Donna Kent
Structural Option
Building: Vickroy Hall
Location: Duquesne University
Pittsburgh, PA 15282
Date: October 5, 2006
Title of Report: Executive Summary for Technical Report I
Faculty Consultant: Dr. Boothby

Executive Summary

This report is designed to explain the existing conditions of the structural system of Vickroy Hall at Duquesne University. The report goes into detail to describe each of the structural systems in the building. This includes the super structure, lateral resisting system, foundations, and roof.

The main code used to design and build the structure was BOCA 1993. Other important standards such as ASTM, ACI, AISC, etc. were also used for design considerations. Using these codes, or similar (BOCA 1999), as was my case because BOCA 1993 was not available, computations of wind loadings, seismic loadings, and gravity were determined. There are figures within the report and appendix that show the calculations and summarize the findings. If more calculations are necessary, they are available for review.

Periodic pictures are placed throughout the report to better explain and visually illustrate certain aspects in the design, systems, or details. These may be replicas of the original drawings, sketches, or photos taken from the site.

Donna Kent
Structural Option
Building: Vickroy Hall
Location: Duquesne University
Pittsburgh, PA 15282
Date: October 5, 2006
Title of Report: Structural Technical Report 1
Faculty Consultant: Dr. Boothby

1. Introduction

Vickroy Hall is a 77,000 square foot, eight floor Living/Learning Center for the upper class students of Duquesne University. It features suites with two double rooms with an attached private bathroom (see Figures 1 through 3). The building also contains laundry facilities and multipurpose rooms. Each room is approximately 150 square feet and contains at least one window. Some rooms have two or three because of their location within the building.

This report is designed to explain the existing conditions of the structural system of Vickroy Hall at Duquesne University. It will provide an overview of each of the systems including the super structure, lateral resisting system, foundations, and roof. It will also give details of the codes used to design and build the structure, as well as computations of the members for the structure using the loads determined from the codes used.

2. Building Breakdown

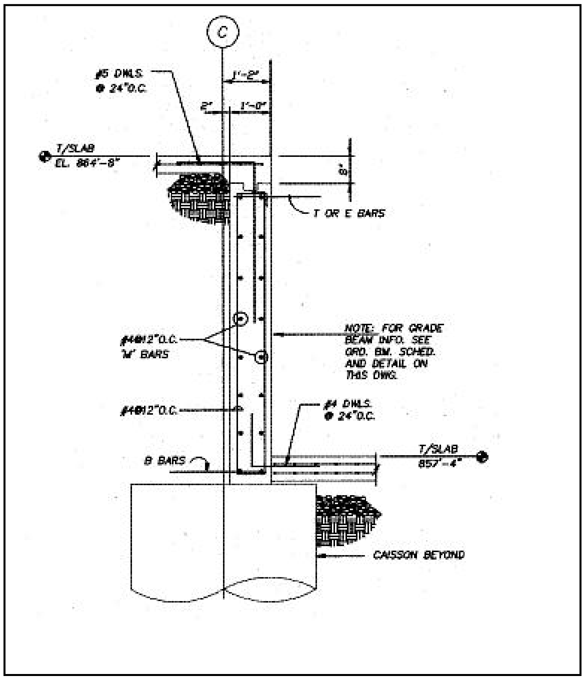
2.1 The Foundation

The foundation consists of grade beams and slabs on grade formed on top of caissons (see figure below).

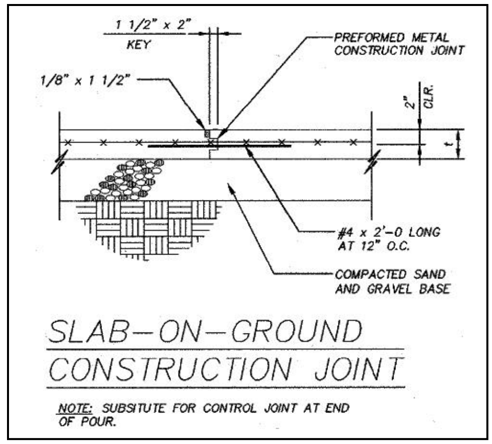
The caissons are constructed of reinforced concrete with a capacity of twenty-five tons per square foot. The caisson holes were to be drilled until auger refusal and then cast in place. The size of the caissons range from thirty to fifty-four inches in diameter.

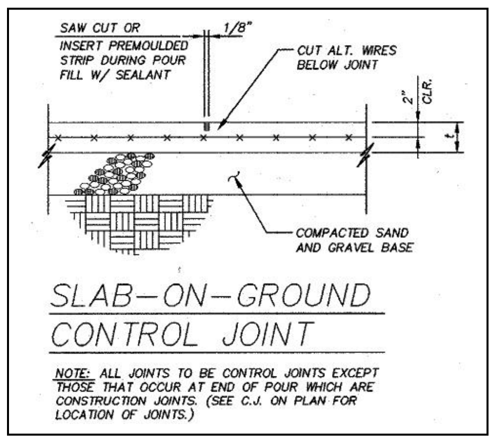
The grade beam widths are from twelve to sixteen inches wide with an average depth of thirty-four inches, but with a maximum depth of eighty-eight inches. The deep grade beams are in and around the elevator shafts and mechanical rooms because of the greater support needed in those areas.

The slabs on grade are generally four inches thick with 6 x 6 – W2.9 x W2.9 welded wire fabric reinforcing over six inches of compacted sand and gravel sub base with a vapor barrier. Beneath the mechanical equipment rooms and elevator shafts, the slabs are thicker, but the depth was not revealed on the structural drawings. This is because the Structural consultants were waiting for the weights of the mechanical equipment to further design those slabs, including the welded wire fabric reinforcing for those slabs. Control and construction joints are found along all column lines excepting those with walls. The type of joint depends on the location within the slab placement. The ends of pours were to be at a column line for the specific reason of placing a construction joint. The control joints were saw cut or had pre-molded strips which were later filled with sealant. The construction joints are composed of a preformed metal joint and also filled with sealant of a predetermined depth and width. The construction joints are formed with a 1-1/2” x 2” key and reinforced with #4 bars 2’-0” long at one foot increments (see figures below).



Caisson, Grade Beam and Slab Detail

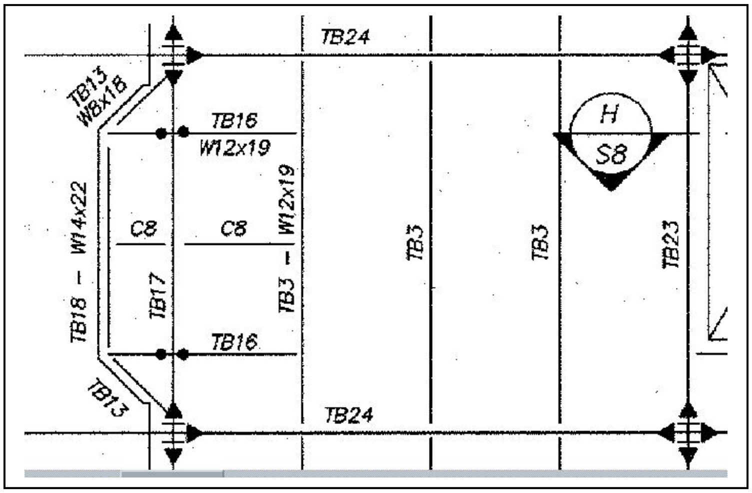




One difficulty in building the foundation was that the building was proposed to go directly over an existing utility tunnel. This tunnel housed pipe lines, communication, and electrical wires. The contractors cut the lines temporarily within the tunnel and proceeded with construction, repairing the lines as the foundation was built. Now, the lines run through the building along the inside of the wall (see Figure 5), and the tunnel continues on the outside of the foundation walls on either side of the building.

2.2 The Super Structure

The main structural system consists of structural steel members. These include W-shapes and C-channels. Each connection is a moment connection, indicated on the drawings as either a wind moment connection or a moment resisting connection. A typical floor plan calls for generally calls for W12 to W16’s. There are also C-channels framing the protrusions of the buildings perpendicular to the regular framing system (see figure of partial typical framing plan (full typical framing plan is in the appendix – Figure 4)).



Typical Framing Plan showing Partial Framing of cantilevered protrusion

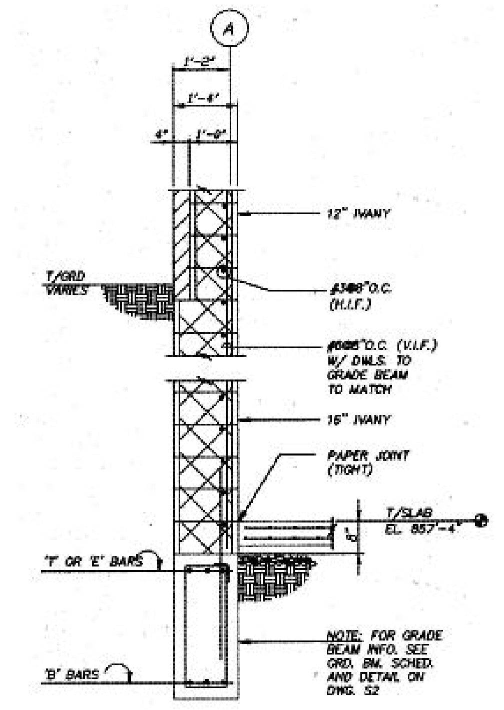
2.3 The Floor System

The floor system is a composite metal and concrete deck. On a typical floor, the deck is 2” - 20 gage with 3 – ¼” light weight concrete with 6x6 - W2.9 x W2.9 welded wire fabric. The composite deck is to span a minimum of two spans. The deck was to be welded to the supporting structural member (see photo below).

Typical Floor System: Shows corrugated metal deck supported by steel framing

2.4 The Lateral Resisting System

At the foundation/first floor level, the walls consist of a reinforced unit masonry system with 16” Ivany blocks below grade and 12” Ivany blocks above grade. In front of the Ivany block, the wall system changes to that of a brick façade. Behind the brick façade, there are 6” – 16 gage structural metal studs with batt insulation between the framing components. Relief angles are positioned at every floor for the brick façade. The windows are composed of aluminum with plastic laminate sills (see photos below).

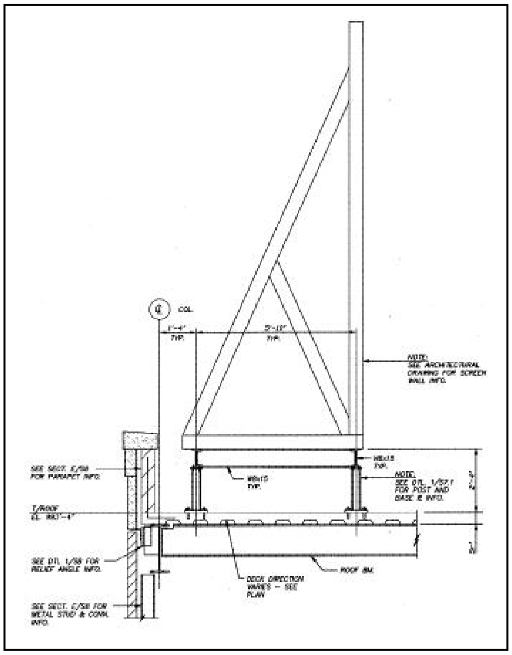


(Above) Reinforced masonry wall

(Right) Reinforced masonry wall detail showing transition from 16” Ivany block to 12” Ivany block

2.5 The Roofing System

The building was designed so an extra six floors could be added to the eight floors which were built for the first phase. Therefore, the roof is designed as a floor with the capacity to hold the same loads. As a result of this, what appears to be a hipped roof is actually light gage metal framing with standing seam metal panels attached called a ‘screen wall’. The framing is mounted to the ‘floor’ system below. This floor system is like the lower levels with the corrugated metal deck and reinforced light weight concrete. The framing is attached through embedded anchor bolts within the concrete. Around the perimeter of the roof is a ten inch parapet. This is composed of concrete masonry units with a metal coping covering. The ‘floor’ system is covered with tapered insulation, EPDM, and ballast (see photo and detail on next page).



(Left) Roof Screen Wall Framing Detail
(Above) Roof Screen Wall and roofing system

2.6 Strengths of Materials

Concrete:

- Slab on Grade, Floor Systems: 3,000 psi at 28 days
- Caissons and Grade Beams: 4,000 psi at 28 days
- Foundations: must have Type II or Type V cement with pozzilith mixture

Steel:

- Reinforcement: 60,000 psi minimum yield
- W shapes: 36,000 psi minimum yield
- Channels, angles, plates, connection materials: 36,000 psi unless other wise noted
- Tubes: 46,000 psi minimum yield

Welds:

- E70XX electrodes

Bolts:

- Regular: all will be ¾" diameter A-325 High Strength friction or bearing type with threads in the shear plane
- Anchor: A-307 or A-36

Façade:

- Ivany Block: 3,000 psi minimum at 28 days
- CMU: 1,500 psi minimum
- Brick: 4,000 psi minimum

Grout:

- Ivany Block: 3,000 psi

- Masonry Unit core filler (grout): 2,500 psi

Mortar:
- Below Grade or in contact with Earth (for Concrete Masonry Units) : Type M
- All other masonry: Type S

3. Codes and Code Requirements

3.1 Applicable Codes and Standards

- BOCA 1993
- AISC (Ninth Edition)
- National Electric Code Standards
- NEMA Standards (National Electric Manufacturers Association)
- ASTM Standards
- AWS Standards
- AISI Metal Standards
- ASCE 7 Standards
- ACI Standards
  - ACI 318 – “Building Code Requirements for Reinforced Concrete”
  - “Detailing Manual No. SP-66”

3.2 Code Requirements

- Gravity (as designed):
  - Roof: 80 psf due to requirement to have capacity of a typical floor for future construction of six additional floors
  - Floor: 80 psf (include 20 psf for partitions)
  - Mechanical Rooms: 150 psf
  - Stairs: 100 psf
  - Corridors: Ground Floor: 100 psf, Other: 80 psf
- Lateral Load (as designed):
  - Exposure B
  - 70 mph wind velocity

4. Analysis

*****General Note: Neither BOCA 1993 nor BOCA 1996 were available in the libraries for my analysis. Therefore I used BOCA 1999 for my analysis. I assumed that the 1999 BOCA had not changed dramatically for 2 issuances of the code.

*Full calculations may be reviewed if requested.

*Design Loads not accounted for: Lateral soil pressure, wind uplift, heave of soils under foundation, building has protrusions and large exposed masonry and steel columns (will change wind loading due to pressures going in between columns – drag, etc.).

4.1 Classification

- Residential Group R-2 for Dormitories
- Type 3 Construction

4.2 Gravity Loading and Spot Check Analysis

- Category II Occupancy
- Pg = 25 psf
- Live Loads:
    - Corridors: First floor = 100 psf, Other = 100 psf
    - Dwelling Units: 40 psf
    - Roof Load: 20 psf minimum
    - Snow Load:
        - Exposure B
        - Ce = 1.0
        - Ct = 1.0
        - I = 1.0
    - *However, the roof is to be a floor if the University decides to add the additional six floors that it was designed for. Therefore, the roof load including snow, etc. is 80 psf.

Spot Check Analysis

Spot Check 1: Typical beam on a typical floor plan between Column Lines 2&3 and D&E (see Figure 6)

- Assumptions:
    - Fully braced due to metal decking
    - Span is typically 18’-8” to 19’-10”, therefore conservatively say 20’ span
    - Typically 6’ on center from beam to beam (24’ bay size)
    - Assuming simple beam for this analysis
    - Greatest moment stems from loading case of 1.2D + 1.6L
- Results:
    - The beam I designed was a W12x19.
    - The beam they designed was a W14x22.
    - I attribute the differences in the design as to the loading from other items and the system acting as a whole (moment frames)
    - The results vary in the moment distribution in the next spot check.

Spot Check 2: Typical beam required to support loaded floor slab and check for fully composite action (see Figure 7)

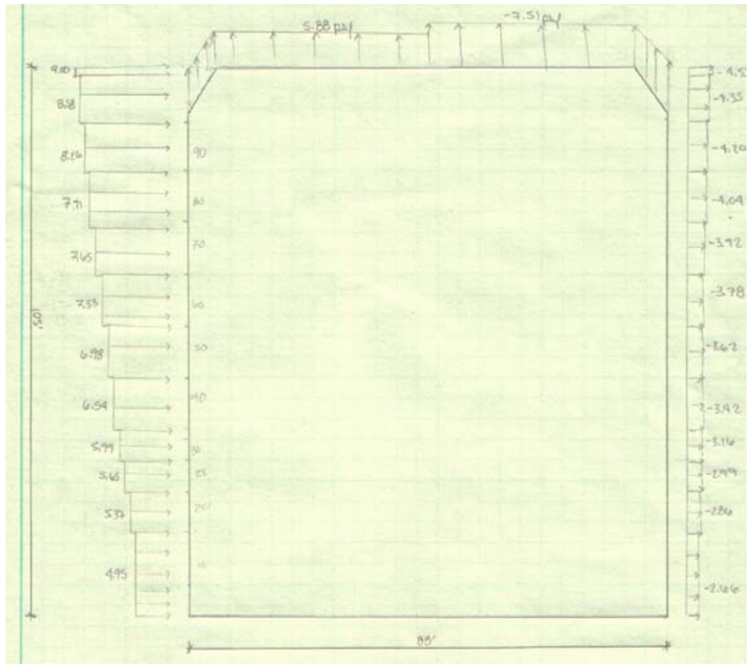
- Assumptions:
  - From www.jamesriversteel.com/comp2.htm and by design
    - 2" – 20 gage metal decking weighs 2.05 psf
    - Corrugation is 2" with 6" troughs
  - By design: 3.25" concrete with WWF reinforcing
  - Use 150pcf for concrete to account for reinforcing
  - 20' span (conservatively)
  - Loading comes from 1.2D + 1.6L
  - Fully braced due to decking
  - Simply supported beam
- Results:
  - By design, the beam was a W14x22
  - I designed a W12x22
  - The beam was ok in both moment capacity and required area of steel.
  - The Plastic Neutral Axis was in the Concrete as assumed in the calculations
    - Therefore, the beam has full composite action and the deck does not contribute to the bending

4.3 Wind Analysis: Main Wind Force Resisting System

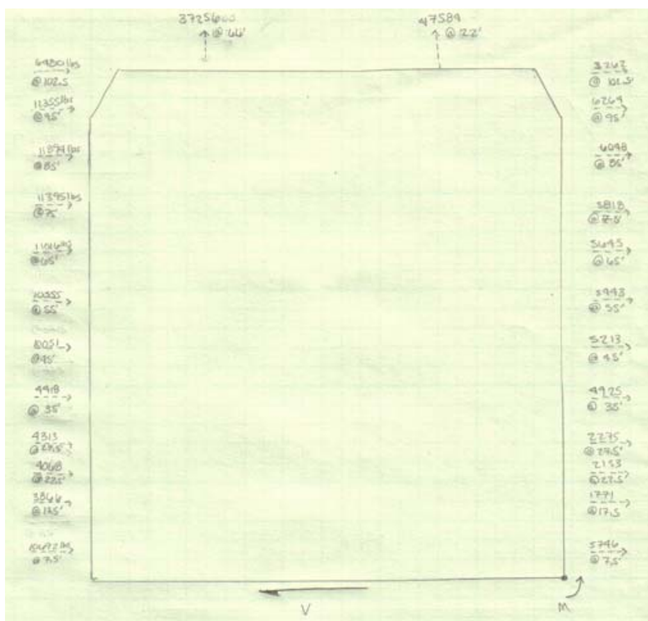
Long Side of Building Windward

Long Side Windward: MWFRS							
z	Kz&Kh	Gh&Gz	Windward		Leeward		
			P=PvI[KzGhCp-Kh(Gcpi)]		P=PvI[KzGhCp-Kh(Gcpi)]		
			Wall	Roof	Roof	Side Walls	Leeward Wall
0-15	0.37	1.65	4.95	-2.68	-4.19	-4.19	-2.66
20	0.42	1.59	5.37	-2.98	-4.53	-4.53	-2.86
25	0.46	1.54	5.65	-3.21	-4.76	-4.76	-2.99
30	0.5	1.51	5.99	-3.45	-5.04	-5.04	-3.16
40	0.57	1.46	6.54	-3.86	-5.50	-5.50	-3.42
50	0.63	1.42	6.98	-4.21	-5.86	-5.86	-3.62
60	0.68	1.39	7.33	-4.49	-6.15	-6.15	-3.78
70	0.73	1.36	7.65	-4.76	-6.41	-6.41	-3.92
80	0.77	1.34	7.91	-4.99	-6.62	-6.62	-4.04
90	0.82	1.32	8.26	-5.27	-6.91	-6.91	-4.20
100	0.86	1.31	8.58	-5.50	-7.17	-7.17	-4.35
120	0.93	1.28	9.00	-5.88	-7.51	-7.51	-4.53

Pv (psf) =	12.5				
I =	1				
h = 105	L = 88'	B = 144'	L/B = 0.61	h/L = 1.19	
Cp =	Windward	Leeward	Side	Roof (leeward)	Roof (windward)
	0.8	-0.5	-0.7	-0.7	-0.2
GCpi =	0.25	-0.25			



Wind Forces in psf



Wind Forces in Pounds

Overturning Moment from wind force:

M = 14,493 ft-kips

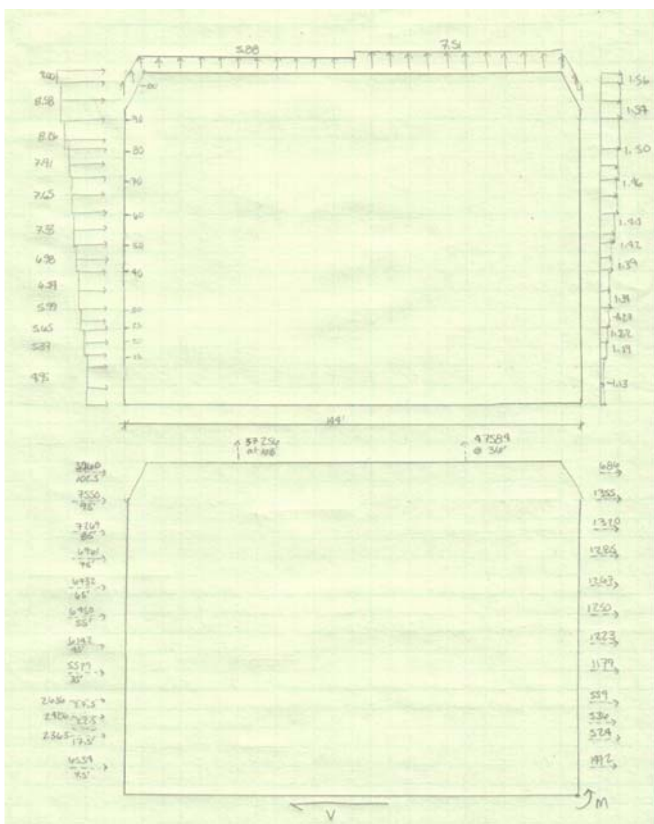
Shear at Ground level of building from wind force:

V = 156 kips

Short Side of Building Windward

Short Side Windward: MWFRS							
z	Kz&Kh	Gh&Gz	Windward P=PvI[KzGhCp-Kh(Gcpi)]		Leeward P=PvI[KzGhCp-Kh(Gcpi)]		
			Wall	Roof	Roof	Side Walls	Leeward Wall
0-15	0.37	1.65	4.95	-2.68	-4.19	-4.19	-1.13
20	0.42	1.59	5.37	-2.98	-4.53	-4.53	-1.19
25	0.46	1.54	5.65	-3.21	-4.76	-4.76	-1.22
30	0.5	1.51	5.99	-3.45	-5.04	-5.04	-1.27
40	0.57	1.46	6.54	-3.86	-5.50	-5.50	-1.34
50	0.63	1.42	6.98	-4.21	-5.86	-5.86	-1.39
60	0.68	1.39	7.33	-4.49	-6.15	-6.15	-1.42
70	0.73	1.36	7.65	-4.76	-6.41	-6.41	-1.44
80	0.77	1.34	7.91	-4.99	-6.62	-6.62	-1.46
90	0.82	1.32	8.26	-5.27	-6.91	-6.91	-1.50
100	0.86	1.31	8.58	-5.50	-7.17	-7.17	-1.54
120	0.93	1.28	9.00	-5.88	-7.51	-7.51	-1.56

Pv (psf) =	12.5				
I =	1				
h = 105	L = 144'	B = 88'	L/B = 1.64	h/L = 0.73	
Cp =	Windward	Leeward	Side	Roof (leeward)	Roof (windward)
	0.8	-0.3	-0.7	-0.7	-0.2
GCpi =	0.25	-0.25			



Wind Forces (top in psf, bottom in pounds)

Overturning Moment from wind force:

M = 10,185 ft-kips

Shear at Ground level of building from wind force:

V = 78 kips

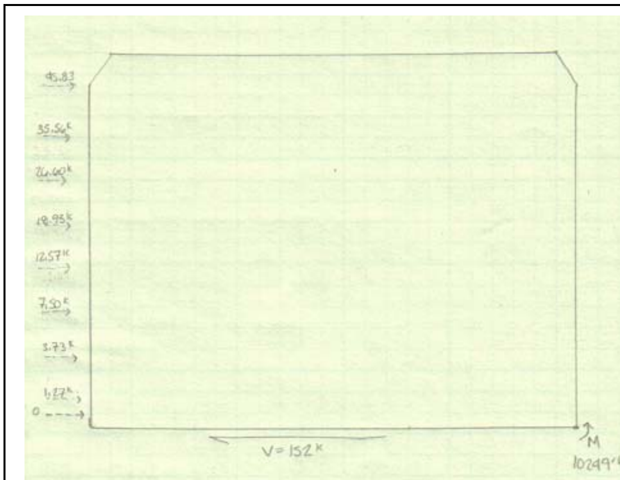
4.4 Seismic Analysis

Assumptions:

- Typical Floor: 77,000sf / 8 floors = 9625 sf
  - However, the Mezzanine Mechanical floor is approximately 200sf
  - Therefore, a typical floor is 9600 sf
- Group E, Group II Seismic Hazard Group
- Av <0.05
- Aa <0.05
- Seismic Performance Category A
  - Not required to be analyzed for seismic forces for the building as a whole
  - MWFRS shall be deemed the seismic resisting system
- Site Coefficient: 2.0 because of unknown site conditions (Geotechnical Report not received)
- Building is designed as Ordinary Moment Frame
  - R = 4.5
  - Cd = 4
  - Height is unlimited
- Gravity Loads: 90 psf for both roof and floors

Seismic Forces				
Story	hx^k*Wx	Cvx	Fx	Mx
Ground	0	0	0	0
Mezz	364.5	0.0000	0.00	0.19
1	217101.4	0.0083	1.27	100.44
2	639132.1	0.0246	3.73	253.36
3	1283629	0.0493	7.50	423.87
4	2150591	0.0827	12.57	567.77
5	3240019	0.1246	18.93	640.88
6	4551913	0.1750	26.60	599.00
7	6086272	0.2340	35.56	397.97
Roof	7843097	0.3015	45.83	-6.42
Total	26012118		152.00	

Wx (psf) =	866
Wx mezz =	18
k =	2.00
h (ft) =	11.33
Hmezz (ft)	4.5
V (k) =	152



Seismic Forces

Technical Assignment 1 Notes

- Some assumptions made in this report were later revised in Technical Assignment 2 and 3. They include the following:
    - Simply supported beams are not the case as the major beam to column connections are all moment connections.
    - Wind speeds and seismic forces were determined in this assignment using BOCA 1999, as a 1993 version was not available. I chose to use the older code so that I could attempt to get the same member sizes and forces as the original engineers. Using an older code also allowed me to gain some necessary experience in interpreting a code that I was not used to (for example, IBC 2003).
    - The wind speeds and seismic forces were determined using IBC 2003 and ASCE-07 in Technical Assignments 2 and 3.
    - The floor loading used: 90 psf, was also revised in the later Technical Assignments to reflect the new code.

Appendix

Figure 1: Double Room Suite

Figure 2: Joint Bathroom showing adjoining suite door

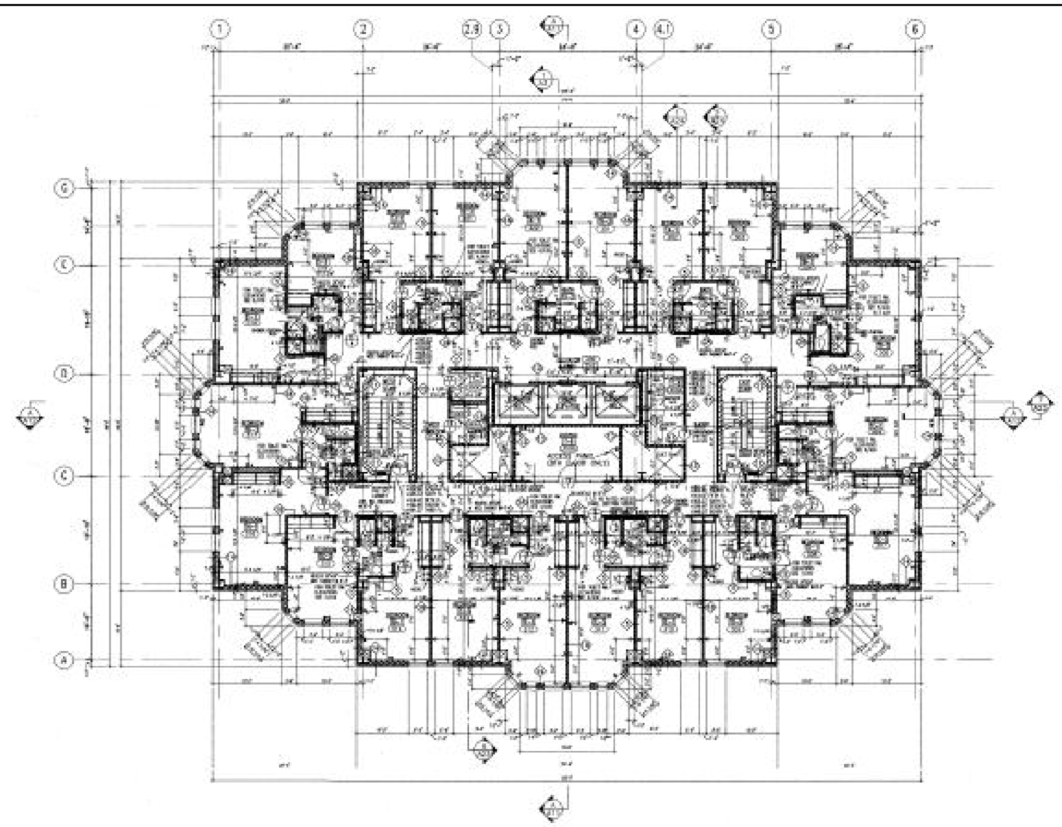


Figure 3: Typical Floor Plan

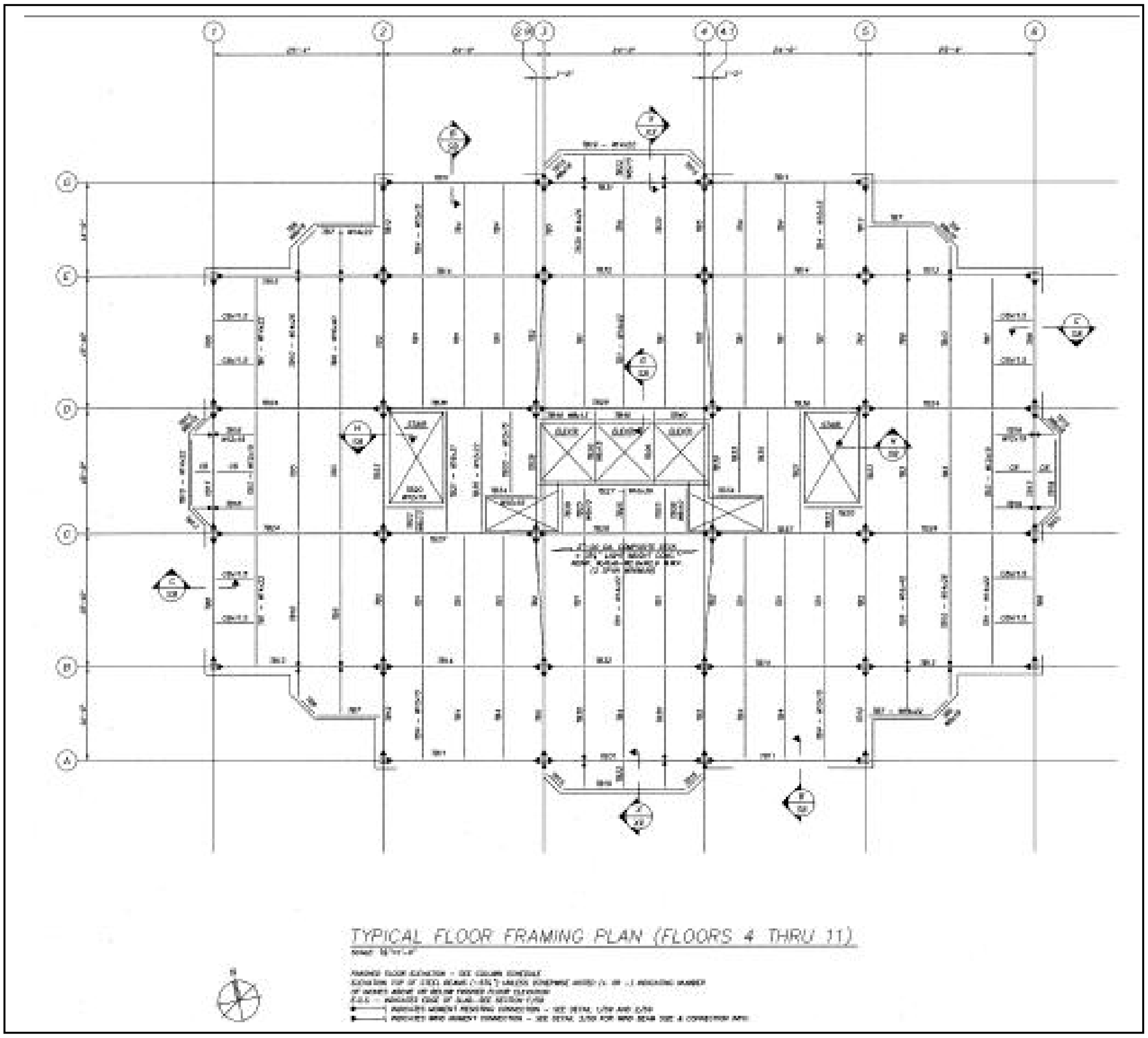


Figure 4: Typical Framing Plan

Figure 5: Steam Lines, communication lines, and electrical lines now run through the building and connect to underground tunnels on the other side of the foundation walls.

Gravity Loads:

- Floor Loads: deck and concrete: $3.25(150\,pcf)/12''/' + 10\,psf$ (estimated weight of deck) $\approx 50\,psf$
  - floor covering: 2 psf
  - ceiling : 2 psf
  - MEP : 10 psf
  - Collateral : 5 psf
  - Partitions/walls: 20 psf
  - $\approx 90\,psf$.
- roof load: same as floor load

Typical floor Load Beam check (between Column Lines 2+3 and D+E)

span: typically 18'-8" to 19'10" ∴ use larger and conservatively say 20'

Width: typically 6' on center

Worst moment:

$W_u = 1.2D + 1.6L$

$= 1.2(90\,psf)(6') + 1.6(100\,psf)(6')$

$= 1608\,plf$

$= 1.6\,klf$

assuming simple beam for simplified analysis

$M = wl^2/8 = 80'\,kips$

$V = 1.6(20)/2 = 16\,kips$

fully braced due to metal decking

Try W12x19 $\phi M_p = 92.6 > 80$ ∴ ok

check Z_{req}

$M_u = F_y z$

$Z_{req} = \phi M_u / F_y$

$= 80'(12''/') / [60(0.9)]$

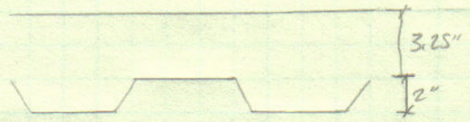
$Z_{req} = 17.76\,in^3 < Z = 24.7\,in^3$

Designed Typical Beam size for between Column Lines 2+3 + D+E is a W14x22

Figure 6: Spot Check for Typical Beam Sizes

BEAM REQUIRED TO SUPPORT LOADED FLOOR SLAB

- BY DESIGN: 2" - 20 gage metal decking; 3.25" concrete w/ WWF, 6' spacing between beams
  from www.jamesriversteel.com/comp2.htm
  - corrugation is 2" with 6" troughs



- weight = 2.05 psf

- concrete: use 150pcf due to reinforcing
  $C_w = 3.25"(150pcf)(1'/12")(6')$
  $= \cancel{244}$ plf
  $f'_c = 3000$ psi

- LL: 90psf

- Total Load:
  $W_u = 1.2(244 + 2.05(6)) + 1.6(90)$
  $W_u = 451.6$ plf

- Moment (assume simple beam)
  - 20' span

  $M = \frac{wl^2}{8} = 22.6'k$

- $b_{eff} \le \frac{1}{4}$span $\Rightarrow \frac{20}{4} = 5' = 60"$ controls
  $\le$ spacing $\Rightarrow 6'$

- To be stable (assuming PNA is in concrete)
  $C = T$
  $C = 0.85 f'_c b_{eff} t$
  $T = A_s F_y$

  $0.85 f'_c b_{eff} t = A_s F_y$
  $0.85(3)(60)(2") = A_s(60)$
  $A_{sreq} = 5.1 in^2$

- Try W12x22 $A = 6.48 in^2 > 5.10 in^2$ ∴ ok $\phi M_n = 110'^k >> 22.6'^k$ ∴ ok

- Make sure steel controls
  $C_c = 0.85(3)(60)(2) = 306$
  $T = A_s F_y = 6.48(60) = 388.8$
  } concrete controls

  $a = \frac{C}{\beta_1 f'_c b_{eff}} = 2"$ ∴ in concrete ∴ deck does not contribute to bending

- Beam will support and have full composite action

Figure 7: Spot Check for Full Composite Action