	30.27	0.00	0.00
10	29.44	0.00	0.00
9	28.55	0.00	0.00
8	27.58	0.00	0.00
7	26.52	0.00	0.00
6	25.34	0.00	0.00
5	24.01	0.00	0.00
4	22.46	0.00	0.00
3	20.58	0.00	0.00
2	9.07	0.00	0.00

CASE 1 LOADING (CONT'D)

East-West Drifts

Story	Drift (in.)	Drift Limit (in.) = H/400	Acceptable?	Total Drift (in.)	Drift Limit (in.) = H/400	Acceptable?
Roof	0.00026	0.3	Y	0.00402	6.24	Y
20	0.00026	0.3	Y	0.00377	5.94	Y
19	0.00026	0.3	Y	0.00351	5.64	Y
18	0.00026	0.3	Y	0.00326	5.34	Y
17	0.00025	0.3	Y	0.00300	5.04	Y
16	0.00025	0.3	Y	0.00275	4.74	Y
15	0.00025	0.3	Y	0.00250	4.44	Y
14	0.00025	0.3	Y	0.00225	4.14	Y
13	0.00024	0.3	Y	0.00200	3.84	Y
12	0.00023	0.3	Y	0.00176	3.54	Y
11	0.00023	0.3	Y	0.00153	3.24	Y
10	0.00022	0.3	Y	0.00130	2.94	Y
9	0.00020	0.3	Y	0.00109	2.64	Y
8	0.00019	0.3	Y	0.00089	2.34	Y
7	0.00017	0.3	Y	0.00070	2.04	Y
6	0.00015	0.3	Y	0.00052	1.74	Y
5	0.00013	0.3	Y	0.00037	1.44	Y
4	0.00011	0.3	Y	0.00024	1.14	Y
3	0.00008	0.3	Y	0.00013	0.84	Y
2	0.00004	0.54	Y	0.00004	0.54	Y

North-South Drifts

Story	Drift (in.)	Drift Limit (in.) = H/400	Acceptable?	Total Drift (in.)	Drift Limit (in.) = H/400	Acceptable?
Roof	0.00101	0.3	Y	0.02421	6.24	Y
20	0.00104	0.3	Y	0.02320	5.94	Y
19	0.00109	0.3	Y	0.02215	5.64	Y
18	0.00113	0.3	Y	0.02107	5.34	Y
17	0.00118	0.3	Y	0.01994	5.04	Y
16	0.00123	0.3	Y	0.01875	4.74	Y
15	0.00128	0.3	Y	0.01752	4.44	Y
14	0.00133	0.3	Y	0.01624	4.14	Y
13	0.00137	0.3	Y	0.01491	3.84	Y
12	0.00140	0.3	Y	0.01355	3.54	Y
11	0.00142	0.3	Y	0.01215	3.24	Y
10	0.00144	0.3	Y	0.01073	2.94	Y
9	0.00144	0.3	Y	0.00929	2.64	Y
8	0.00142	0.3	Y	0.00786	2.34	Y
7	0.00138	0.3	Y	0.00644	2.04	Y
6	0.00132	0.3	Y	0.00505	1.74	Y
5	0.00123	0.3	Y	0.00373	1.44	Y
4	0.00110	0.3	Y	0.00250	1.14	Y
3	0.00091	0.3	Y	0.00140	0.84	Y
2	0.00049	0.54	Y	0.00049	0.54	Y

CASE 2 LOADING

East-West Wind Loads

Story	Force (k)	Moment (ft-k)	Moment (in-k)
Roof	4.63	173.50	2082
20	5.61	210.56	2527
19	5.53	207.46	2490
18	5.45	204.25	2451
17	5.36	200.90	2411
16	5.26	197.41	2369
15	5.17	193.76	2325
14	5.06	189.92	2279
13	4.96	185.88	2231
12	4.84	181.61	2179
11	4.72	177.07	2125
10	4.59	172.23	2067
9	4.45	167.01	2004
8	4.30	161.35	1936
7	4.14	155.15	1862
6	3.95	148.26	1779
5	3.75	140.45	1685
4	3.50	131.38	1577
3	3.21	120.41	1445
2	1.42	53.06	637

North-South Wind Loads

Story	Force (k)	Moment (ft-k)	Moment (in-k)
Roof	13.69	513.32	6160
20	26.99	1012.29	12147
19	26.60	997.41	11969
18	26.19	981.96	11783
17	25.76	513.32	6160
16	25.31	1012.29	12147
15	24.84	997.41	11969
14	24.35	981.96	11783
13	23.83	513.32	6160
12	23.28	1012.29	12147
11	22.70	997.41	11969
10	22.08	981.96	11783
9	21.41	513.32	6160
8	20.69	1012.29	12147
7	19.89	997.41	11969
6	19.01	981.96	11783
5	18.01	513.32	6160
4	16.84	1012.29	12147
3	15.44	997.41	11969
2	6.80	981.96	11783

CASE 2 LOADING (CONT'D)

East-West Drifts

Story	Drift (in.)	Drift Limit (in.) = H/400	Acceptable?	Total Drift (in.)	Drift Limit (in.) = H/400	Acceptable?
Roof	0.00024	0.3	Y	0.00445	6.24	Y
20	0.00024	0.3	Y	0.00422	5.94	Y
19	0.00024	0.3	Y	0.00398	5.64	Y
18	0.00025	0.3	Y	0.00373	5.34	Y
17	0.00025	0.3	Y	0.00349	5.04	Y
16	0.00025	0.3	Y	0.00323	4.74	Y
15	0.00026	0.3	Y	0.00298	4.44	Y
14	0.00026	0.3	Y	0.00272	4.14	Y
13	0.00026	0.3	Y	0.00247	3.84	Y
12	0.00026	0.3	Y	0.00221	3.54	Y
11	0.00025	0.3	Y	0.00195	3.24	Y
10	0.00025	0.3	Y	0.00170	2.94	Y
9	0.00024	0.3	Y	0.00145	2.64	Y
8	0.00023	0.3	Y	0.00121	2.34	Y
7	0.00022	0.3	Y	0.00097	2.04	Y
6	0.00021	0.3	Y	0.00075	1.74	Y
5	0.00019	0.3	Y	0.00054	1.44	Y
4	0.00016	0.3	Y	0.00035	1.14	Y
3	0.00013	0.3	Y	0.00019	0.84	Y
2	0.00006	0.54	Y	0.00006	0.54	Y

North-South Drifts

Story	Drift (in.)	Drift Limit (in.) = H/400	Acceptable?	Total Drift (in.)	Drift Limit (in.) = H/400	Acceptable?
Roof	0.00123	0.3	Y	0.03016	6.24	Y
20	0.00127	0.3	Y	0.02893	5.94	Y
19	0.00132	0.3	Y	0.02766	5.64	Y
18	0.00138	0.3	Y	0.02634	5.34	Y
17	0.00144	0.3	Y	0.02496	5.04	Y
16	0.00151	0.3	Y	0.02352	4.74	Y
15	0.00158	0.3	Y	0.02201	4.44	Y
14	0.00164	0.3	Y	0.02043	4.14	Y
13	0.00169	0.3	Y	0.01879	3.84	Y
12	0.00174	0.3	Y	0.01710	3.54	Y
11	0.00177	0.3	Y	0.01537	3.24	Y
10	0.00180	0.3	Y	0.01359	2.94	Y
9	0.00180	0.3	Y	0.01180	2.64	Y
8	0.00179	0.3	Y	0.00999	2.34	Y
7	0.00175	0.3	Y	0.00820	2.04	Y
6	0.00169	0.3	Y	0.00645	1.74	Y
5	0.00158	0.3	Y	0.00476	1.44	Y
4	0.00142	0.3	Y	0.00319	1.14	Y
3	0.00118	0.3	Y	0.00177	0.84	Y
2	0.00059	0.54	Y	0.00059	0.54	Y

CASE 3 LOADING

East-West Wind Loads

Story	Force (k)	Moment (ft-k)	Moment (in-k)
Roof	4.63	0.00	0.00
20	5.61	0.00	0.00
19	5.53	0.00	0.00
18	5.45	0.00	0.00
17	5.36	0.00	0.00
16	5.26	0.00	0.00
15	5.17	0.00	0.00
14	5.06	0.00	0.00
13	4.96	0.00	0.00
12	4.84	0.00	0.00
11	4.72	0.00	0.00
10	4.59	0.00	0.00
9	4.45	0.00	0.00
8	4.30	0.00	0.00
7	4.14	0.00	0.00
6	3.95	0.00	0.00
5	3.75	0.00	0.00
4	3.50	0.00	0.00
3	3.21	0.00	0.00
2	1.42	0.00	0.00

North-South Wind Loads

Story	Force (k)	Moment (ft-k)	Moment (in-k)
Roof	13.69	0.00	0.00
20	26.99	0.00	0.00
19	26.60	0.00	0.00
18	26.19	0.00	0.00
17	25.76	0.00	0.00
16	25.31	0.00	0.00
15	24.84	0.00	0.00
14	24.35	0.00	0.00
13	23.83	0.00	0.00
12	23.28	0.00	0.00
11	22.70	0.00	0.00
10	22.08	0.00	0.00
9	21.41	0.00	0.00
8	20.69	0.00	0.00
7	19.89	0.00	0.00
6	19.01	0.00	0.00
5	18.01	0.00	0.00
4	16.84	0.00	0.00
3	15.44	0.00	0.00
2	6.80	0.00	0.00

Building Drifts

Story	Drift (in.)	Drift Limit (in.) = H/400	Acceptable?	Total Drift (in.)	Drift Limit (in.) = H/400	Acceptable?
Roof	0.00078	0.3	Y	0.01843	6.24	Y
20	0.00081	0.3	Y	0.01764	5.94	Y
19	0.00084	0.3	Y	0.01684	5.64	Y
18	0.00087	0.3	Y	0.01600	5.34	Y
17	0.00091	0.3	Y	0.01513	5.04	Y
16	0.00094	0.3	Y	0.01422	4.74	Y
15	0.00098	0.3	Y	0.01328	4.44	Y
14	0.00101	0.3	Y	0.01230	4.14	Y
13	0.00104	0.3	Y	0.01129	3.84	Y
12	0.00106	0.3	Y	0.01025	3.54	Y
11	0.00108	0.3	Y	0.00919	3.24	Y
10	0.00109	0.3	Y	0.00811	2.94	Y
9	0.00109	0.3	Y	0.00702	2.64	Y
8	0.00107	0.3	Y	0.00593	2.34	Y
7	0.00105	0.3	Y	0.00486	2.04	Y
6	0.00100	0.3	Y	0.00381	1.74	Y
5	0.00093	0.3	Y	0.00281	1.44	Y
4	0.00083	0.3	Y	0.00188	1.14	Y
3	0.00068	0.3	Y	0.00105	0.84	Y
2	0.00037	0.54	Y	0.00037	0.54	Y

CASE 4 LOADING

East-West Wind Loads

Story	Force (k)	Moment (ft-k)	Moment (in-k)
Roof	3.47	130.24	1562.91
20	4.21	158.06	1896.69
19	4.15	155.73	1868.82
18	4.09	153.32	1839.86
17	4.02	150.81	1809.71
16	3.95	148.19	1778.26
15	3.88	145.45	1745.35
14	3.80	142.57	1710.81
13	3.72	139.54	1674.43
12	3.64	136.33	1635.96
11	3.54	132.92	1595.09
10	3.45	129.28	1551.42
9	3.34	125.37	1504.44
8	3.23	121.12	1453.47
7	3.11	116.47	1397.59
6	2.97	111.29	1335.50
5	2.81	105.43	1265.21
4	2.63	98.63	1183.52
3	2.41	90.39	1084.63
2	1.06	39.83	478.00

North-South Wind Loads

Story	Force (k)	Moment (ft-k)	Moment (in-k)
Roof	10.28	385.33	4624.00
20	20.26	759.89	9118.72
19	19.97	748.72	8984.69
18	19.66	737.12	8845.47
17	19.33	385.33	4624.00
16	19.00	759.89	9118.72
15	18.65	748.72	8984.69
14	18.28	737.12	8845.47
13	17.89	385.33	4624.00
12	17.48	759.89	9118.72
11	17.04	748.72	8984.69
10	16.57	737.12	8845.47
9	16.07	385.33	4624.00
8	15.53	759.89	9118.72
7	14.93	748.72	8984.69
6	14.27	737.12	8845.47
5	13.52	385.33	4624.00
4	12.64	759.89	9118.72
3	11.59	748.72	8984.69
2	5.11	737.12	8845.47

Building Drifts

Story	Drift (in.)	Drift Limit (in.) = H/400	Acceptable?	Total Drift (in.)	Drift Limit (in.) = H/400	Acceptable?
Roof	0.00102	0.3	Y	0.02495	6.24	Y
20	0.00106	0.3	Y	0.02393	5.94	Y
19	0.00110	0.3	Y	0.02287	5.64	Y
18	0.00115	0.3	Y	0.02177	5.34	Y
17	0.00120	0.3	Y	0.02062	5.04	Y
16	0.00126	0.3	Y	0.01942	4.74	Y
15	0.00131	0.3	Y	0.01817	4.44	Y
14	0.00136	0.3	Y	0.01686	4.14	Y
13	0.00140	0.3	Y	0.01550	3.84	Y
12	0.00144	0.3	Y	0.01410	3.54	Y
11	0.00147	0.3	Y	0.01266	3.24	Y
10	0.00149	0.3	Y	0.01119	2.94	Y
9	0.00149	0.3	Y	0.00971	2.64	Y
8	0.00148	0.3	Y	0.00822	2.34	Y
7	0.00145	0.3	Y	0.00674	2.04	Y
6	0.00139	0.3	Y	0.00530	1.74	Y
5	0.00130	0.3	Y	0.00391	1.44	Y
4	0.00116	0.3	Y	0.00261	1.14	Y
3	0.00097	0.3	Y	0.00145	0.84	Y
2	0.00048	0.54	Y	0.00048	0.54	Y

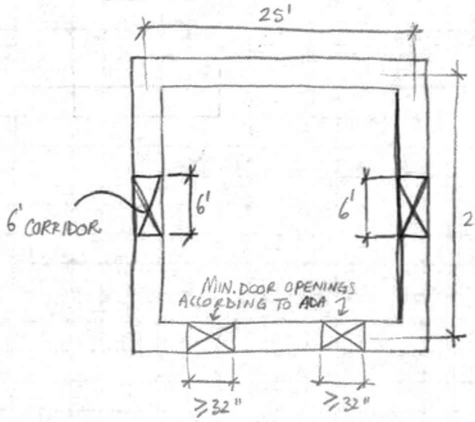
Appendix G – Spot-Check Calculations: Concrete Cores

TECH 1 (REVISION 10.11.2011)
APPENDICES - APPENDIX H
SOUTHWEST STUDENT HOUSING

SPOT CHECKS (CONTINUED)

* CORES

WEAKEST SECTION:



ACTUAL CORE LAYOUT:

AREAS — OUTER CORES: $(25\times12)(8'')+(25-6)(12)(8)(2)+(25\times12-32\times2)(8)=7936\text{ IN}^2$ ← PER CORE

MIDDLE CORE: $(25\times12)(8'')(2)+(25\text{-}6)(12)(8)(2)=8448\text{ IN}^2$

TOTAL CROSS-SECTIONAL AREA @ WEAKEST POINT: $(7936)(2)+8448=24320\text{ IN}^2$

TOTAL AREA ELSEWHERE: $(25\times12)(8)(4)\times3=28800\text{ IN}^2$

GRAVITY LOAD — P = AXIAL LOAD = $L\times W\times 20$

CURTAIN WALL: $15\text{ PSF}\times10'\times(250\times2+52\times2)=90.6^{K}$ (FOR ALL FLOORS ABOVE 1ST FLOOR)

$90.6\times\left(\frac{18'}{10'}\right)=163.1^{K}$ (ON 1ST FLOOR)

CORE SELF WEIGHT @ BASE:

ASSUME CORRIDOR & DOOR OPENINGS ARE 8' TALL

→ OVER 20 FLOORS, $20\times8=160'$ OF HEIGHT W/ WEAKEST SECTION

$\therefore\ SW=(150\text{ PCF})\left[160(24320/144)+(208-160)(28800/144)\right]=5493\text{ KIPS}$

↑ REINFORCED CONCRETE

$$P=1.2\left[\frac{(59+15)(250\times52)\times20}{1000}+90.6\times19+163.1+5493\right]+1.6\left[\frac{40\times250\times52\times20}{1000}\right]$$

← 20 ELEVATED FLOOR SLABS.

← $0.5L_0$ (REDUCED)

$=31941+16640$ FACTORED + REDUCED LIVE LOAD

↑ FACTORED DEAD LOAD

$P_u=48581\text{ KIPS}$

$f_c=\frac{48581}{24320}=1.998\approx2\text{ KSI}$

$P_{MAX}=0.65(0.8)\left[0.85\ f'_c\ A\right]=0.442(4000)(24320)=42999\text{ K}\ .<P_u=48581\text{ K}$

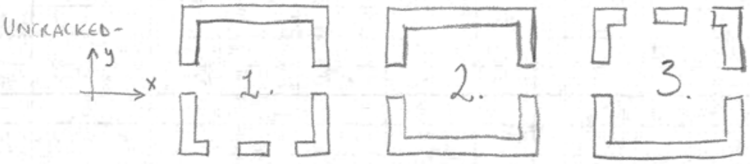
NG

TECH 1 (REVISION 10.11.2011)

SPOT CHECKS (CONT'D)

*CORES (CONT'D)

MOMENT OF INERTIA :



I_x :

1. SIDES – $\frac{bh^3}{12} = \frac{8" \left[(25-6)\times 12\right]^3}{12} \times 4 = 7901568 \text{ IN}^4$

TOP – $\frac{bh^3}{12} = \frac{(25'\times 12)(8")^3}{12} = 12800 \text{ IN}^4$

$Ad^2 = (25'\times 12)(8")\left(\frac{25}{2}\times 12\right)^2 = 54\ 000\ \ 000 \text{ IN}^4$

BOTTOM – $\frac{bh^3}{12} = \frac{(25'\times 12 - 32"\times 2)(8")^3}{12} = 10069 \text{ IN}^4$

$Ad^2 = \left[(25'\times 12 - 32\times 2)(8")\right]\left(\frac{25}{2}\times 12\right)^2 = 42480000 \text{ IN}^4$

$I_{x1} = 7901568 + 12800 + 54000000 + 10069 + 42480000 = 104404437 \text{ IN}^4 = 5035 \text{ FT}^4$

2. SIDES – $\frac{bh^3}{12} = 7901568 \text{ IN}^4$

TOP/BOTTOM – $\frac{bh^3}{12} = 10069 \text{ IN}^4$ $Ad^2 = 54\,000\,000 \text{ IN}^4$

$I_{x2} = 7901568 + 2\times\left[10069 + 54\ 000\ 000\right] = 115921706 \text{ IN}^4 = 5590 \text{ FT}^4$

3. SEE CALCULATIONS FOR 1.

$I_{x3} = 104\ 404\ \ 437 \text{ IN}^4 = 5035 \text{ FT}^4$

$I_X = 5035 + 5590 + 5035 = 15660 \text{ FT}^4$

$E_{STL} = 29\ 000 \text{ KSI}$ $E_{CONC} = 57000\sqrt{4000} = 3605 \text{ KSI}$ $\eta = \frac{E_{STL}}{E_{CONC}} = 8.0$

$\rho_{MIN} = 0.0012$ FOR BARS $\leq$ #5 (ACI 318-05 14.3.2 ⓐ)

$= \frac{A_s}{bd}$

TECH I (REVISION 10.11.2011)

SPOT CHECKS (CONT'D)

☆ CORES (CONT'D)

$$M_{CR} = \frac{f_R I_{UNCR,TR}}{y_{BOTT}} \qquad y_{BOTT} = h - \bar{y}$$

→ NEGLECTING STEEL FOR I.

BECAUSE OF ALMOST SYMMETRICAL LAYOUT OF CORES, (ABOUT X-AXIS) $\bar{y}$ CAN BE ASSUMED @ 150" (THE CENTERLINE) 150" = 12.5'

$$f_R = 7.5\sqrt{f'_c} = 7.5\sqrt{4000} = 474.34 \text{ PSI} = 68305 \text{ PSF} = 68.3 \text{ KSF}$$

$$M_{CR} = \frac{(68.3 \text{ KSF})(15660 \text{ FT}^4)}{12.5'} = 85566.24 \text{ K-FT}$$

M_{WIND} = 68 855 K-FT @ BASE (UNFACTORED) OR 110 168 K-FT (FACTORED) NG

$M_{SEISMIC}$ = 37469 K-FT @ BASE OK

→ SECTION IS CRACKED.

[STILL POTENTIALLY UNTRUE - NEGLECTING STEEL IN I_X CALCULATIONS LEADS TO A SMALLER I_X (SINCE THE STEEL IS EVENLY DISTRIBUTED THROUGHOUT THE WALLS)]

$$I_{X,NEEDED} = \frac{(110\,168 \text{ K-FT})(12.5')}{68.3 \text{ KSF}} = 20162.5 \text{ FT}^4$$

QUICK RECALC OF I_X'S: d IS ALWAYS THE X-CENTERLINE FOR A ~~CONCRETE~~ SEGMENT.
(WITH MORE ACCURACY)

$$I_{TR} = \frac{bh^3}{12} + bh\left(\frac{h}{2} - \bar{y}\right)^2 + (n-1)A_s(d - \bar{y})^2$$

1. SIDES: $\rho = 0.0012 = \frac{A_s}{(8")(150")}$ $A_s = 1.44 \text{ IN}^2$ $d = 150"$

$$\frac{bh^3}{12} + bh\left(\frac{h}{2} - \bar{y}\right)^2 = \frac{(8)[(12.5-3)\times 12]^3}{12} + [12.5-3](12)(8")\left[\left(\frac{12.5-3}{2} + 3\right)\times 12\right]^2 = 3950784 \text{ IN}^4$$

↑ PER CHUNK OF WALL. THERE ARE 4 CHUNKS TOTAL.

$$I_{X\,TR} = 3950784 \times 4 + (8-1)(1.44)(150-150)^2 = 15803136 \text{ IN}^4$$

TOP: $A_s = 1.44 \text{ IN}^2$ $d = 4"$ $(b = 300")$

$$\frac{bh^3}{12} + Ad^2 = 12800 + 54\,000\,000 = 54012800 \text{ IN}^4$$

$$I_{X\,TR} = 54012800 + (8-1)(1.44)(0-150)^2 = 54239600 \text{ IN}^4$$

↑ (ALL AREA & I CALCS WERE DON FROM ℄ OF WALL, WHERE THE REINFORCEMENT IS LOCATED)

BOTTOM: $A_s = 1.44 \text{ IN}^2$

$$\frac{bh^3}{12} + Ad^2 = 10069 + 42480000 = 42490069 \text{ IN}^4$$

$$I_{X\,TR} = 42490069 + (8-1)(1.44)(300-150)^2 = 42716869 \text{ IN}^4$$

$$I_{X\,1\,TR} = 15803136 + 54239600 + 42716869 = 112759605 \text{ IN}^4 = 5438 \text{ FT}^4$$

TECH 1 (REVISION 10.11.2011)

SPOT CHECKS (CONT'D)

✗ CORES (CONT'D)

2. SIDES – 15803136 IN^4
   TOP/BOTTOM – 54239600 IN^4

$I_{x2\ TR}$ = 15803136 + 2 × 54239600 = 124282336 IN^4 = 5994 FT^4

3. SAME AS 1.

$I_{x\ TR}$ = 5438 × 2 + 5994 = 16866 FT^4 ← INSUFFICIENT TO PREVENT CONCRETE FROM CRACKING.

WHAT ABOUT $t = 10''$?

I_x: 1. SIDES – $(3950784)\left(\frac{10}{8}\right) \times 4 = 952.64$ FT^4
TOP – $(12800)\frac{10^3}{8^3} + 54000000\,\frac{10}{8} + (7)(1.44)(150)^2 = 3267.35$ FT^4
BOTTOM – $(10069)\frac{10^3}{8^3} + 42480000\,\frac{10}{8} + (7)(1.44)(150)^2 = 2572.65$ FT^4

2. SIDES – 952.64 FT^4
   TOP/BOTTOM – 3267.35 FT^4

3. SAME AS 1.

$I_{x\ TR}$ = 952.64 × 3 + 3267.35 × 4 + 2572.65 × 2 = 21072.62 FT^4 > $I_{x\ NEEDED}$ $\underline{\underline{OK}}$

$M_{WIND} < M_{CR}$ @ $t = 10''$ → CAN ASSUME UNCRACKED SECTION.

THEORIES FOR DEVIATION: THE ACTUAL DESIGN MIGHT USE MORE REINFORCEMENT. THE DESIGNERS ALSO MAY HAVE CALCULATED THE WIND LOADS DIFFERENTLY.

FROM PROVIDED CALCULATIONS, IT WAS SHOWN THAT THE SECTION WAS ASSUMED UNCRACKED, THUS THE CALCULATIONS TO FIND AN APPROPRIATE GEOMETRY THAT JUSTIFIES THIS ASSUMPTION.

SHEAR CHECK:

NOT A BEAM

$V_c = 2\lambda\sqrt{f'_c}\,A_{TO\ STEEL\ INT.} = 2(1.0)\sqrt{4000}\,\left[8 \times 200 + 4 \times 300 + (25-6) \times 12 \times 8 \times 2\right]$
$= 916.81$ KIPS

$\phi V_n = 0.75(1.0 V_c) = 687.6$ KIPS / CORE × 3 CORES

$\phi V_n = 2062.82$ KIPS

$V_{WIND} = 565^k$ (UNFACTORED) & 904^k (FACTORED) $< \phi V_n$ $\underline{\underline{OK}}$

$V_{SEISMIC} = 235^k < V_{WIND}$ $\underline{\underline{OK}}$

IF RUN W 0.5V_c: $\phi V_n = 0.75(0.5 V_c) = 343.8^k$/CORE × 3 CORES = $1031.4^k > V_{WIND}$ $\underline{\underline{OK}}$