Daniel Hancock
Structural Option
Dr. Hanagan

S&T Bank
Corporate Headquarters

Indiana, PA

Executive Summary

The S&T Bank located at 800 Philadelphia Ave, Indiana PA is the corporate headquarters of the bank. S&T Bank is a 4-story steel frame building. The foundation consists of spread footings to support the weight of the building. The framing of the building forms directly into the columns. The floor system is a form deck supported by joists that are 2'-0" on center. To resist lateral loads (from seismic or wind) many of the connections are moment connections. These moment connections however are fastened by "wind clips", which only resist partial lateral loads.

During the analysis of the structure, as can be seen in the appendices, the girder and column that was calculated came to be the same as the girder and column that is used in the structure, W24x68 and W12x53 respectively. The decking also came out as was expected, or at least close to the same. The existing deck is 28 Gage Bowman SF-1 and the calculated decking is 28 Gage UFS deck. Both of these decks are the same depth and thickness. However there were some slight discrepancies when designing the joists. Where the joist design originally called for a 22k2 joist @ 2'-0" on center, the calculated joist for a load of 210psf and a span of 28' turns out to be a 22k4 joist (as provided in Vulcraft Joist Design Manual).

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Structural Concepts/Existing Conditions Report

Introduction:

S&T Bank Corporate Headquarters located in Indiana PA is a 4-story, aesthetically pleasing building. On the first floor of the building is a branch bank. The rest of this floor along with floors 2, 3 and 4 is designated for office space and large lobby areas for dealing with customers. The exterior façade of the building complements its surroundings very well. The building is clad with a cranberry velour brick with large spandrel glass sections and is accented perfectly with a limestone colored concrete as an architectural feature.

In the following paragraphs, this report will completely describe the existing structural system from the foundation to the roof. After presenting some of the design considerations and providing the actual design loads, the report will show a lateral and gravity load analysis of the building. This analysis will design a beam, a column, the decking, and the joists. The product from this design will then be compared to the existing design. Finally a short conclusion summing up the report will be provided at the end.

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Building codes:

IBC 2003- International Building Code
ASCE 7-98
ACI 318-02

Overall Structural System

Foundation:

A geotechnical report provided by Triad Engineering Inc. establishes that the bearing capacity of the soil below the building can be no more than 6000psf. The foundation of the building rests on spread footings, which have a concrete strength of f'_c=4000psi at 28 days. The footings are as small as 5'x 5' and 1' thick or can be as large as 10'x10' and 2'-6" thick. A typical footing is 7'-6"x 7'6" and is 2' thick. Concrete piers are also used throughout the masonry wall; these piers are typically 1'-8" by 2'-4". Both spread footings and piers have reinforcing steel ranging from #5's to #9's. The columns are attached to the concrete with A36 steel base plates and anchor bolts. The basement exterior wall is a typical 12" Ivany block except for the section under the rotunda entrance, which is reinforced concrete. This wall extends to the 2nd floor and is then replaced by a curtain wall. Basement floor construction consists of a 4" concrete slab, reinforced with 6x6 w1.4 x w1.4 WWF on a 6mil vapor barrier placed on a minimum of 4" compacted stone.

Framing System:

S&T Bank Corporate Headquarters is a steel frame building. The steel frame is four stories high and has a typical layout on every floor. The story heights are 15′-4″, 13′-4″, 13′-4″ and 13′-4″ for the 1st through 4th floors respectively. The building footprint is 141 ft. in the North-South direction by 127 ft. in the East-West direction. In general, differences in floor framing layout consists of the sizes of the beams and the addition/subtraction of shafts which typically appear near the staircases. However in the S-E corner of the first floor only, there is a bank vault. On the floors above a 12′ x 12′ section is taken out of the S-E corner. There are 6 bays in both the N-S direction and the E-W direction. The first and the last bays in either direction are 12′ bays. Typically the central bays are 28′ wide, but can be as large as 30′ or as small as 16′. A general layout of a typical floor can be seen in Figure #1. All of the structural columns, beams, and girders are A992 steel and have a yield strength of 50ksi. The arrows near the columns designate moment connections that resist lateral load in the direction of the arrow. These moment connections are attached by "wind clips" which are angles welded to the top and bottom of the connection. Due to this connection, they can only partially resist lateral loads.

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Corporate Headquarters
Indiana, PA

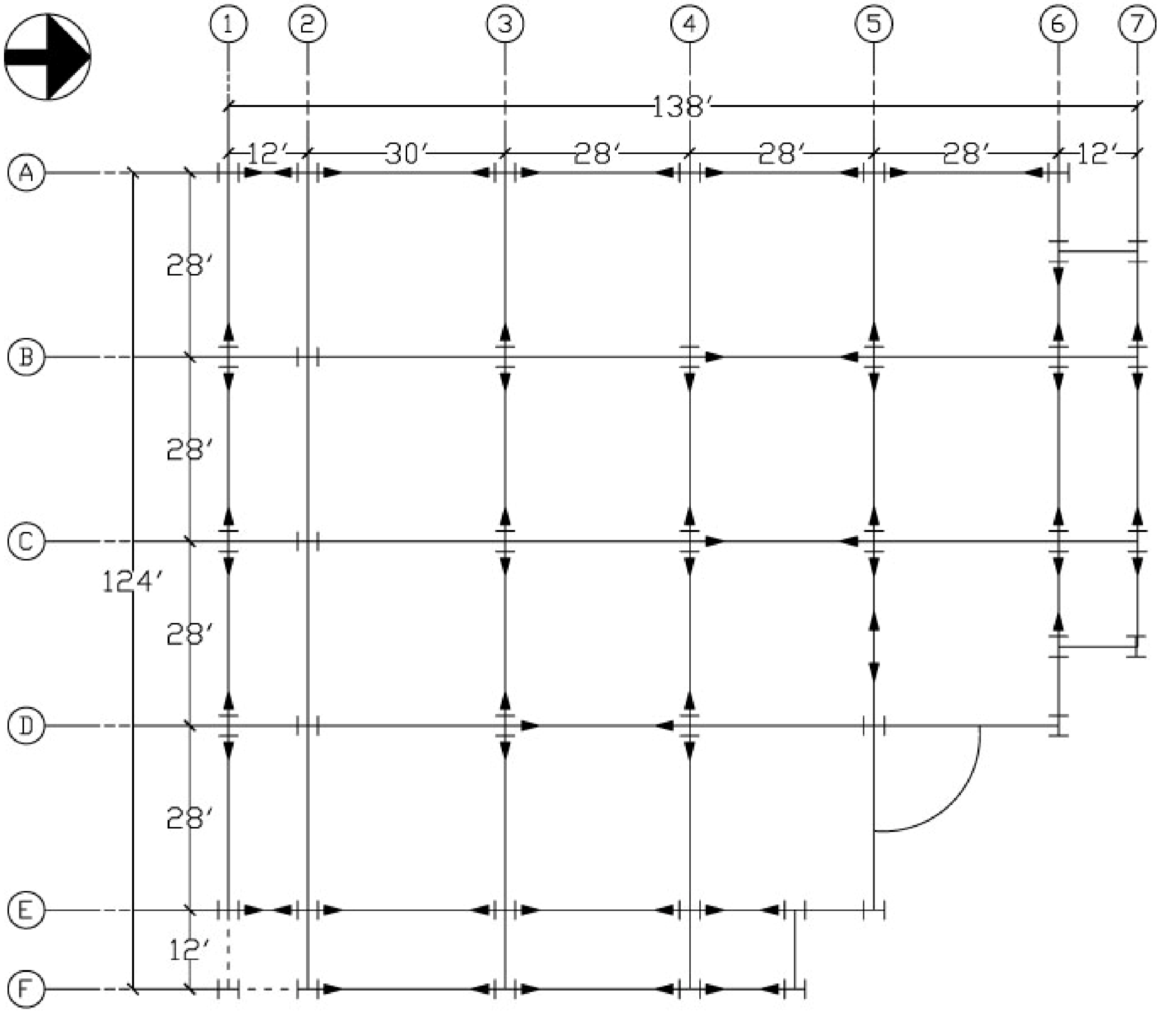

Figure #1: Typical Framing Layout

The building's columns range from W10x33 to W12x87, while a typical column size used is W12x53. As you can see in the figure above, every beam forms into a column. Therefore, beams and girders cannot be considered in their normal sense. Girders will be considered those running in the E-W direction and beams will be considered to be running in the N-S direction. Girder sizes range from W16x26 up to W24x76 with a typical

girder size of W24x55. Beams that run in the N-S direction are much smaller than the girders that frame in from the E-W direction. Beams running N-S range from W12x16 up to W16x26 with a typical beam size of W14x22. Any other structural components, such as angles or base plates, are A36 steel and have a yield strength f_y=36ksi.

All of the structures walls, girders, and beams run orthogonal except for a small section on the North-East corner of the building. This corner is rounded at the foundation with a 24′-0″ radius semi-circle (See Figure #2).

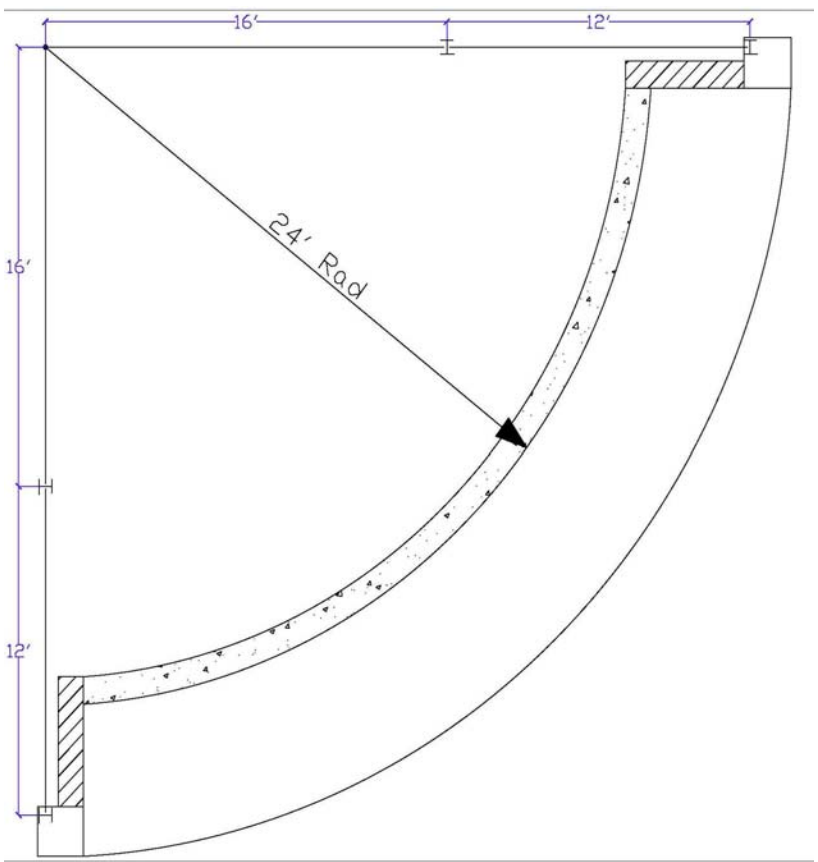

Figure #2: Non-typical Wall Detail

Decking System:

The deck system consists of a 3" normal weight concrete slab topping, reinforced with 6x6 W1.4 x W1.4 WWF on Bowman 28 Gage SF-1 galvanized deck. This non-composite decking is set on 24k4 joists that are spaced at 2' apart. The concrete topping is rated at 3000psi. The roof decking is relatively the same as the floors below except when placed under the AHU, the decking then sits on 24k6 joists. The depth of the Bowman deck with the concrete is 3". The joists are 24" deep which gives a total floor system depth of 27".

Design Considerations:

The building guides that were used during the development of the existing building are IBC 2003, in accordance with ASCE7-00. For this project, the building will be analyzed and designed with IBC 2003, in accordance with ASCE7-02. Also decking and joist catalogs will not be the same as those used by the design professional. Due to these changes, designs that are determined in this project may differ slightly from those in the actual structure.

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Indiana, PA

Design Loads:

Dead Loads

Superimposed DL: **15psf**

Calculated Structural Framing Loads

1st Floor:	6.42psf
2nd Floor:	6.42psf
3rd Floor:	6.42psf
4th Floor:	6.42psf
Roof:	3.69psf

Floor Loads: **40psf**
(Decking)

Live Loads

Floors 1, 2, 3, & 4 **100psf**
(Lobby area)

Snow Loads

P_f = 20psf

ASCE7-02 was used in accordance with IBc 2003

Wind load calculations are provided in Appendix A-1

Seismic load calculations are provided in Appendix A-2

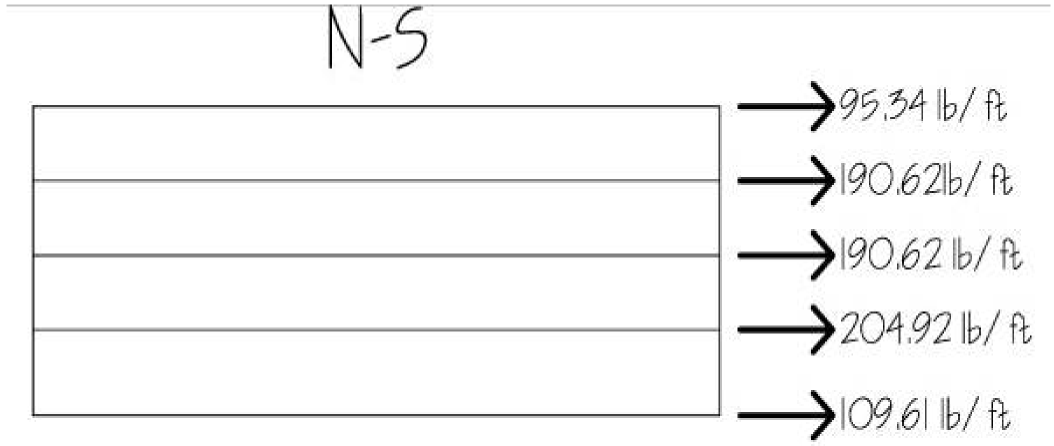

Figure #3: Distributed Wind Loads
(Simplified Analysis)

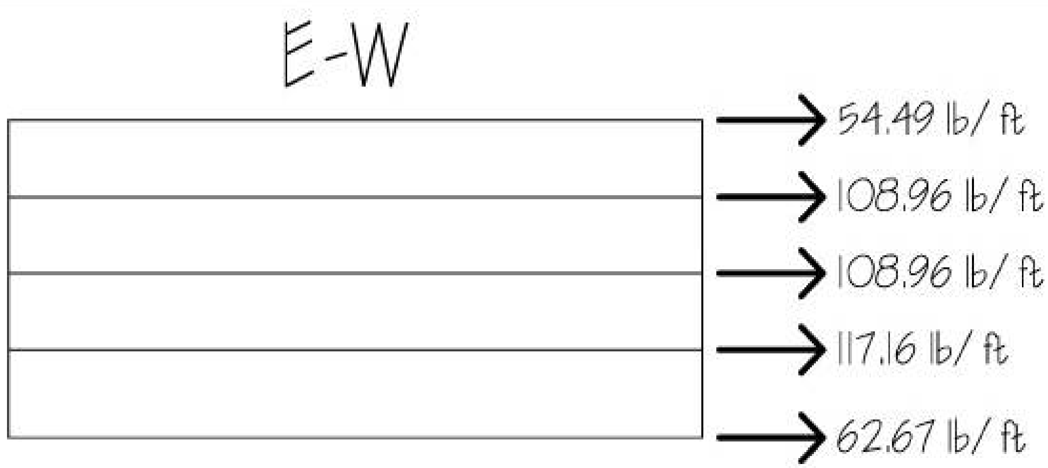

Figure #4: Concentrated Story Wind Loads
(Simplified Analysis)

Above in figure #3 is a representation of the distributed wind loads on S&T Bank. Since the overall building height is less than 60′, IBC2003 allows a simplified analysis of the wind loads that affect the building.

Figure #4 shows the story shears on each level. These story shears are dependant on the wind pressure and each of the story heights.

Also using IBC2003 in accordance with ASCE7-02, seismic loads were developed for the building. Below in Figure #5 is a representation of the story shears due to seismic loads. Since the story shears due to wind loading are more significant, they will be used when designing for lateral resistance.

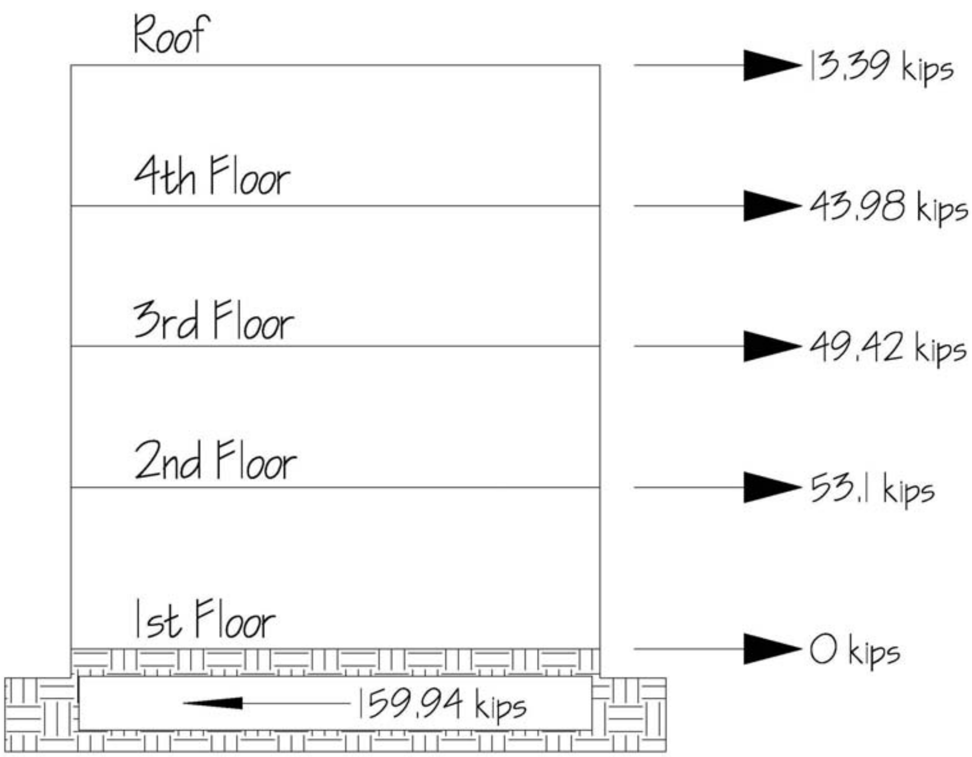

Figure #5: Seismic Story Shear Loads

Lateral & Gravity Analysis

Beam Design

A beam located on the 3rd floor was analyzed for this design procedure (See Figure #6). The beam has a span of 28 ft. and is located in an interior bay. Since the beam is part of the moment frame, the portal method was used to determine the lateral loads on the member. Combining the lateral loads with the gravity loads, the beam was subjected to a moment of 602.04 ft-kips.

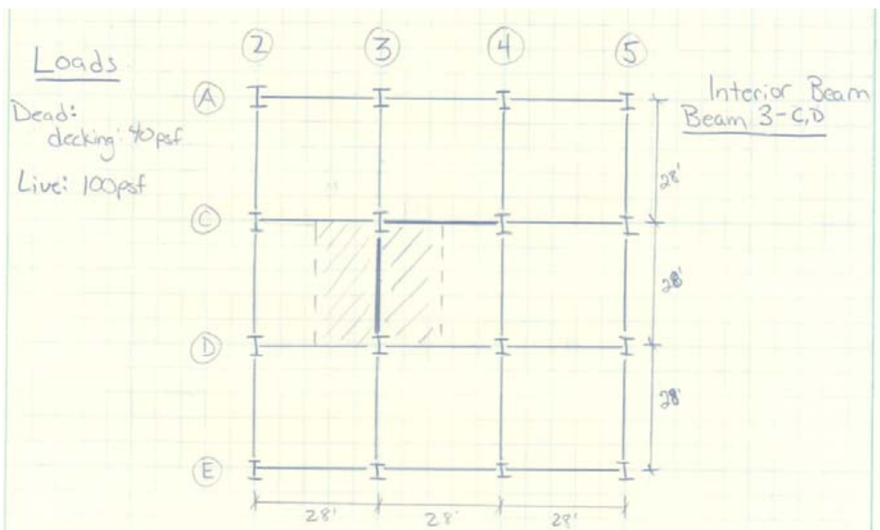

Figure #6: Typical Bay

According to the AISC Manual 3rd Edition Table 5-3, the most economical beam would be a W24x68 which has a ΦMp=664ft-kips. The beam actually used in this structure is a W24x68. Though the beams are identical, the moment calculated on the beam may differ slightly than that which was

found by the design professional. This could be due to the difference in load combinations and between ASD and LRFD.

Column Design

To determine a column size appropriate to resist axial loads from dead and live loads, and bending moments due to the controlling story shears, a spot check was performed on a column located in an interior bay. This column is designated as E-3, which can be referenced in Figure #1. By combining axial loads and bending moments, an effective load was determined to be 503.4 kips. Using Table 5 in the AISC Manual and entering the charts with 503.4 kips as a load and a KL=11, a W12x53 column is adequate to resist the loading. This is what was expected since the column used in the actual design was a W12x53.

Decking design

With a live load of 100psf, a superimposed dead load of 15psf, and the load combination 1.2D + 1.6L, a uniform load of 178psf was found on the decking. Using the USD Design Manual, 28 gage UFS decking for a single span spaced at 2'-0" is strong enough (216psf) to support the loading. This is what was expected since the actual design is 28 Gage Bowman SF-

1decking. The depth for both of these decking systems will be 3" total when the concrete is poured on top of the deck.

Joist Design

Using a live load of 100psf, a decking dead load of 40psf and a superimposed dead load of 15psf, the total distributed load that a joist spaced at 2'-0" is subjected to is 230plf. Using the Vulcraft Joist and Joist Girder Design Manual, a 22k4 joist spaced at 2'-0" is adequate and economical the loading. The joists used in the actual building are 22k2 joists spaced at 2'-0". The differences in design can most probably be accredited to the different design manuals used.

Conclusion

S&T Bank is a 4-story steel frame building. The foundation consists of spread footings to support the weight of the building. The framing of the building forms directly into the columns. The floor system is a form deck supported by joists that are 2'-0" on center. To resist lateral loads (from seismic or wind) many of the connections are moment connections. These moment connections however are fastened by "wind clips", which only resist partial lateral loads. During the analysis of the structure, as can be

seen in the appendices, the girder and column that was calculated came to be the same as the girder and column that is used in the structure, W24x68 and W12x53 respectively. The decking also came out as was expected, or at least close to the same. The existing deck is 28 Gage Bowman SF-1 and the calculated decking is 28 Gage UFS deck. Both of these decks are the same depth and thickness. However there were some slight discrepancies when designing the joists. Where the joist design originally called for a 22k2 joist @ 2'-0" on center, the calculated joist for a load of 210 and a span of 28' is a 22k4 joist (as provided in Vulcraft Joist Design).

Appendix A-1: Seismic Load Calculations

Seismic Use Group = I
Importance Factor = 1.0
Site Class C
S_s= 12.7
S_1= 5.4
F_a = 1.2
F_v = 1.7
$S_{ms} = F_a*S_s$ = 15.24%
$S_{m1} = F_v*S_1$ = 9.18%
$S_{ds} = (2/3)S_{ms}$ = 10.16%
$S_{D1} = (2/3)S_{m1}$ = 6.12%
S_a = 10.16; $T_o<T_a<T_s \rightarrow S_a=S_{ds}$
Design Category A: $S_{ds}<.167g$
R=4
Ω=3
C_d=3.5
$C_s= S_{ds}/(R/I)$ = 0.0254
W=6296.81 kips

$V=C_s*W$
=6296.81*0.0254=<u>159.94 kips</u>

Level	Height (ft)	Weight (kips)	C_v	F (kips)
Roof	55'-4"	720.71	0.0837	13.39
4th	42'-0"	1716.02	0.275	43.98
3rd	28'-8"	1930.04	0.309	49.42
2nd	15'-4"	1930.04	0.332	53.1
1st	0	1996.08	0	0

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Seismic Loads

Seismic use group = I
Importance = 1.0
S_S = 12.7
S_1 = 5.4
Site Class C

F_a = 1.2 F_v = 1.7

$S_{MS} = F_a S_S$
$= 1.2(12.7) = 15.24\%$

$S_{M1} = F_v S_1$
$= 1.7(5.4) = 9.18\%$

$S_{DS} = \frac{2}{3} S_{MS} = 10.16\%$

$S_{D1} = \frac{2}{3} S_{M1} = 6.12\%$

$S_a = 10.16$

$T_a = .1(\# \text{ stories}) = .1(4) = .4$ (approximate)

$T_0 = \frac{.2(S_{D1})}{(S_{DS})} = 0.121$

$T_s = \frac{S_{D1}}{S_{DS}} = \frac{6.21}{10.16} = .611$

$T_0 < T_a < T_s \quad \therefore S_a = S_{DS}$

Design Category A : $S_{DS} < .167g$
Ordinary Steel moment frames
$R^a = 4$ $C_d^b = 3.5$
$\Omega_0^g = 3$

$C_s = \frac{S_{DS}}{R/I} = \frac{.1016}{4/1.0} = 0.0254$

$$\boxed{\begin{array}{c} V = C_s W \\ 6296.81 \times 0.0254 = 159.94^k \end{array}}$$

Base Shear = 124.39^k

$T_a = 0.1N = .1 \times 4$ stories
$T_a = .4 \rightarrow k = 1$

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Weight @ floors w/ cladding | Seismic Loads

Roof: $696.59^k + [2(3' \times 141 \times 15psf) + 2(3' \times 127' \times 15psf)] = \underline{720.71^k}$

4^{th}: $720.71 + 601.28 + 1007.57 + [2(13.33' \times 141' \times 15) + 2(13.33' \times 127' \times 15)]$
individual floor weight: 1716.02 $\quad = \underline{2436.73^k}$

3^{rd}: $2436.73 + 636.68 + 1007.57 + [2(13.33' \times 141 \times 40) + 2(13.33 \times 127' \times 40)]$
Individual floor weight: 1930.04 $\quad = \underline{4366.77}$

2^{nd}: $4366.77 + 636.68 + 1007.57 + [2(13.33' \times 141 \times 40) + 2(13.33' \times 127' \times 40)]$
Individual floor weight = 1930.04 $\quad = \underline{6296.81^k}$

1^{st}: $6296.81 + 659.84 + 1007.57 + [2(15.33 \times 141 \times 40) + 2(15.33 \times 127 \times 40)]$
Individual floor weight = 1996.08 $\quad = \underline{8292.89^k}$

$F_x = C_{vx} V \qquad K = 1$

$$C_{vx} = \frac{w_x h_x^k}{\sum w_i h_i^k}$$

$w_r h_r^k = (720.71)(9.67) = 6969.27$
$w_4 h_4^k = (1716.02)(13.33') = 22874.55$
$w_3 h_3^k = (1930.04)(13.33') = 25727.43$
$w_2 h_2^k = (1930.04)(14.33') = 27657.47$

$\sum w_i h_i^k = 83228.72$

$C_{vr} = .0837$
$C_{v4} = 0.275$
$C_{v3} = 0.309$
$C_{v2} = 0.332$
$C_{v1} = 0$

$$\boxed{\begin{aligned} F_r &= 0.0837 \times 159.94^k = 13.39^k \\ F_4 &= 0.275 \times 159.94^k = 43.98^k \\ F_3 &= 0.309 \times 159.94^k = 49.42^k \\ F_2 &= 0.332 \times 159.94^k = 53.1^k \\ F_1 &= 0 \end{aligned}}$$

Appendix A-2: Wind Load Calculations

Height of Building < 60' → *can use simplified wind load method (IBC 2003)*

V= 90mph
I_w= 1.0
Exposure Category B
λ= 1.22
P_{s30}= -6.7 psf

$P_s = \lambda * I_w * P_{s30}$ = 1.22*1.0*(-6.7)

P_s = -8.174 psf → Pressure from wind is leeward side only.

Wind Load Analysis | S&T Bank

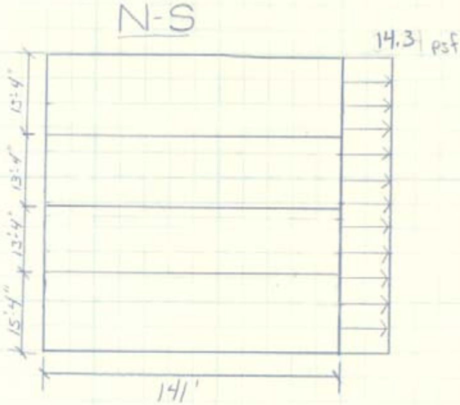
$h \le 60'$ → use simplified wind load method

$V = 90$ mph (40 m/s)
$I_w = 1.0$
Exposure Category B
$\lambda = 1.22$
$p_{s30} = -6.7$ psf

$p_s = \lambda I_w p_{s30}$
$= 1.22(1.0)(-6.7)$
$p_s = -8.174$ psf

Must check wind loads @ each surface (IBC2008-1609.6.2.1)

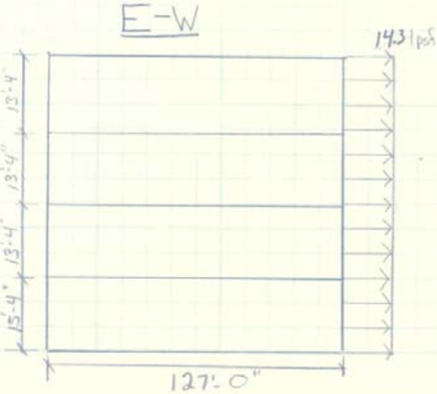
N-S



Zone	Adjusted Pressure P_s
A	14.3 ←
B	-7.6
C	9.4
D	-4.1
E	-16.5
F	-9.7
G	-11.1
H	-6.9

Zone A is the significant wall loading

E-W



Wind Load Analysis | S&T Bank

N-S

Top = 6.667 × 14.3 psf = 95.34 lb/

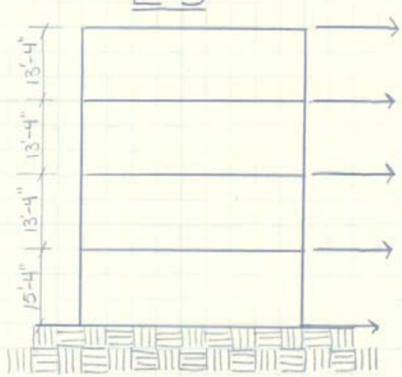
4th = 13.33' × 14.3 psf = 190.62 lb/ft

3rd = 13.33' × 14.3 psf = 190.62 lb/ft

2nd = 14.33' × 14.3 psf = 204.92 lb/ft

1st = 7.665' × 14.3 psf = 109.61 lb/ft
(to footer)

E-S



Top = 6.667' 14.3 psf = 95.34 lb/ft

4th = 13.33' × 14.3 psf = 190.62 lb/ft

3rd = 13.33' × 14.3 psf = 190.62 lb/ft

2nd = 14.33' × 14.3 psf = 204.92 lb/ft

1st = 7.665' × 14.3 psf = 109.61 lb/ft
(to footer)

* Simplified Analysis method Assumes Building is approximately Symetric.

Appendix A-3: Snow Load Calculations

Snow Loading | S&T Bank

P_g = 30 lbs/sf Fig 1608.2

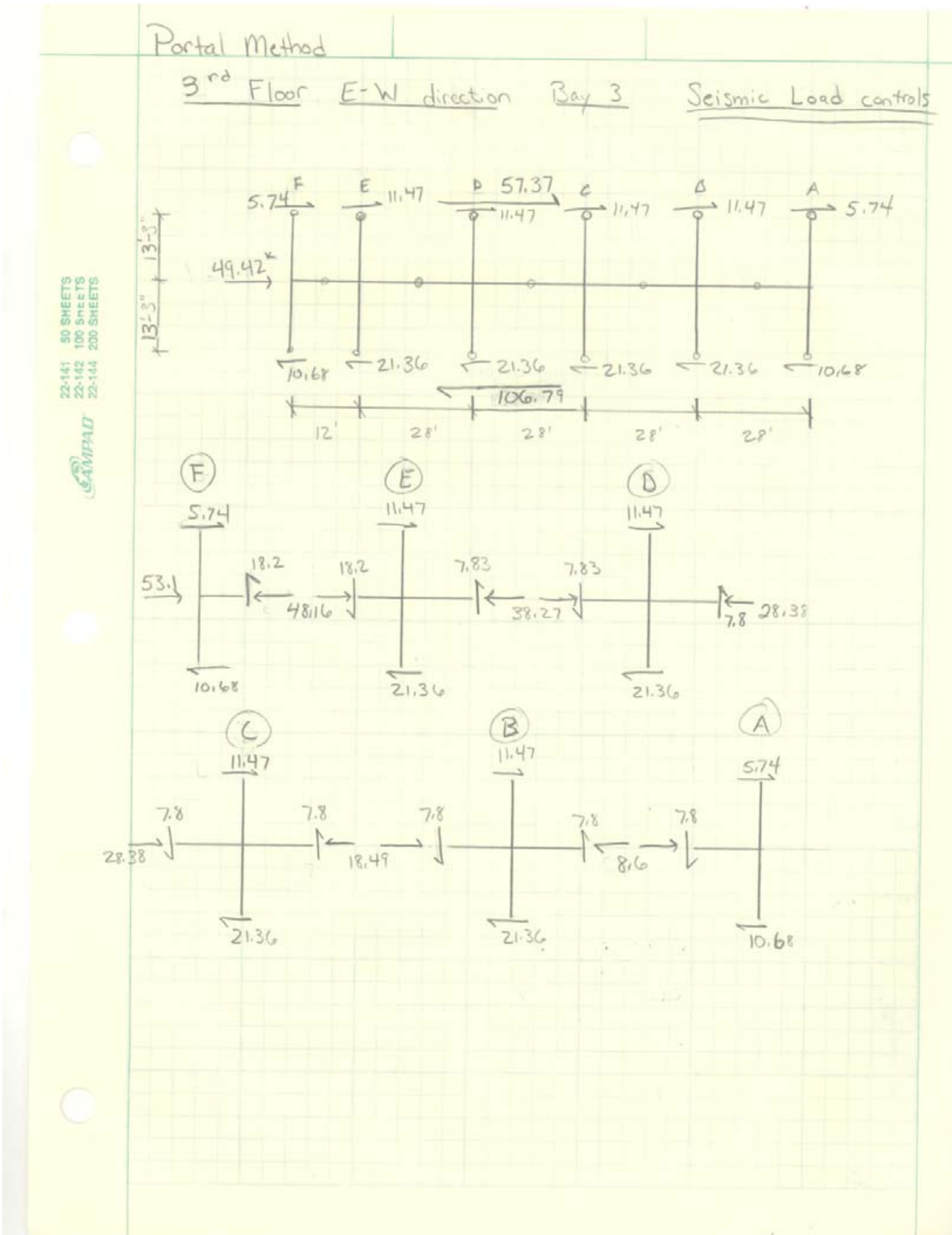
Terrain Category B
C_e = 0.9 Fig 1608.3.1
C_t = 1.0 Fig 1608.3.2
I_s = 1.0

$P_f = 0.7 C_e C_t I P_g$
$= 0.7(0.9)(1.0)(1.0)(30)$
$= 18.9$ psf

but since $P_g > 20$ psf ; $P_f = 20(I) = 20(1.0) = 20$psf

$\boxed{P_f = 20\text{psf}}$ ASCE 7-02

Appendix A-4: Portal Analysis



Appendix A-5: Spot Calculations

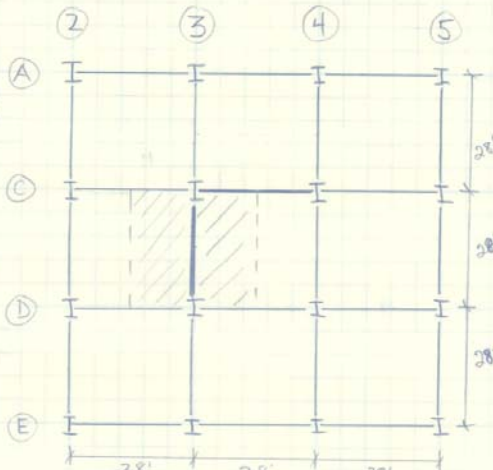
Beams

Spot check — Typical Bay

Loads

Dead:
decking: 40psf

Live: 100psf

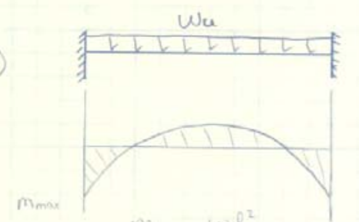


Interior Beam
Beam 3-C,D

Using $1.2D + 1.6L$

$w_u = 1.2(40 \times 28) + 1.6(100 \times 28)$

$= \underline{5.824 \text{ k/ft}}$



$M_{max} = \frac{w\ell^2}{12}$

$M_u = \frac{5.824(28)^2}{12}$

$= \underbrace{380.5'k}_{\text{gravity}} + \underbrace{7.8(28')}_{\text{lateral}} = 602.04'^{k}$

full lateral support w/ deck

$\phi M_{px} = 664'^{k}$ for W24x68
Z_x table

Beam 3-C,D use W24x68

Check self wt

$w_u = 5.824 + .048 = 5.872$

$M_u = \frac{5.872(28)^2}{12}$

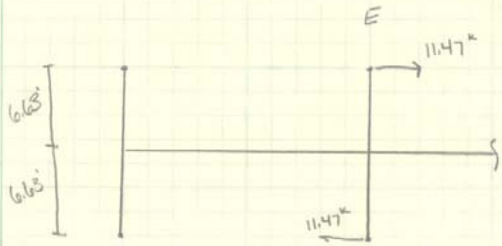
$= 383.64'^{k} + 218.4'^{k} < \phi M_{px}$

$\therefore$ use W24x68

The Structure uses W24x68. Differences in actual loads may occur between ASD + LRFD

Columns

Spot check | Column



Moment on Column E

$M = 11.47^K (6.63')$

$M = 76.1'^K$

Gravity Check

$A_T = \frac{(28'+28')(12'+28')}{2}$ (1 story) $= 1120\ ft^2$

$K_{LL} = 4$ column

$A_I = 1120(4) = 4480\ ft^2$

Live Load Reduction

$L = L_o\left(.25 + \frac{15}{\sqrt{A_I}}\right)$

$L = .474L_o > .4$ use $.474L_o$

$L = 100(.474) = 47.4$ psf

above 3rd floor
$DL = 80$ (decking) $+ 15$ (super imp) $+ 10.11$ (framing load above)

$P = 1.4D = 1.4(105.11) = 147.154$

$P = 1.2D + 1.6L + .5S$

$= 1.2(105.11) + 1.6(47.4) + .5(20)$
$= 211.972$ ← controls

$P_u = PA_T = 211.972(1120)$

$P_u = 237.4^K$ Guess W12x?

$\alpha = 24/d = 24/12 = 2$

$M_u = \frac{wL^2}{11} = 61.4'^K$

$M_{tot} = 61.4 + 76.1 = 133$

$P_{eff} = P_u + \alpha M_u$

$237.4 + 2(133) = 503.4^K$

Assume $KL = 10.33' \rightarrow$ use $11'$

$W12x53 = \phi P_n = 539^K > 503.4^K$

use: W12x53

Is a typical column used!

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JOIST

Spot Check | Joists

LL = 100 psf

DL = 40 psf + 15 psf = 55 psf

w = 155 psf

Spaced @ 2' w = 310 plf

w/ span = 28' use 22K4 joists

Allowable load = 230 plf

Joists used in actual building are 22K4 @ 2'-0"

Differences in design could be credited to different design manuals.

Decking

Spot check Decking

Decking Design using USD design manual

LL = 100 psf
Superimposed DL = 15 psf

$1.2D + 1.6L = 1.2(15) + 1.6(100)$
uniform Load = 178 psf

28 gage UFS Decking; single span is strong enough
Total Load = 216 psf
Service Loading that produces l/180 Deflection
is only 131 psf < 115 psf

∴ Deflection criteria is ok ✓

* Use 28 gage UFS Decking; single span spaced @ 2'-0" for decking in Bay 3-4, C-D.

* Design of ACTUAL Decking is Bowman 28 gage SF-1

Appendix A-6: Building Weight Calculations

Building Weight | Seismic Loads

Beams/Girders

*Assume Floors are exactly same as 2nd floor frame.

W24x68: 12 @ 28' x 3 floors = 68.54 kips
W24x55: 14 @ 28' x 3 floors = 64.68 kips
W18x35: 7 @ 28' x 3 floors = 20.58 kips
W16x26: 9 @ 28' x 3 floors = 19.66 kips
W14x22: 17 @ 18' x 3 floors = 20.20 kips
W12x16: 15 @ 12' x 3 floors = 8.64 kips

$\boxed{W_{beam/Girder} = 202.3 \text{ kips}}$

Decking:

*assume 40 psf for decking

1st = 16,496 SF x 40 = 659.84
2nd = 15,917 SF x 40 = 636.68
3rd = 15,917 SF x 40 = 636.68
4th = 15,032 SF x 40 = 601.28

$\boxed{W_{decking} = 1874.64 \text{ Kips}}$

$\boxed{W_{total} = 4,897.35 \text{ kips}}$ + Cladding = 40 psf 1st to 4th
15 psf 4th to roof

Roof DL:

W21x50: 12@ 28' = 16.80
W18x35: 3@ 28' = 2.94
W16x31: 16@ 28' = 13.88
W14x22: 14@ 28' = 8.63
W12x16: 17@ 16' = 4.35
W14x41: 4@ 28' = 4.59
W10x22: 15@ 12' = 3.96

2 AHU @ 22 Kips = 44 Kips

Decking

14,936 sf x 40 psf

597.44 kips

$\boxed{W_{roof} = 696.59 \text{ Kip}}$ = 46.64 psf

Doesn't include 1st floor

Building Weight | Seismic Loads

Masonry: 476ft 15'-4" high 12" thick

assume 125 lb/ft³

$476' \times 1' \times 15.33' = 7297.1\ ft^3$
$\times\ 125.0\ lb/ft^3$

$\boxed{\text{Weight masonry} = 912.2\ kips}$

Elevator Shaft / Stairwell shaft:

Shaft masonry weight: $202' \times 8/12 \times 60'$ high

$\boxed{W = 1{,}010\ kips}$

Concrete entrance Weight:

$37.7' \times 12" \times 15.33' \times 150\ pcf = 86.71\ kips$

$\boxed{W = 86.71\ kips}$

Columns:

W10x33: $9 \times 58.67' \times 33\ lb/ft = 17.43\ kip$
W10x49: $11 \times 58.67' \times 49\ lb/ft = 31.62\ kip$
W10x54: $3 \times 58.67' \times 54\ lb/ft = 9.41\ kip$
W12x40: $9 \times 58.67' \times 40\ lb/ft = 21.12\ kip$
W12x79: $5 \times 58.67' \times 79\ lb/ft = 23.17\ kip$
W12x65: $3 \times 58.67' \times 65\ lb/ft = 11.44\ kip$

$\boxed{W_{column} = 114.19\ kips}$