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**09.23.2011**

Architectural Engineering
Pennsylvania State University 2012

# Southwest Housing, Arizona State University

Technical Report #1
Existing Conditions

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# Executive Summary

The intent of Technical Assignment #1 is to provide descriptions and calculations that provide insight into the intricacies of the chosen Senior Capstone Project building. The goal is to have a better understanding of the building, in this case the Southwest Student Housing building in Tempe Arizona, and to be able to describe the various structural systems throughout the building with confidence. This goal is attained through investigation and description of the existing design's structural systems, ranging from the foundation, to the gravity and lateral systems. A detailed discussion of prescribed framing systems and the theory behind their selection for the design allows for a greater understanding of the purpose of the building, structurally.

Gravity and lateral loads were calculated for the building using procedures from ASCE7-05, after which investigations were carried out on typical framing members, portions of floor system, and the lateral and gravity system to evaluate the design. It was found that seismic loading governed the lateral system design, which involves three 8" thick, 25'x25' concrete cores designed as walls according to ACI 318-05.

Most of the structural steel floor framing members were sized to meet service deflection criteria, which required significant camber in addition to upsizing members because of the residual ~2" of deflection at mid-span on typical interior beams with lengths of 52'. The 3-1/2" lightweight concrete on 3", 20 gage metal deck was found to be slightly undersized, with the hand calculations yielding a 18 gage deck as the wiser choice. This deviation, as well as several others in sizing typical interior beams and girders, can be attributed to the more conservative estimation of loading in the hand calculations provided in the appendices of this report.

# 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, 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

[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 and mats, with no pier shaft sized to a diameter of less than 12". The predicted settlement 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).

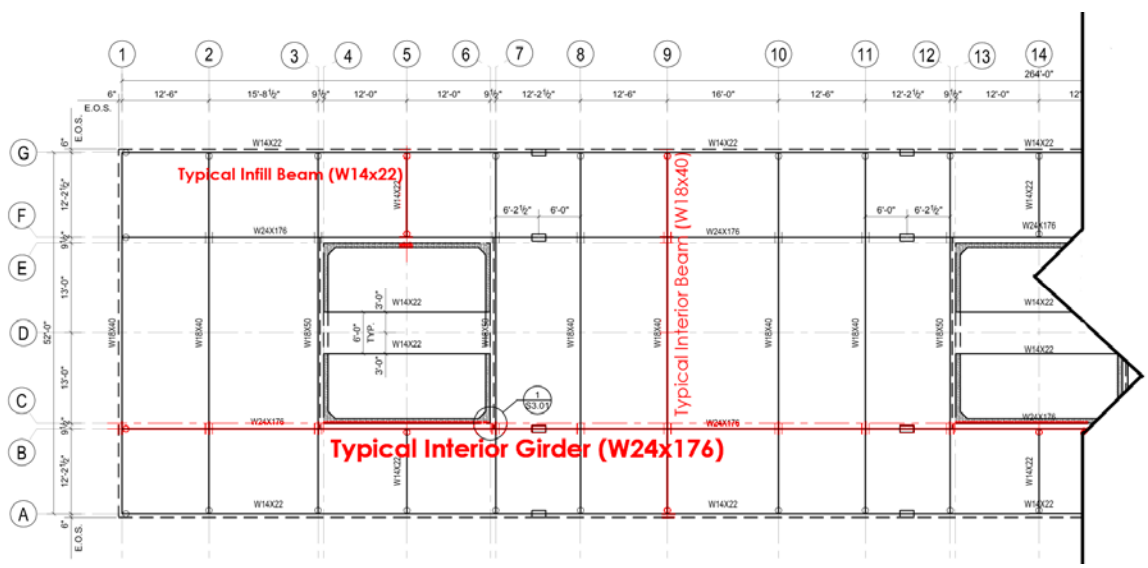


Figure 3: Typical Framing Plan (building is symmetric about line 14)

## 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).

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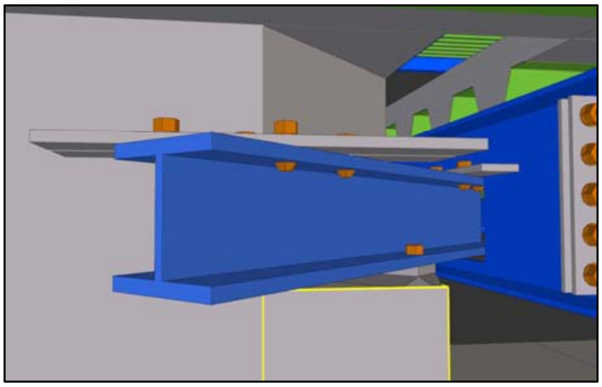

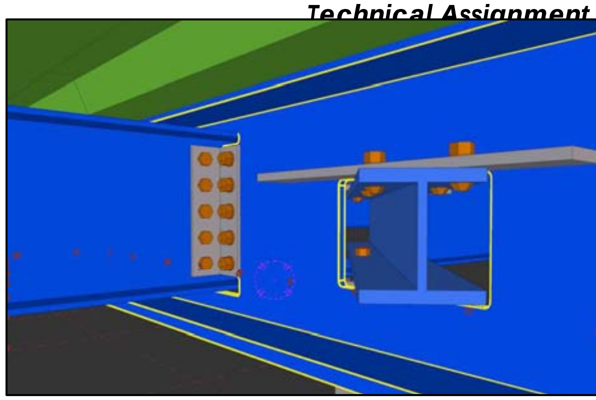

The concrete cores are also the building's sole lateral system, and provide lateral bracing in both directions in the form of shear walls. For clarity, the cores are highlighted in the typical building floor plan below in Figure 5, with boundaries at openings selected. It can be observed in Figure 6 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.

Figures 4.1 and 4.2: Corner detail at every floor, framing into the interior girder to support each level


Figure 5: Typical Building Floor Plan (Core areas are highlighted in rec

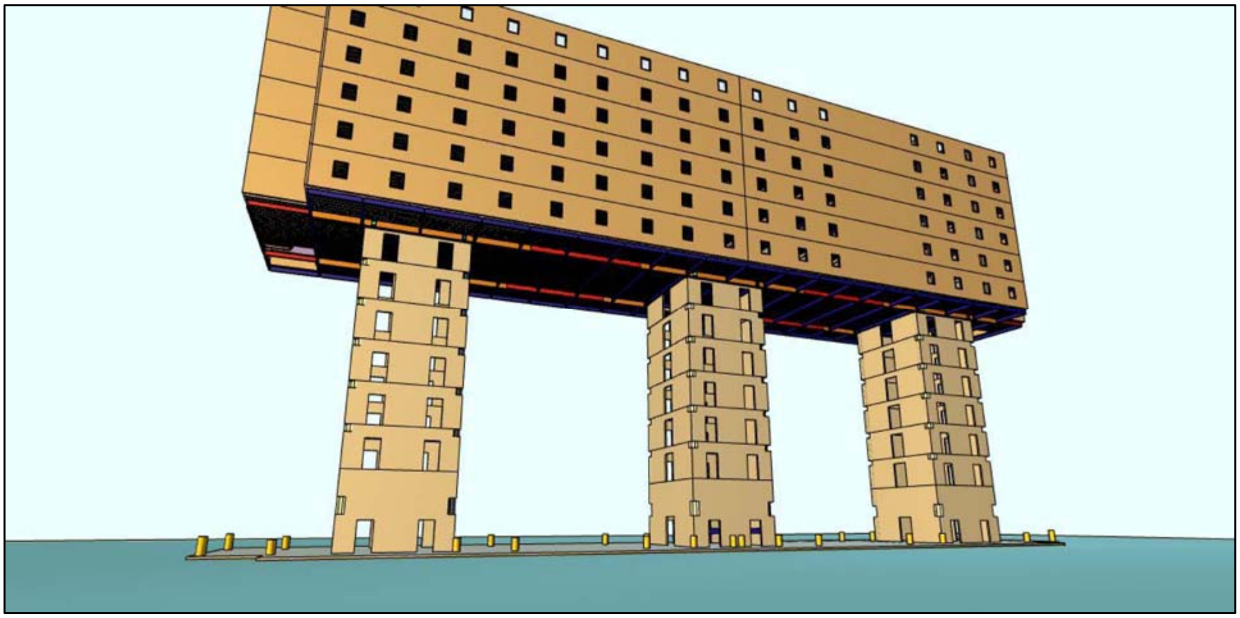

Figure 6: Rendering of visible openings in concrete cores

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 ($13^{th}$ ed.) and AISC 360-10 ($14^{th}$ 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

# 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.; Grade 70 for #9, #10, #11
- Welded Wire Fabric – ASTM A195

Welding Electrodes:

- E70xx Low Hydrogen

Bolting Materials:

- ASTM 325 or A490

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# 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 E-W 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) | |
|---|---|---|---|---|---|---|---|
| | | N-S | E-W | N-S | E-W | N-S | E-W |
| **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 E-W direction has higher base shear, which is the result of the considerably larger façade area in that direction when compared to the N-S direction. The base shear in the E-W direction is almost 5 times as large as the base shear in the N-S direction. The following figures show the pressure distribution on the façade in each direction, as well as a summary diagram of the final calculated wind pressures on the building in each direction.

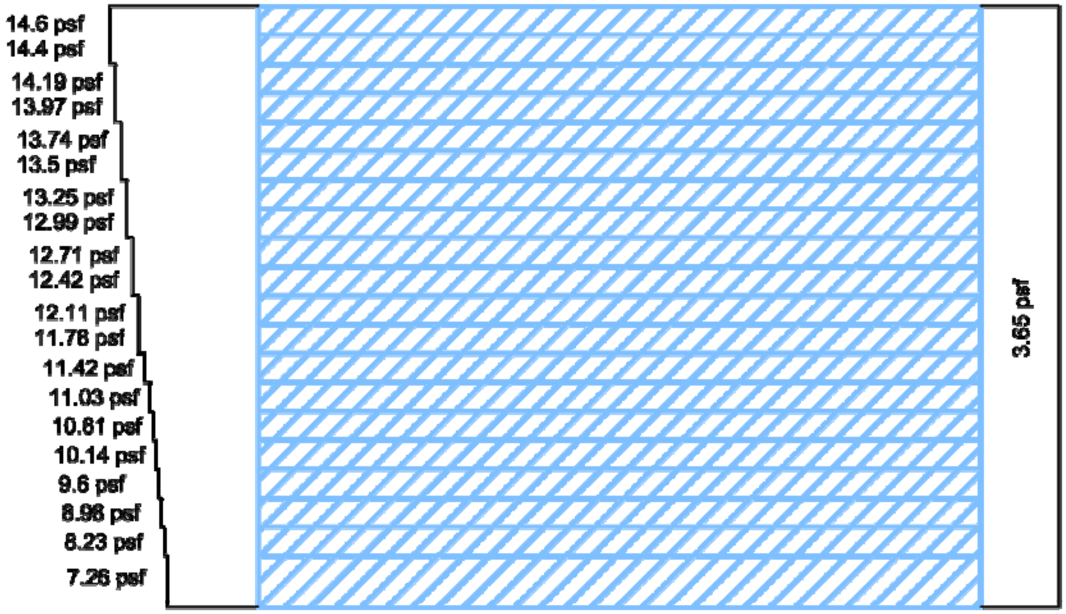



Figure 7: East-West Direction Wind Pressures

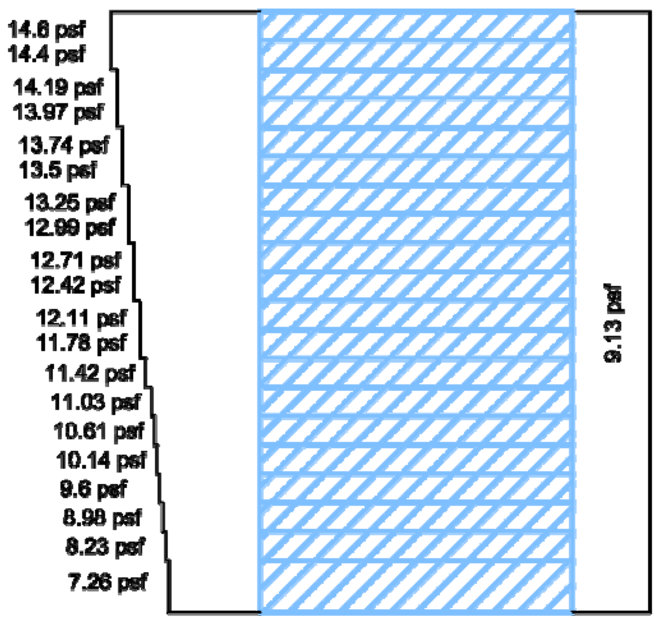



Figure 8: North-South Direction Wind Pressures

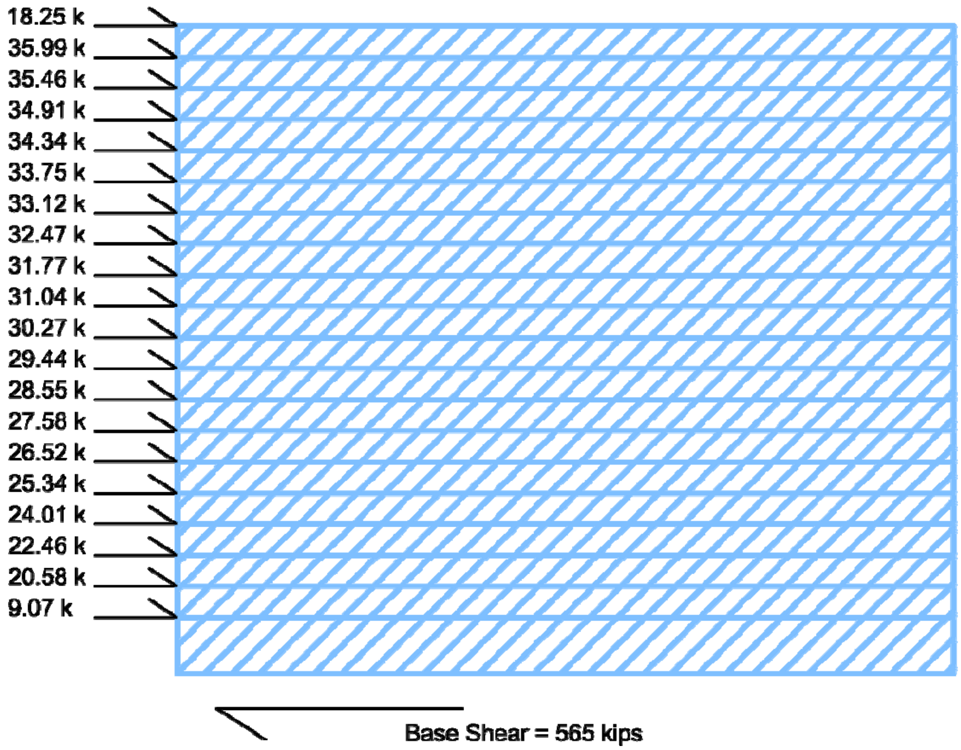



Figure 9: East-West direction wind forces at each story

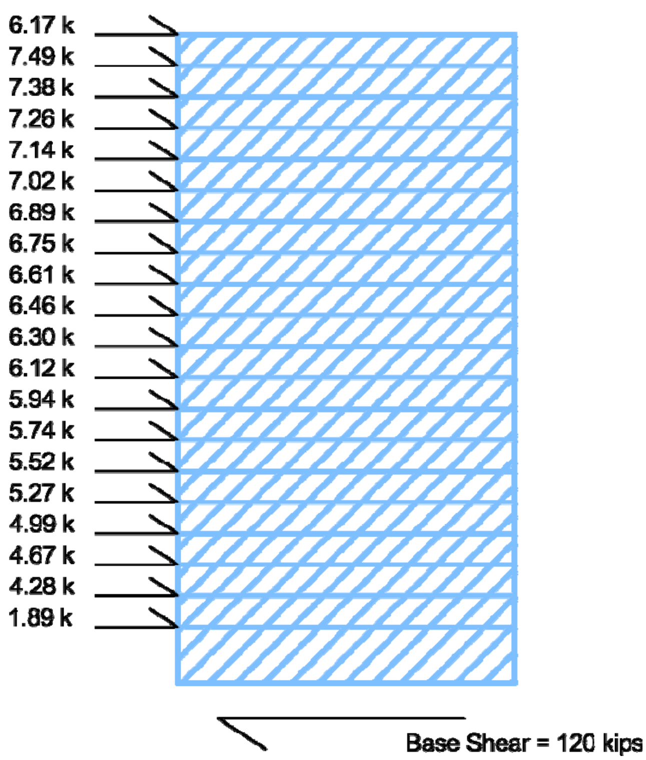



Figure 10: North-South direction wind forces at each story

*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 seismic loads govern this building design. The seismic base shear is 1154 kips, as opposed to the maximum wind base shear of 565 kips (a little over half as much). Seismic loading also produces significantly higher moments, as well as higher story forces. 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 already governs building design.

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

| | | |
|---|---|---|
| **T=** | 1.100 | s |
| **k=** | 2.000 | |
| **$V_b$=** | 1154 | 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) | Moment $M_x$ (ft-k) |
|---|---|---|---|---|---|---|---|
| **Roof** | 208 | 962 | 41619968 | 0.124 | **143** | 0 | **29826** |
| **20** | 198 | 1052.6 | 41266130 | 0.123 | **142** | 143 | **28151** |
| **19** | 188 | 1052.6 | 37203094 | 0.111 | **128** | 286 | **24097** |
| **18** | 178 | 1052.6 | 33350578 | 0.100 | **115** | 414 | **20453** |
| **17** | 168 | 1052.6 | 29708582 | 0.089 | **102** | 529 | **17196** |
| **16** | 158 | 1052.6 | 26277106 | 0.078 | **91** | 631 | **14304** |
| **15** | 148 | 1052.6 | 23056150 | 0.069 | **79** | 722 | **11757** |
| **14** | 138 | 1052.6 | 20045714 | 0.060 | **69** | 801 | **9531** |
| **13** | 128 | 1052.6 | 17245798 | 0.051 | **59** | 870 | **7605** |
| **12** | 118 | 1052.6 | 14656402 | 0.044 | **50** | 929 | **5959** |
| **11** | 108 | 1052.6 | 12277526 | 0.037 | **42** | 980 | **4568** |
| **10** | 98 | 1052.6 | 10109170 | 0.030 | **35** | 1022 | **3413** |
| **9** | 88 | 1052.6 | 8151334 | 0.024 | **28** | 1057 | **2471** |
| **8** | 78 | 1052.6 | 6404018 | 0.019 | **22** | 1085 | **1721** |
| **7** | 68 | 1052.6 | 4867222 | 0.015 | **17** | 1107 | **1140** |
| **6** | 58 | 1052.6 | 3540946 | 0.011 | **12** | 1124 | **708** |
| **5** | 48 | 1052.6 | 2425190 | 0.007 | **8** | 1136 | **401** |
| **4** | 38 | 1052.6 | 1519954 | 0.005 | **5** | 1145 | **199** |
| **3** | 28 | 1052.6 | 825238 | 0.002 | **3** | 1150 | **80** |
| **2** | 18 | 1215.68 | 393880 | 0.001 | **1** | 1153 | **24** |
| | **S** | **21124.48** | **334944008** | **1.000** | **1154** | **1154** | **183606** |

# Spot-Check Calculations

Figure 11 shows the typical floor framing diagram for every floor in this building. Highlighted are the typical interior beam analyzed in the spot-check calculations, as well as the typical interior girder that was analyzed. In addition, the metal deck and concrete cores were also spot-checked.

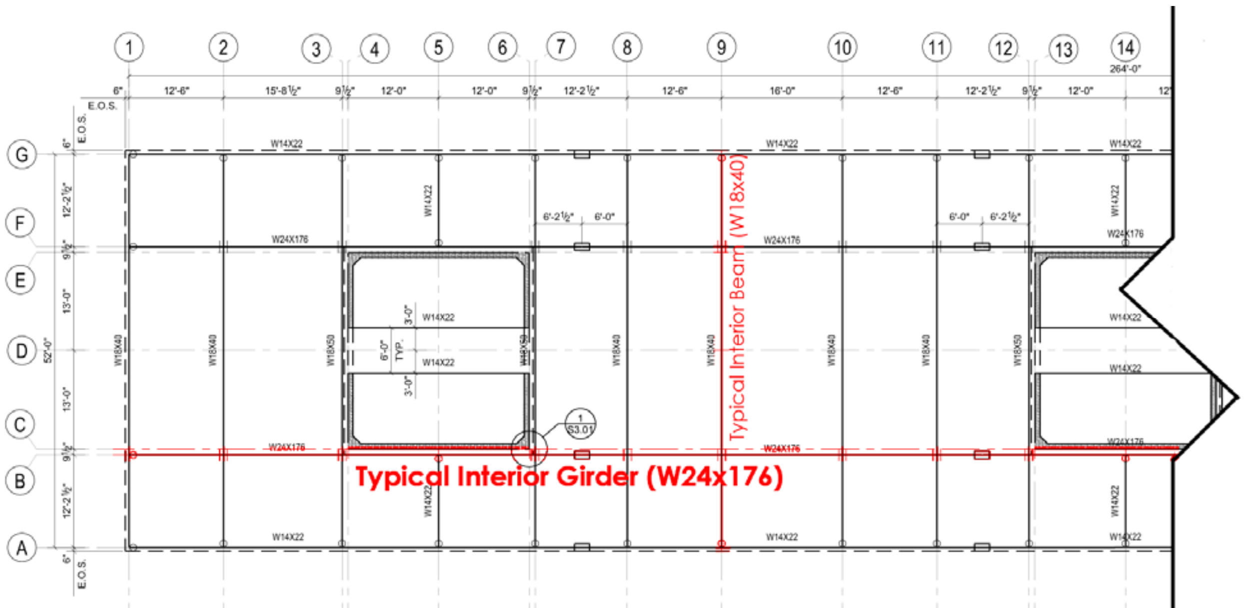



Figure 11: Typical elements chosen for spot-check calculations

## Metal Deck

The metal deck chosen by the structural engineer was a 3", 20 gage deck with 3-1/4" of lightweight concrete. In conformance with these choices, the 3VLI20 deck from the Nucor Vulcraft Steel Deck Manual was chosen for evaluation. Table 4 shows the Vulcraft specified maximum allowable unshored spans and maximum live (service) loads for 3VLI in a variety of gages, with a 3-1/4" thick lightweight concrete slab.

Table 4: Vulcraft specifications for 3VLI with t=3.25"

**(N=14.15) LIGHTWEIGHT CONCRETE (110 PCF)**

| TOTAL SLAB DEPTH | DECK TYPE | SDI Max. Unshored Clear Span | | | Superimposed Live Load, PSF Clear Span (ft.-in.) | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 1 SPAN | 2 SPAN | 3 SPAN | 8'-0 | 8'-6 | 9'-0 | 9'-6 | 10'-0 | 10'-6 | 11'-0 | 11'-6 | 12'-0 | 12'-6 | 13'-0 | 13'-6 | 14'-0 | 14'-6 | 15'-0 |
| | 3VLI22 | 9'-1 | 10'-4 | 11'-6 | 191 | 172 | 155 | 113 | 101 | 91 | 82 | 74 | 67 | 60 | 55 | 50 | 45 | 41 | |
| 6.25 | 3VLI20 | 10'-6 | 12'-10 | 13'-3 | 221 | 198 | 179 | 163 | 149 | 137 | 98 | 88 | 80 | 73 | 66 | 60 | 55 | 50 | 46 |
| (t=3.25) | 3VLI19 | 11'-10 | 14'-2 | 14'-6 | 250 | 224 | 202 | 184 | 168 | 154 | 142 | 131 | 93 | 84 | 77 | 70 | 64 | 59 | 54 |
| 46 PSF | 3VLI18 | 12'-9 | 15'-0 | 15'-0 | 329 | 300 | 275 | 253 | 235 | 218 | 204 | 191 | 180 | 169 | 131 | 122 | 115 | 108 | 101 |
| | 3VLI16 | 13'-4 | 15'-6 | 15'-10 | 374 | 343 | 316 | 293 | 272 | 254 | 239 | 225 | 212 | 201 | 190 | 151 | 143 | 135 | 128 |

The highlighted column features the typical clear span from beam to beam on the typical framing plan, thus the distance that the deck must span. The maximum unshored span for a 2-span condition (as seen on the edges of the building) is 12'-10", for which 3VLI20 is adequate. The 3VLI20 deck is inadequate for the calculated service loads, which add up to 95psf, as opposed to the table's maximum value of 73psf at a span of 12'-6". Calculations for these conclusions can be found in Appendix E.

Ultimately, 3VLI18 is adequate at a span of 12'-6" for the superimposed service loads expected. The difference between the spot-check and the engineer-prescribed decking recommendation most likely comes from the fact that, in the spot-checks, the live load was not reduced in order to take into account the 6' corridor that runs through the center of the building.

The calculated deck conditions in the spot-check calculations are as follows:

*Use 3" Deck (18 gage, minimum), with 3-1/4" lightweight concrete, for a span of 12'-6" and a minimum of 2 spans.*

## Typical Interior Beam

A typical beam running through the structural steel frame of this building covers three spans, but is simply supported by the interior girders that rest on the corner pieces that connect each floor to the concrete cores, with cantilevers framing into girders at the ends. The edge girders serve to keep the floors shaped correctly, and provide little to no vertical support. Simplified drawings of the loaded beam, as well as subsequent calculations for analysis can be found in Appendix F.

Each beam experiences composite action from the metal deck above it. For deflection analysis, the beam can be viewed as a simply supported beam with variable end moments (case 32 in AISC 360-05 13$^{th}$ ed.) applied by the cantilevers at the ends.

The analysis of the typical beam revealed that a W10x22 would be adequate for carrying the loads exerted on the beam, with a $\Phi M_n$ of 171.5 ft-kips to support a $M_u$ of167.3 ft-kips. The problem with the W10x22 is that it fails to adhere to the total load deflection criteria for the middle span, also known as serviceability criteria. A solution to this would be to either camber the beam or increase the beam size. The engineers on this project did both- because, once the girder deflections are taken into account, even the W18x40 they chose to handle the load is inadequate for meeting total load deflection criteria for the middle span. The maximum W18x40 mid-span deflection of 2.077" exceeds the L/240 requirement of 1.3".

## Typical Interior Girder

A typical interior girder, running along the core and supporting the main typical interior beams, involves more complex analysis. The girder is supported at the corners of the concrete core with which it comes into contact. As a result, the typical interior girder has 6 supports, though it is symmetric about its midpoint. Analysis for the typical interior girder, which also experiences composite action, was performed using the moment distribution method, with an additional end moment applied to the evaluated fixed-end-moment in order to account for the moments exerted by the cantilevers that hang off of the final edge supports (25' away from the core).

Moment distribution tables, distribution factor calculations, fixed end moments and diagrams, as well as deflection calculations for the typical interior girder can be found in Appendix G. This appendix also includes the recalculation of the typical interior beam deflections, with the inclusion of maximum girder deflection.

The continuity of the girder over a 250' foot span and 6 supports results in relatively small deflections when considering the overall span. The largest deflection calculated was a deflection due to total load of 0.94" in the 62'-6" span between supports that runs between each core. The maximum allowable deflection due to total loads is 1.25". Girder loading was assumed to be uniform due to the proliferation of beams framing into the typical interior girder (21 in all). Live load serviceability was taken into account, with a reduction applied to the total live load, when calculating deflections.

Torsion of the beam was not taken into account in this analysis: it was assumed that the cores and the metal deck would brace the girder enough laterally to counteract the uneven loading and effective width of the girder (13' on one side of the girder, and 6'-6" on the other side of the girder). It was calculated that 102 shear studs would need to be used in the typical interior girder, which is just under 1 stud every two feet.

The structural engineers that designated beam sizes on this project chose an unusual beam size- W24x176. This size beam is not featured in the composite beam tables (Tables 3-19 and 3-20) in AISC 360-10 (14$^{th}$ ed.). The beam chosen for design through hand calculations was a W27x102. The moment of inertia for the W24x176 is 5680 in$^4$, and the moment of inertia for the W27x102 is 3620 in$^4$. While the moments of inertia are not off-hand comparable, the structural engineer on the project mentioned that most of the typical floor framing doesn't take advantage of the composite action of the concrete on the metal deck. The W27x102 takes advantage of the entire 3-1/4" thickness of concrete when resisting the subjected loading, which increases its overall moment of inertial to 7280 in$^4$, a markedly larger moment of inertia than that for a W24x176.

The reason for this over-conservatism in girder choice when compared to the prescribed girders on the typical framing plan could relate to another over-estimation of the loads applied to the girder. In the moment distribution analysis, all of the point loads applied to the girder (representing beams framing into it) were equal, whereas in reality there are 6 beams that experience around half the typical load (they rest along the girder). Additionally, the live load could be further reduced when calculating the design moment and shear, but were only reduced for testing the live load serviceability.

## 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.

Seismic moment governs over wind loading. Stresses in the concrete cores when evaluating combined seismic and gravity loads results in a tensile stress of 0.3 ksi on one end and a compressive stress of 2.79 ksi on the other end, for a spot-check calculation that looks only at the concrete in the core and doesn't include rebar in the analysis. In theory, the inclusion of a layer of rebar along the center of the wall according to minimum ACI 318-05 standards would alleviate the tension caused by the seismic moment.

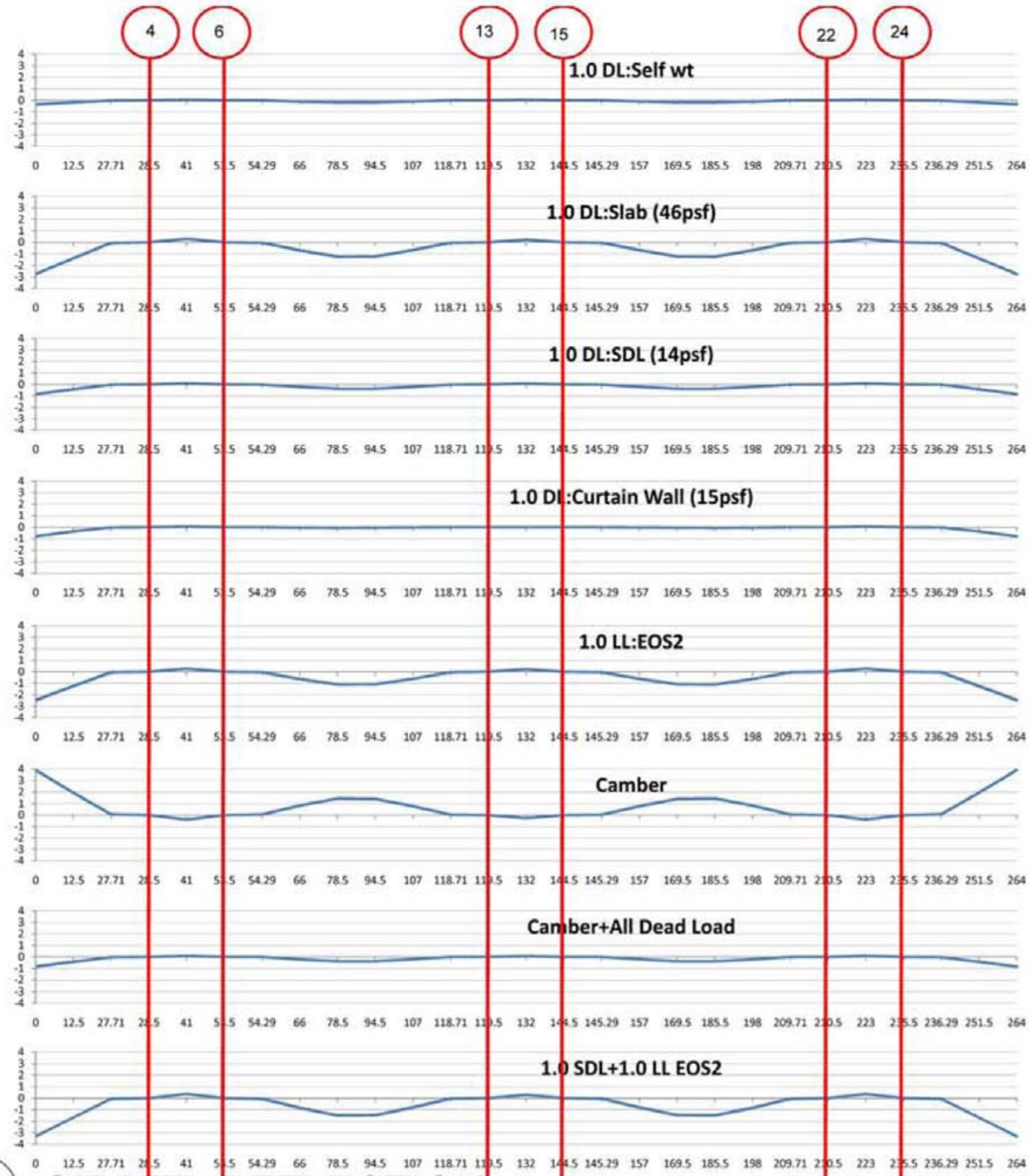
Diagrams and hand calculations for the concrete cores can be found in Appendix H.

# Conclusion

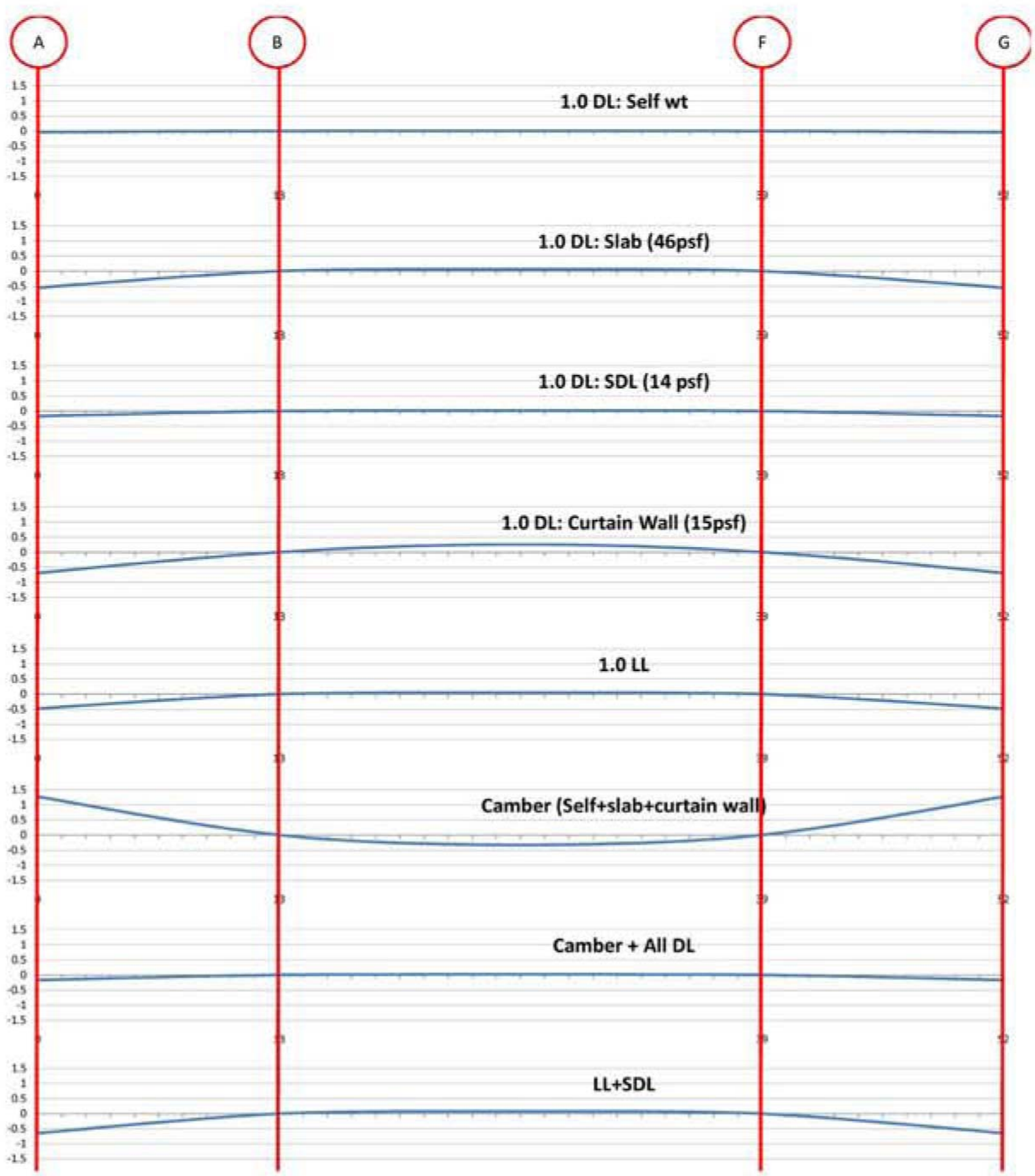
Technical assignment #1 involves exploring existing conditions of the chosen Senior Capstone Thesis building, in this case the Southwest Student Housing building in Tempe, Arizona. Investigations were conducted to obtain structural details for the various building systems including the foundation system, gravity system and lateral system. Loads were evaluated for all basic conditions using design load requirements, equations, cases and methods from ASCE7-05. Spot-checks were then carried out to confirm the prescribed selections for structural elements discussed. It was observed that the structural engineers on the project were not conservative in their estimations, but that they did not make any assumptions that could be deemed dangerous or not allowed by code.

# Appendices

## Appendix A – Building Information Notes




01 GIRDER DEFLECTION

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## Appendix B – Gravity Load Calculations

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## Appendix C – Lateral Load Calculations: Wind

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## Appendix D – Lateral Load Calculations: Seismic

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## Appendix E – Spot-Check Calculations: Metal Deck

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## Appendix F – Spot-Check Calculations: Typical Beam

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## Appendix G – Spot-Check Calculations: Typical Girder

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## Appendix H – Spot-Check Calculations: Concrete Cores

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## Appendix I – Foundation Calculation

**Table 1:** Recommended Allowable Unit Skin Friction and End Bearing Values for Drilled Shafts

| Stratum | Depth (feet) | Skin Friction (ksf) | End Bearing (ksf) |
|---|---|---|---|
| I Silty Sand and Poorly Graded Sand | 10 to 35 feet | 0.4 | N/A |
| II – Sand Gravel Cobble (SGC) | 35 to 100 feet | 2.5 | 30 |

We recommend that the shafts penetrate at least 2.5 times the pier diameter into the Sand Gravel Cobble (SGC) layer. Based on the final grade at the foundation locations, the length of the shaft and the penetration into the bearing stratum should be determined using the recommended allowable side friction and end bearing values. Once the design is finalized, PSI should be given the opportunity to check the final length, embedment depth and bearing elevation. In no case should piers be designed with a shaft diameter less than 12 inches. Piers should

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