Robert M. Arnold Public Health Sciences Building
The Fred Hutchinson Cancer Research Center
Seattle, Washington

Technical Report 1

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Architectural Engineering
Structural

executive summary

The Robert M. Arnold Public Health Sciences Building was constructed on the campus of the Fred Hutchinson Cancer Research Center (FHCRC). The Public Health Sciences Building houses four programs: Epidemiology, Cancer Biology, Biostatistics & Mathematics, and Cancer Prevention. This purpose of this report is to provide and introduction and initial investigation of the structural system used for Arnold building. Included in the report are detailed descriptions of the various elements which make up the structural system of the building. There are also spot check calculations of gravity members and one lateral force resisting member. The assumptions made in these analyses may differ from those made by the hired professionals.

The structure of Robert M. Arnold Building has various different elements. The floor system is compose primarily of two way slabs. These slabs transfer the load to what are typically concrete columns. At the base of the columns the loads are then transferred to spread footings. Lateral loads are resisted by a combined system of shear walls and braced frames.

Code Requirements:

The Robert M. Arnold Building was designed and completed prior to the City of Seattle adopting the International Building Code (IBC). The applicable building code at that time was the 1997 Uniform Building Code (UBC) as amended by the Department of Planning and Development. The design of concrete structures shall also be in accordance with standards set forth by the American Concrete Institution (ACI). The Seattle Building Code is comprised of the 1997 Uniform building code and the amendments made by the City of Seattle.

Load Calculations:

All loads shall meet the minimum design loads specified by The 1997 UBC.

Gravity Loads:

All loads shall conform to standards specified in the 1997 UBC. Live load are taken from Table 16-A, which is a table that was amended by Seattle.

Live Loads

Description	Uniform Load (lb/ft^2)	
	Code	Structural Drawings
Floor		
Offices	50	80
Levels 1—4 (Office)	50	75
Laboratories	-	100
Interstitial	-	25
Corridors	100	100
Parking	50	50
Sidewalks & Driveways	250	250
Partition Load	20	20
Roof		
Roof	25	25

Table 1-1

Live Loads

Table 1-1 shows the live loads as obtained from the code and also those obtained from the structural drawings. Certain loads are not specified by the Seattle Building Code and do not fall into a broader category. The loads listed on the structural drawings in some areas differ from the code. For the purpose of analysis the Live loads determined by the design professionals will be used. The structural engineers had more information regarding building occupancy, building equipment, and building use. The office live load takes into account the additional loads of filing systems. In accordance with the Seattle Building Code reduction of live loads are permitted, however, the structural engineers have specified that there will be no live load reduction for the first level through the fourth level.

Dead Loads

As specified by the Seattle Building Code, the dead loads are considered to be, "the weight of all materials and fixed equipment incorporated into the structure." Unlike the live loads, there is no table or reference specified by the code. Where necessary minimum design dead loads from ASCE 7-05(Appendix 2) will be used.

Dead Loads

Description		
Materials		
Steel		In superimposed dead load
Concrete	150 lb/ft^3	

Table 1-2

Snow Loads

The Uniform Building Code does not specify a method for determining snow loads. The division concerning snow loads states only that snow loads in excess of 20 lb/ft^2 may use a reduction factor. For this initial investigation the snow load will be taken to be 20 lb/ft^2.

Lateral Loads:

Wind Loads

The wind loads were calculated in accordance to the methods determined by the Seattle building Codes. For the purpose of this report calculations of wind pressures were completed through the use of tables and queries in a Microsoft access database. A report of these calculations maybe be found in appendix 1.

Design Wind Pressures (lb/ft^2)
Numbers correspond to pressure calculations found in appendix 1

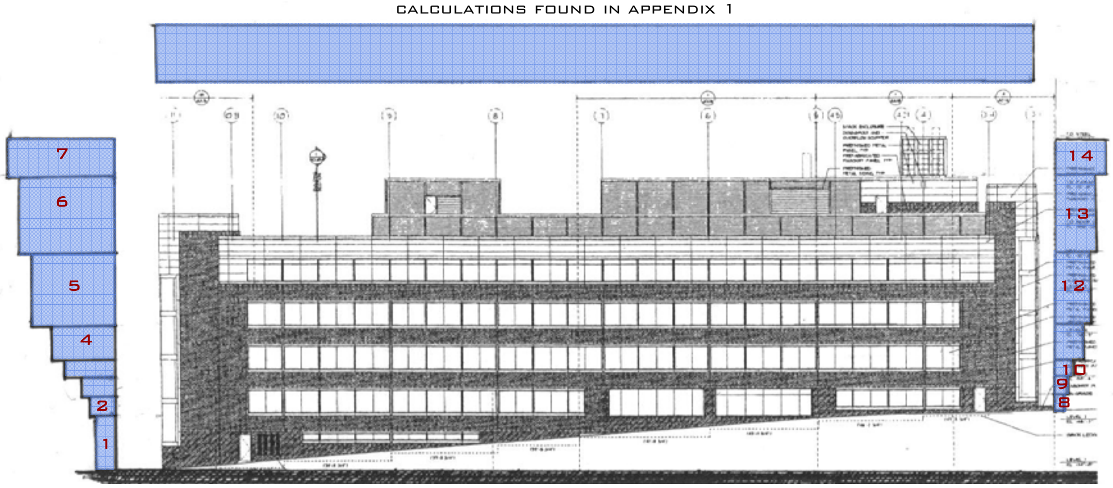
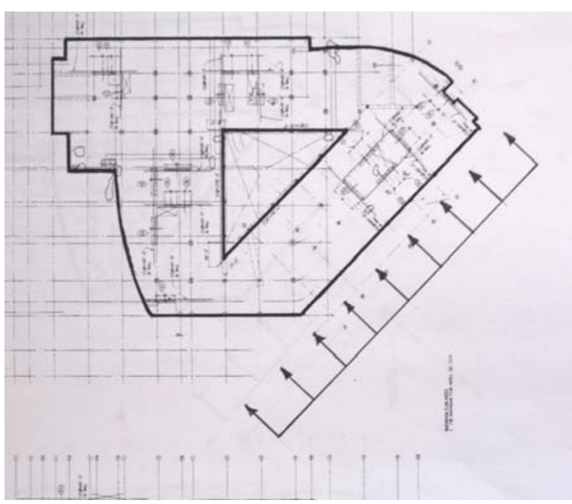
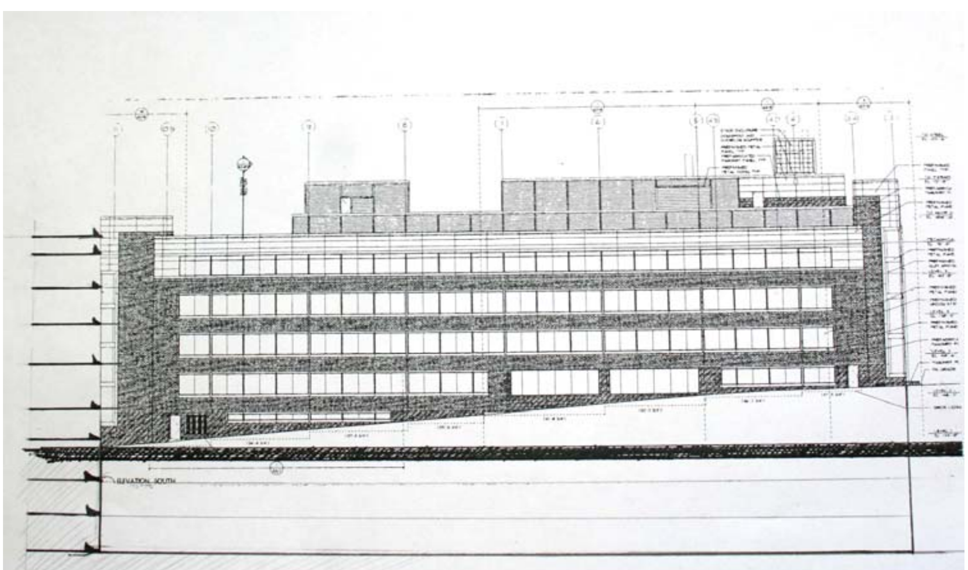


Figure 1-1



Seismic Loads

The seismic loads were calculated using one of the methods determined by the Seattle Building Code. The Static Force Procedure was used in calculating the base shear of Arnold building. Assumptions regarding soil conditions were taken from the drawings since the soils report is currently being retrieved. The weight of the building was approximated using data from Level D. The self weight of the structure of level D was calculated using the cubic weight of concrete and the size of each member. Appendix three is a summary of the weight of the concrete members. This weight was then divided by the square footage of the floor to provide the approximation, in lbs/ft^2, which was used in calculating the weight of the structure. The building base shear was calculated to be 5900 kips and the base shear listed on the drawings is listed as 5980 kips. The difference is probably due to the approximation made in calculating building self weight.



Structural System:

The Robert M. Arnold Public Health Sciences Building is an interesting collage of structural systems. Different portions of this building employ different methods of supporting the necessary loads. The building itself consists of five stories above grade plus a mechanical "penthouse" on the roof, while also extending 3 stories below grade. The triangular transfer of load around the atrium provides an element of structural complexity unseen in rectilinear buildings. Arnold Building houses the Public Health Science Department of the Fred Hutchinson Cancer Research Center. FHCRC specified that the building a standard of structural integrity higher than that of the code.

Foundation

The foundation of the Public Health Sciences Building consists mainly of spread footings and wall footings. Where the foundation is required to resist lateral loads carried down by shear walls, Arnold Building uses deeper drilled piers. The average footing is about 12 feet square, however, they could be various sizes ranging from eight feet square to 28 feet by 24 feet. The depth ranges from 30 inches to 48 inches deep, but is typically around 40 inches deep.

Framing

The framing of Arnold building is mainly composed of concrete structure, however, there are some portions of the building where steel has been used. Steel framing was used for the stairs and skylight in the atrium. A special stipulation was made that the structure of the atrium be design such that it would not cause any torsional load on the rest of the building. The columns on the fifth story are made of tube steel with the typical size being TS 12x12x$^{5}/_{8}$. Steel was also employed in the design of the roof structure that houses the building's mechanical equipment. The typical steel column in this area is a TS 4x4x4$^{1}/_{4}$. The irregularity of the steel roof structure lend itself to atypical beam and girder sizes. They range from W 10x12 to w 30x132. There also are a few steel columns in the main structure.

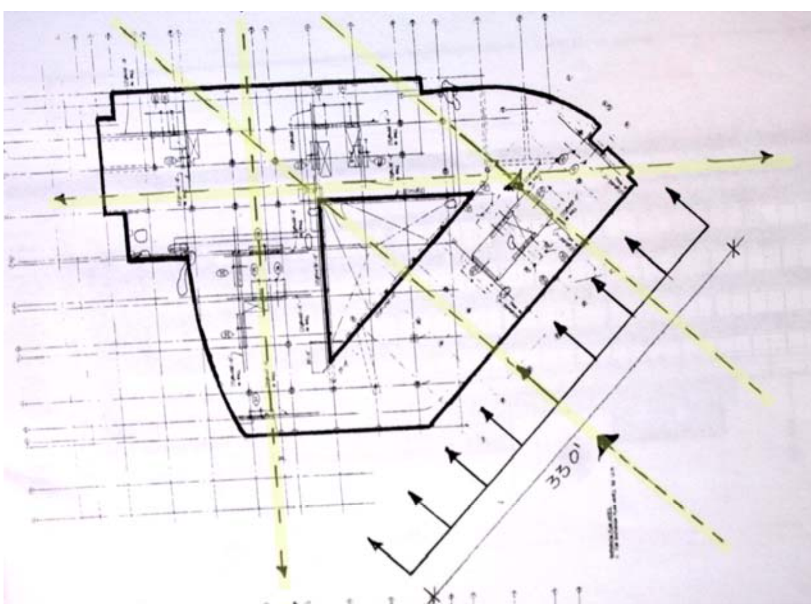
Almost all of the remaining portions of the structure are made of concrete. The columns are continuous cast in place reinforced concrete columns. The typical columns are 24 inches square and are on an average grid of 30 feet by 30 feet. The columns do not taper towards the top, however, the amount of reinforcement can vary. The shape of some columns varies. On certain floors columns may had a diameter of 24 inches instead of a width of 24 inches. Supporting Campus Drive, the turnaround, and the entrance plaza, under which the building extends, is an area of the building which uses cast in place reinforced concrete. The average beam size is 24 inches wide by 30 inches deep.

Structural Slabs

The floor system of Arnold Building is mainly composed of post-tensioned concrete floor slabs. The slab in the basement is not post-tensioned but instead is made of fiber reinforced concrete. The portion of the building that is under the entrance plaza uses reinforced concrete slabs. The roof slab is composed of reinforced concrete. With the noted exceptions the typical floor system is a flat post-tensioned concrete slab with column capitals.

Lateral Force Resisting System

For the purposes of this report it has been assumed that the lateral for is resisted solely by the shear wall and braced frames that are present in the structure. Located on the mechanical level is a lateral system of braced frames which transfer the load directly to the shear walls Further assumptions have been made in the analysis of lateral loads. The shear walls in planes parallel to the applied lateral are assumed to fully resist the load. The shear walls in planes that are not parallel are assumed to have an effective depth equivalent to that of a column and are consequently assumed to resist no portion of the lateral load. In further analysis of the lateral system these shear walls will be factored into the resistance of the resulting building torsion caused by the distance between the centroid of the applied load and that of the resisting force.



Structural Analysis;

In order to verify the assumptions made for this analysis spot checks of various structural elements of the building done. These May be found in appendix four.

APPENDIX 1

Wind Pressure Calculations

Exposure Category: B

Importance Factor: 1

Occupancy Category: I

Basic Wind Speed: 80

P=(Ce)(Cq)(qs)(I)(w)

1 M1 Windward Wall

Wind Stagnation Pressure (Ce) 16.4

Height 15 ft

Pressure Coefficient Cq =0.8

Exposure _Gust Factor Coefficient (Ce) 0.6

Design Wind Pressure 8.134

2 M1 Windward Wall

Wind Stagnation Pressure (Ce) 16.4

Height 20 ft

Pressure Coefficient Cq =0.8

Exposure _Gust Factor Coefficient (Ce) 0.7

Design Wind Pressure 8.790

3 M1 Windward Wall

Wind Stagnation Pressure (Ce) 16.4

Height 25 ft

Pressure Coefficient Cq =0.8

Exposure _Gust Factor Coefficient (Ce) 0.7

Design Wind Pressure 9.446

4 M1 Windward Wall

Wind Stagnation Pressure (Ce) 16.4

Height 30 ft

Pressure Coefficient Cq =0.8

Exposure _Gust Factor Coefficient (Ce) 0.8

Design Wind Pressure 9.971

5 M1 Windward Wall

Wind Stagnation Pressure (Ce) 16.4

Height 40 ft

Pressure Coefficient Cq =0.8

Exposure _Gust Factor Coefficient (Ce) 0.8

Design Wind Pressure 11.02

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6 M1 Windward Wall

Wind Stagnation Pressure (Ce) 16.4

Height 60 ft

Pressure Coefficient Cq =0.8

Exposure _Gust Factor Coefficient (Ce) 1

Design Wind Pressure 12.46

---

7 M1 Windward Wall

Wind Stagnation Pressure (Ce) 16.4

Height 80 ft

Pressure Coefficient Cq =0.8

Exposure _Gust Factor Coefficient (Ce) 1.0

Design Wind Pressure 13.64

---

8 M1 Leeward Wall

Wind Stagnation Pressure (Ce) 16.4

Height 15 ft

Pressure Coefficient Cq =0.5

Exposure _Gust Factor Coefficient (Ce) 0.6

Design Wind Pressure -5.08

---

9 M1 Leeward Wall

Wind Stagnation Pressure (Ce) 16.4

Height 20 ft

Pressure Coefficient Cq =0.5

Exposure _Gust Factor Coefficient (Ce) 0.7

Design Wind Pressure -5.49

---

10 M1 Leeward Wall

Wind Stagnation Pressure (Ce) 16.4

Height 25 ft

Pressure Coefficient Cq =0.5

Exposure _Gust Factor Coefficient (Ce) 0.7

Design Wind Pressure -5.90

---

11 M1 Leeward Wall

Wind Stagnation Pressure (Ce) 16.4

Height 30 ft

Pressure Coefficient Cq =0.5

Exposure _Gust Factor Coefficient (Ce) 0.8

Design Wind Pressure -6.23

---

12 M1 Leeward Wall

Wind Stagnation Pressure (Ce) 16.4

Height 40 ft

Pressure Coefficient Cq =0.5

Exposure _Gust Factor Coefficient (Ce) 0.8

Design Wind Pressure -6.89

---

13 M1 Leeward Wall

Wind Stagnation Pressure (Ce) 16.4

Height 60 ft

Pressure Coefficient Cq =0.5

Exposure _Gust Factor Coefficient (Ce) 1

Design Wind Pressure -7.79

---

14 M1 Leeward Wall

Wind Stagnation Pressure (Ce) 16.4

Height 80 ft

Pressure Coefficient Cq =0.5

Exposure _Gust Factor Coefficient (Ce) 1.0

Design Wind Pressure -8.53

---

15 M1 Roof

Wind Stagnation Pressure (Ce) 16.4

Height 25 ft

Pressure Coefficient Cq =0.7

Exposure _Gust Factor Coefficient (Ce) 0.7

Design Wind Pressure -8.27

---

16 M1 Roof

Wind Stagnation Pressure (Ce) 16.4

Height 60 ft

Pressure Coefficient Cq =0.7

Exposure _Gust Factor Coefficient (Ce) 1

Design Wind Pressure -10.9

---

17 M1 Roof

Wind Stagnation Pressure (Ce) 16.4

Height 80 ft

Pressure Coefficient Cq =0.7

Exposure _Gust Factor Coefficient (Ce) 1.0

Design Wind Pressure -11.9

TABLE C3-1 MINIMUM DESIGN DEAD LOADS*

Component	Load (psf)	Component	Load (psf)
CEILINGS		Decking, 2-in. wood (Douglas fir)	5
Acoustical Fiber Board	1	Decking, 3-in. wood (Douglas fir)	8
Gypsum board (per 1/8–in. thickness)	0.55	Fiberboard, 1/2-in.	0.75
Mechanical duct allowance	4	Gypsum sheathing, 1/2-in.	2
Plaster on tile or concrete	5	Insulation, roof boards (per inch thickness)	
Plaster on wood lath	8	Cellular glass	0.7
Suspended steel channel system	2	Fibrous glass	1.1
Suspended metal lath and cement plaster	15	Fiberboard	1.5
Suspended metal lath and gypsum plaster	10	Perlite	0.8
Wood furring suspension system	2.5	Polystyrene foam	0.2
COVERINGS, ROOF, AND WALL		Urethane foam with skin	0.5
Asbestos-cement shingles	4	Plywood (per 1/8-in. thickness)	0.4
Asphalt shingles	2	Rigid insulation, 1/2-in.	0.75
Cement tile	16	Skylight, metal frame, 3/8-in. wire glass	8
Clay tile (for mortar add 10 psf)		Slate, 3/16-in.	7
Book tile, 2-in.	12	Slate, 1/4-in.	10
Book tile, 3-in.	20	Waterproofing membranes:	
Ludowici	10	Bituminous, gravel-covered	5.5
Roman	12	Bituminous, smooth surface	1.5
Spanish	19	Liquid applied	1
Composition:		Single-ply, sheet	0.7
Three-ply ready roofing	1	Wood sheathing (per inch thickness)	3
Four-ply felt and gravel	5.5	Wood shingles	3
Five-ply felt and gravel	6	FLOOR FILL	
Copper or tin	1	Cinder concrete, per inch	9
Corrugated asbestos-cement roofing	4	Lightweight concrete, per inch	8
Deck, metal, 20 gage	2.5	Sand, per inch	8
Deck, metal, 18 gage	3	Stone concrete, per inch	12

*Weights of masonry include mortar but not plaster. For plaster, add 5 lb/ft^2 for each face plastered. Values given represent averages. In some cases there is a considerable range of weight for the same construction.

(*continued*)

TABLE C3-1 continued
MINIMUM DESIGN DEAD LOADS*

Component				Load (psf)
FLOORS AND FLOOR FINISHES				
Asphalt block (2-in.), 1/2-in. mortar				30
Cement finish (1-in.) on stone-concrete fill				32
Ceramic or quarry tile (3/4-in.) on 1/2-in. mortar bed				16
Ceramic or quarry tile (3/4-in.) on 1-in. mortar bed				23
Concrete fill finish (per inch thickness)				12
Hardwood flooring, 7/7-in.				4
Linoleum or asphalt tile, 1/4-in.				1
Marble and mortar on stone-concrete fill				33
Slate (per mm thickness)				15
Solid flat tile on 1-in. mortar base				23
Subflooring, 3/4-in.				3
Terrazzo (1-1/2-in.) directly on slab				19
Terrazzo (1-in.) on stone-concrete fill				32
Terrazzo (1-in.), 2-in. stone concrete				32
Wood block (3-in.) on mastic, no fill				10
Wood block (3-in.) on 1/2-in. mortar base				16
FLOORS, WOOD-JOIST (NO PLASTER)				
DOUBLE WOOD FLOOR				
Joint sizes (in.)	12-in. spacing (lb/ft^2)	16-in. spacing (lb/ft^2)	24-in. spacing (lb/ft^2)	
2 × 6	6	5	5	
2 × 8	6	6	5	
2 × 10	7	6	6	
2 × 12	8	7	6	
FRAME PARTITIONS				
Movable steel partitions				4
Wood or steel studs, 1/2-in. gypsum board each side				8
Wood studs, 2 × 4, unplastered				4
Wood studs, 2 × 4, plastered one side				12
Wood studs, 2 × 4, plastered two sides				20
FRAME WALLS				
Exterior stud walls:				
2 × 4 @ 16-in., 5/8-in. gypsum, insulated, 3/8-in. siding				11
2 × 6 @ 16-in., 5/8-in. gypsum, insulated, 3/8-in. siding				12
Exterior stud walls with brick veneer				48

Component						Load (psf)
Windows, glass, frame, and sash						8
Clay brick wythes:						
4 in.						39
8 in.						79
12 in.						115
16 in.						155
Hollow concrete masonry unit wythes:						
Wythe thickness (in inches)		4	6	8	10	12
Density of unit (105 pcf)						
No grout		22	24	31	37	43
48 in. o.c.			29	38	47	55
40 in. o.c.	grout		30	40	49	57
32 in. o.c.	spacing		32	42	52	61
24 in. o.c.			34	46	57	67
16 in. o.c.			40	53	66	79
Full grout			55	75	95	115
Density of unit (125 pcf)						
No grout		26	28	36	44	50
48 in. o.c.			33	44	54	62
40 in. o.c.	grout		34	45	56	65
32 in. o.c.	spacing		36	47	58	68
24 in. o.c.			39	51	63	75
16 in. o.c.			44	59	73	87
Full grout			59	81	102	123
Density of unit (135 pcf)						
No grout		29	30	39	47	54
48 in. o.c.			36	47	57	66
40 in. o.c.	grout		37	48	59	69
32 in. o.c.	spacing		38	50	62	72
24 in. o.c.			41	54	67	78
16 in. o.c.			46	61	76	90
Full grout			62	83	105	127
Solid concrete masonry unit wythes (incl. concrete brick):						
Wythe thickness (in mm)		4	6	8	10	12
Density of unit (105 pcf)		32	51	69	87	105
Density of unit (125 pcf)		38	60	81	102	124
Density of unit (135 pcf)		41	64	87	110	133

*Weights of masonry include mortar but not plaster. For plaster, add 5 lb/ft^2 for each face plastered. Values given represent averages. In some cases there is a considerable range of weight for the same construction.

(continued)

Appendix 3

Area	Level	wi	Expr1	Expr2
80365	Level F	10045625	1808.2125	0
96309	Level E	12038625	2166.9525	8848.389375
75604	Level D	9450500	1701.09	13892.235
66059	Level 1	8257375	1486.3275	16845.045
62713	Level 2	7839125	1411.0425	21753.571875
63404	Level 3	7925500	1426.59	27818.505
63404	Level 4	7925500	1426.59	33643.7475
35193	Level 5	4399125	791.8425	21907.6425

~~STAT~~

[Seismic Zone] (3)
Occupancy (3)

Static Lateral Force Procedure

$$V = \frac{C_v I}{R T} W$$

W = Total Seismic Dead Load
I = Importance Factor
C_v = Seismic Coefficient [Table 16-R]
T = Elastic Fundamental Period of Vibration. [Seconds]

R = Numerical Coefficient
Table 16N or 16-P

Building Frame System:
Shear Walls
Concrete $R = 5.5$
$\Omega_o = 2.8$
$H_{max} = 240$

Dual
Sh. Walls
Concrete w/ SMRF
$R = 8.5$ $\Omega 2.8$ H = N.L.

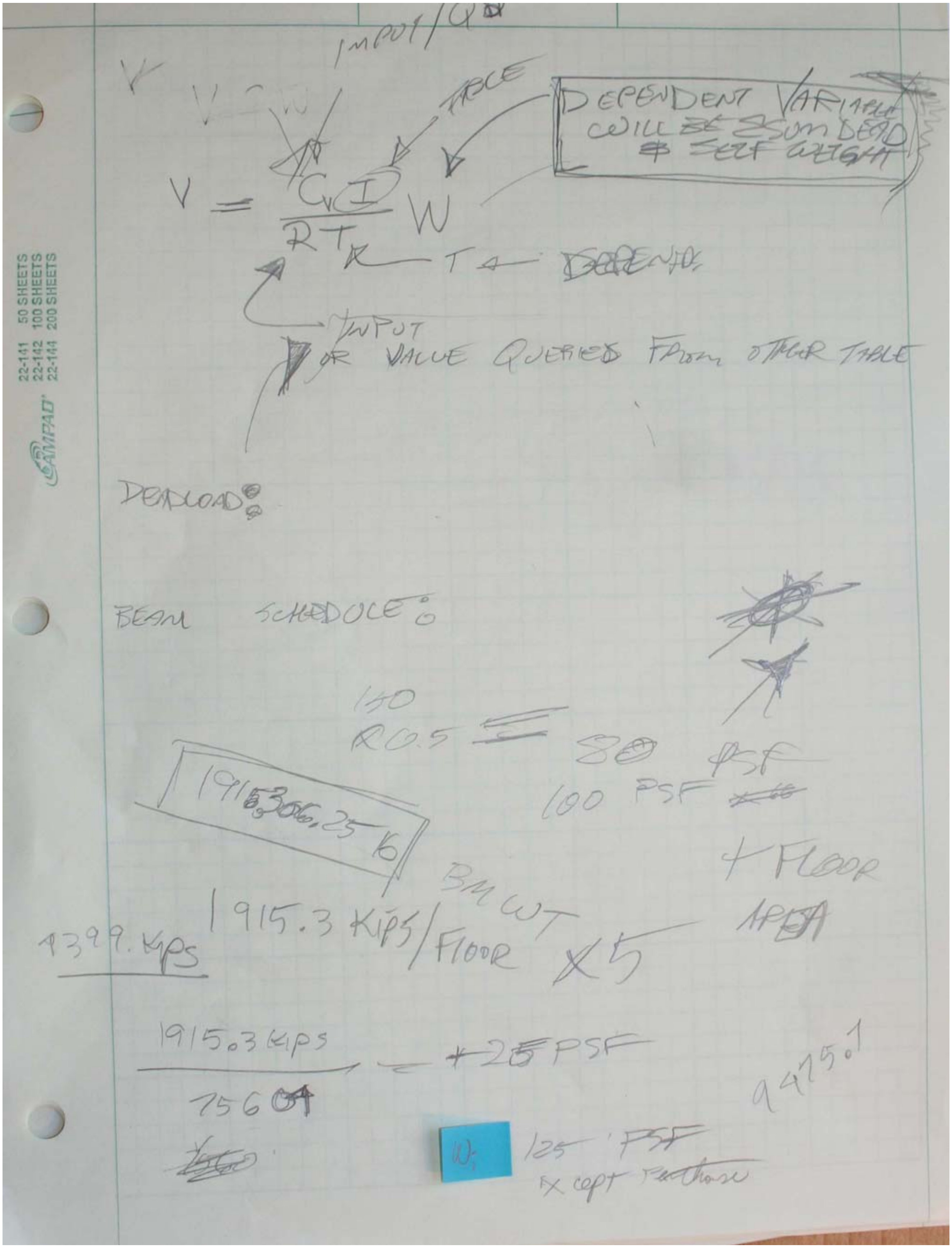
C_a = Seismic Coefficient
Tbl. 16-Q

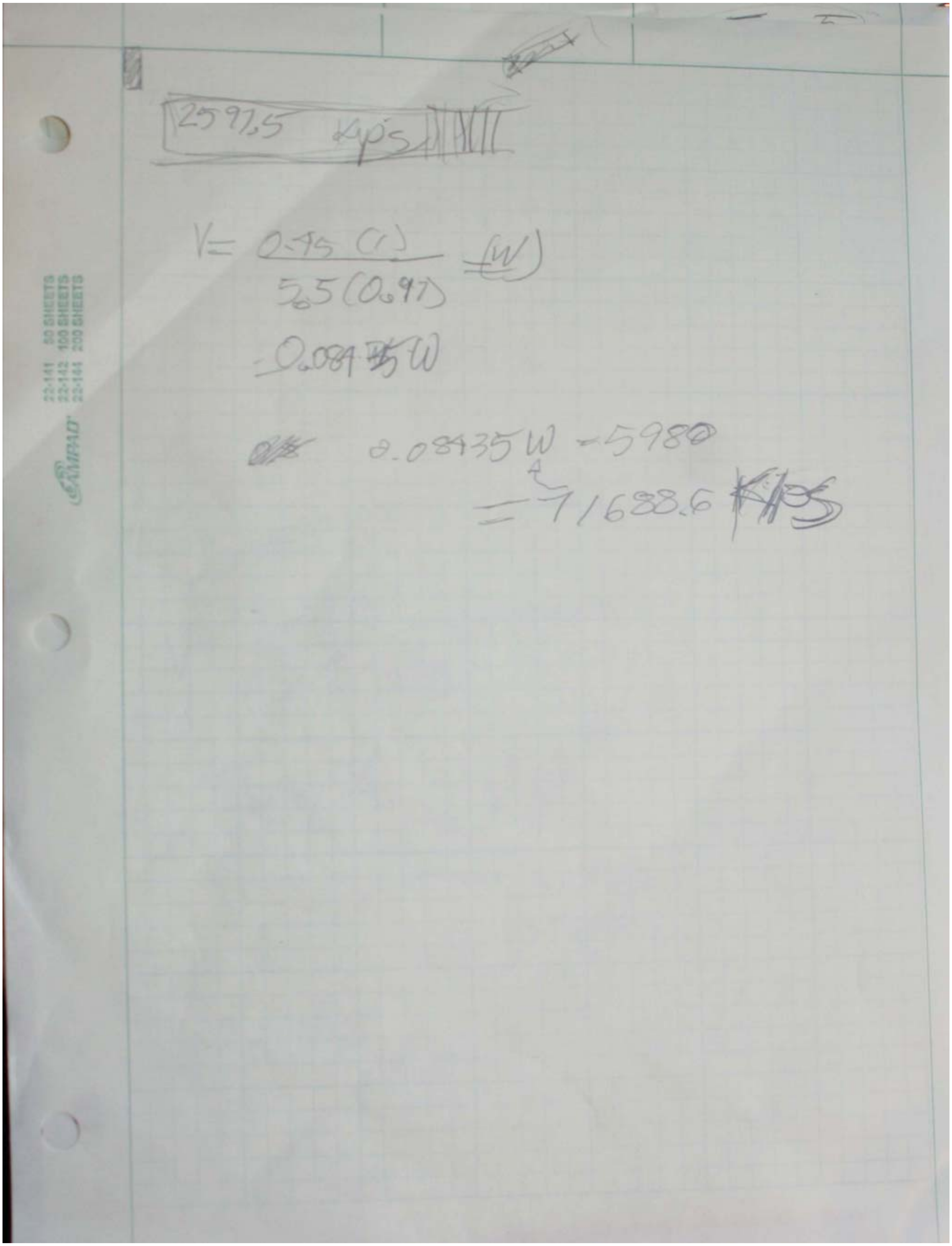
$\therefore$ Soil = S_C

Table 16-1 — Seismic
Z = 0.3

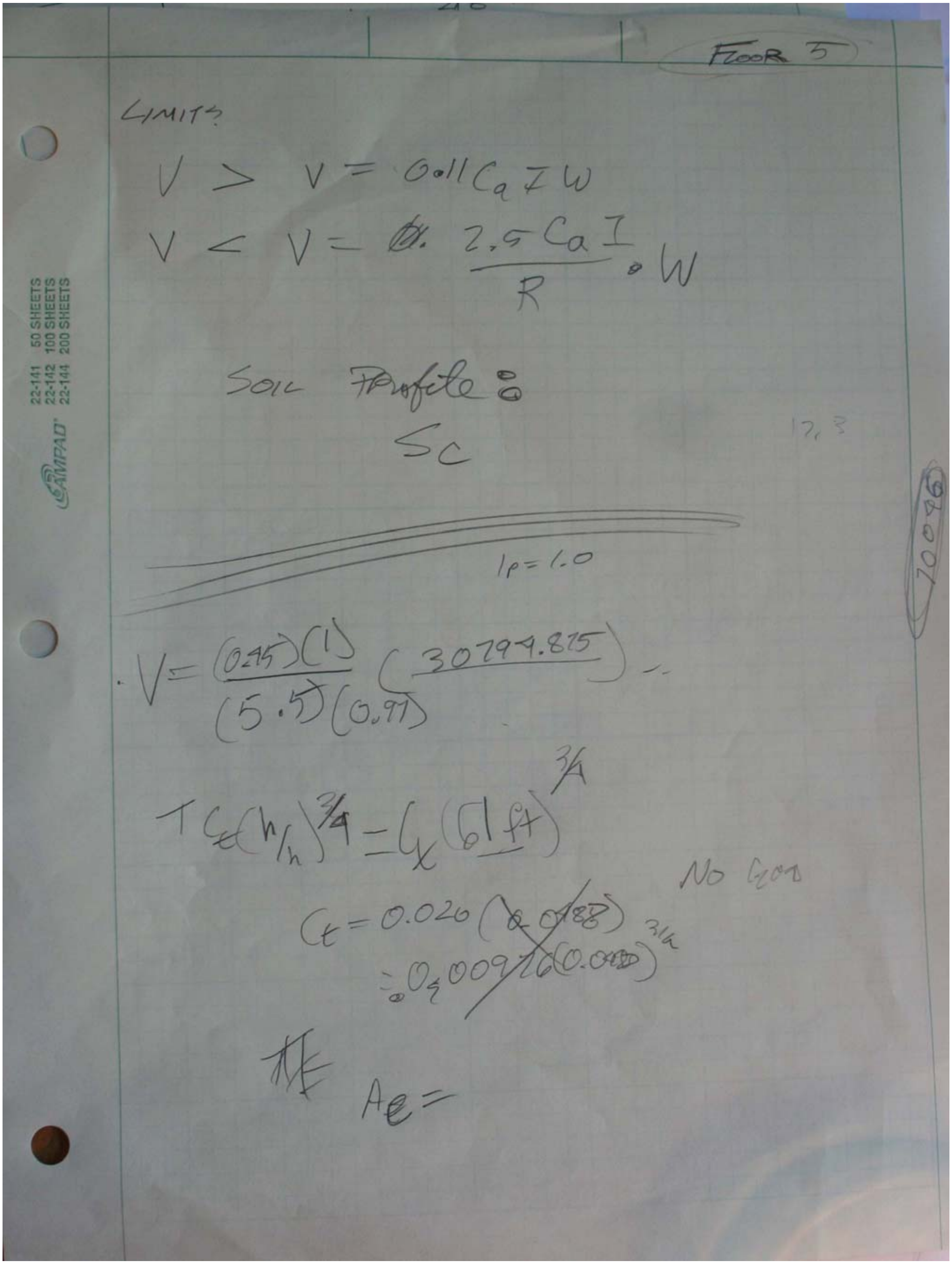
$C_a = 0.33$ ← Taken from Start Notes Page

Appendix 4


INPUT/Q
TABLE
DEPENDENT VARIABLE WILL BE ΣSUM DEAD & SELF WEIGHT
V = CvI/RTR W
T ← DEPENDS
INPUT OR VALUE QUERIED FROM OTHER TABLE
DEADLOAD:
BEAM SCHEDULE:
150
RC0.5
80 PSF
100 PSF
+ FLOOR AREA
BY WT
1915.3 KIPS/FLOOR
X 5
1915.3 KIPS
+25 PSF
125 PSF
Except Penthouse


2597.5 Kips
V = 0.45 (1) / 5.5 (0.97) (W)
= 0.08435 W
0.08435 W = 5980
= 71688.6 Kips

Appendix 4



FLOOR 5

LIMITS

$V > V = 0.11 C_a I W$

$V < V = \frac{2.5 C_a I}{R} \cdot W$

SOIL Profile: S_C

12.3

70096

$I_p = 1.0$

$V = \frac{(0.45)(1)}{(5.5)(0.97)}(30794.875)$

$T = C_t (h_n)^{3/4} = C_t (61 ft)^{3/4}$

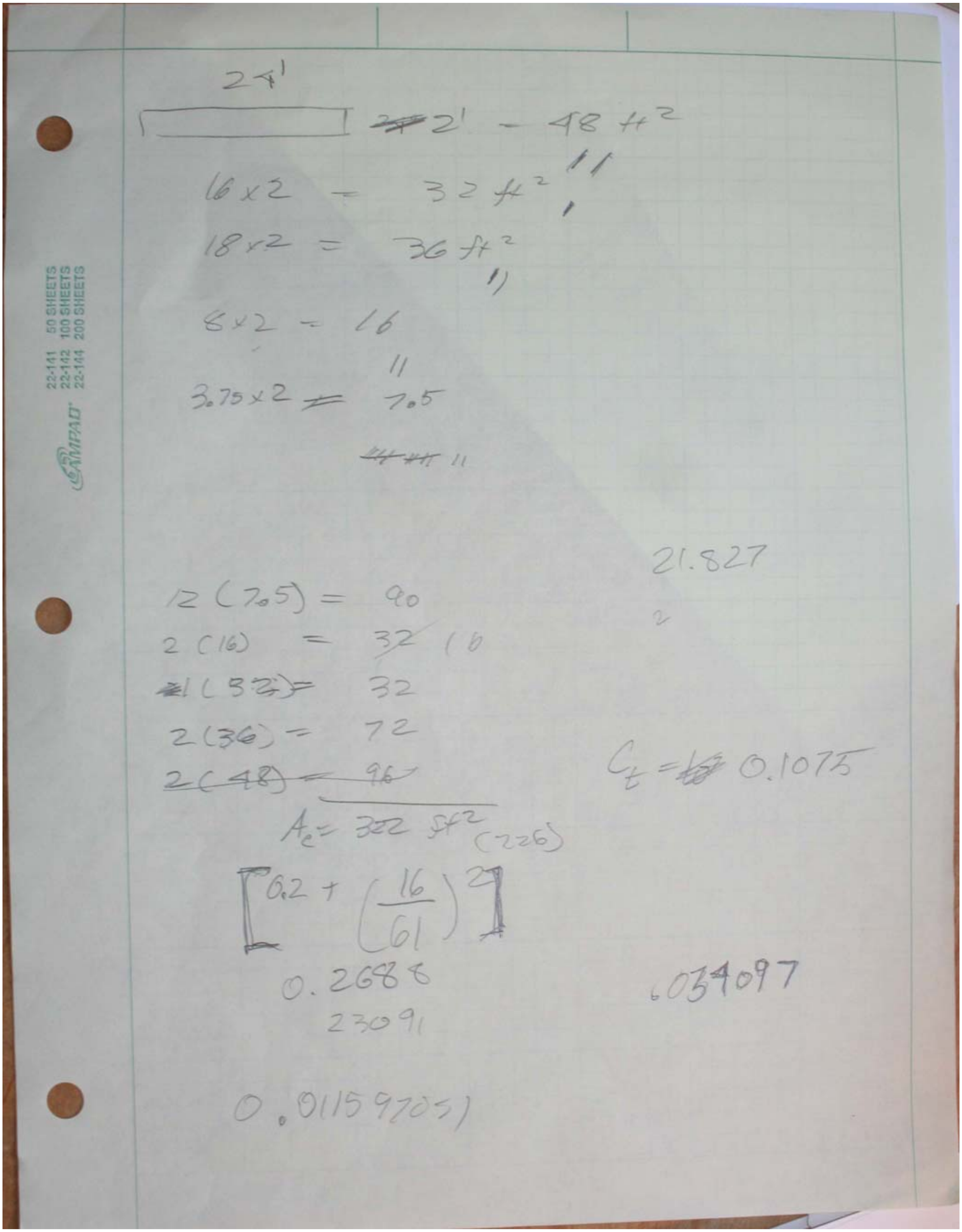
No Good

$C_t = 0.020 \; (0.0188)^{3/4}$

$= 0.00926 \; (0.0488)$

$A_e =$

APPENDIX 4



24'

2' – 48 ft²

16 x 2 = 32 ft²

18 x 2 = 36 ft²

8 x 2 = 16

3.75 x 2 = 7.5

21.827

12 (7.5) = 90

2 (16) = 32

1 (32) = 32

2 (36) = 72

~~2 (48) = 96~~

A_e = 322 ft² (226)

C_t = 0.1075

$$\left[0.2 + \left(\frac{16}{61}\right)^2\right]$$

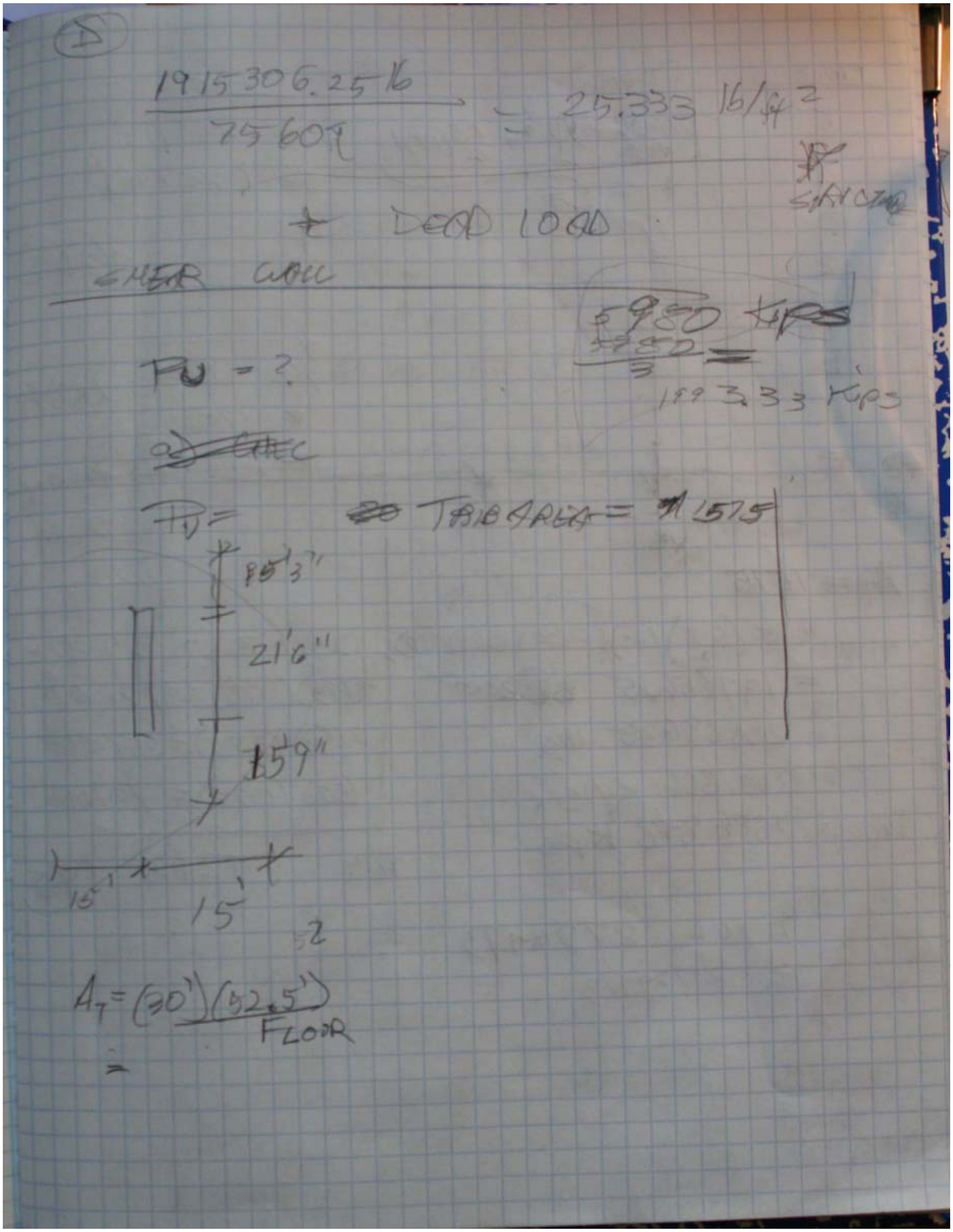
0.2688

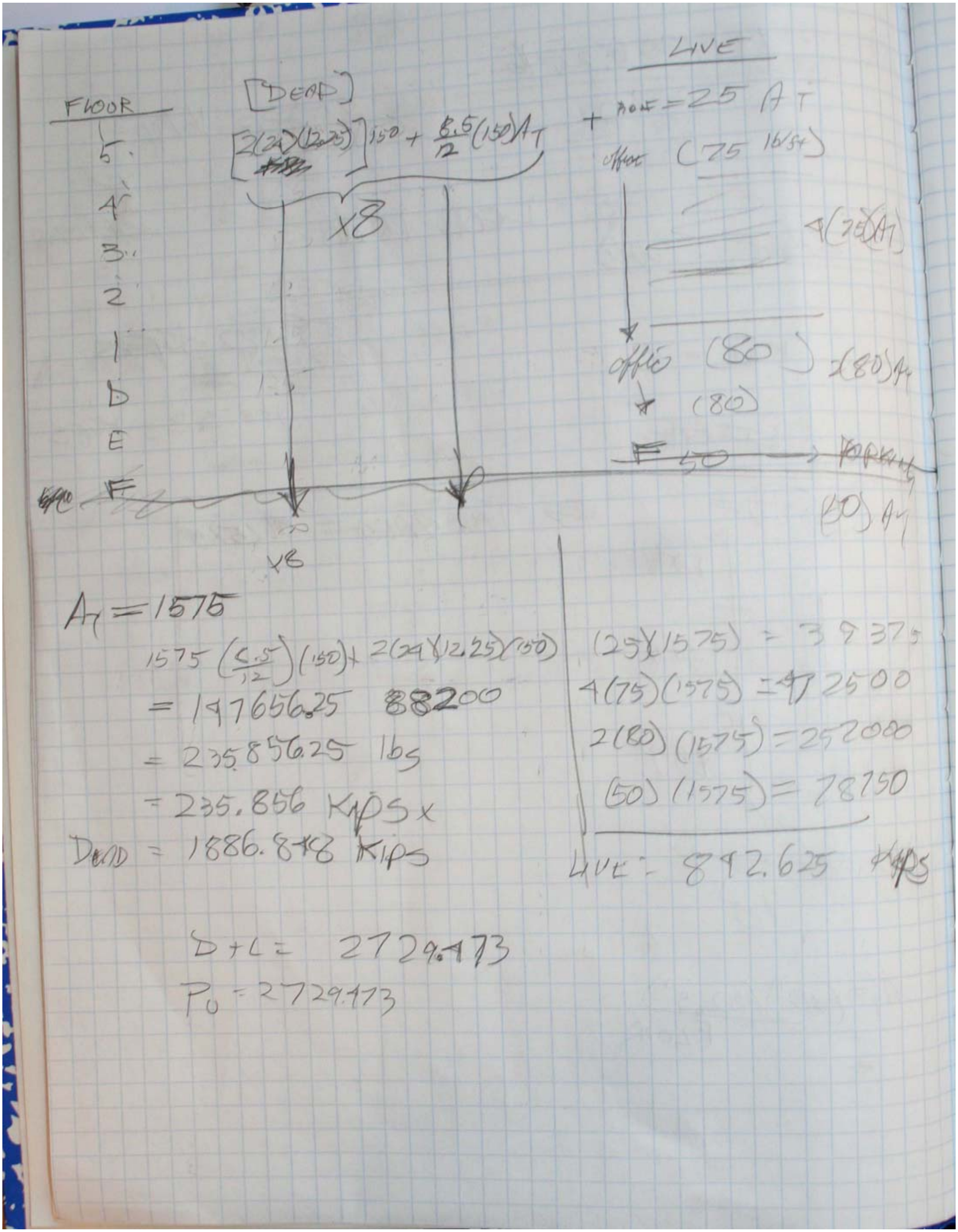
23091

.034097

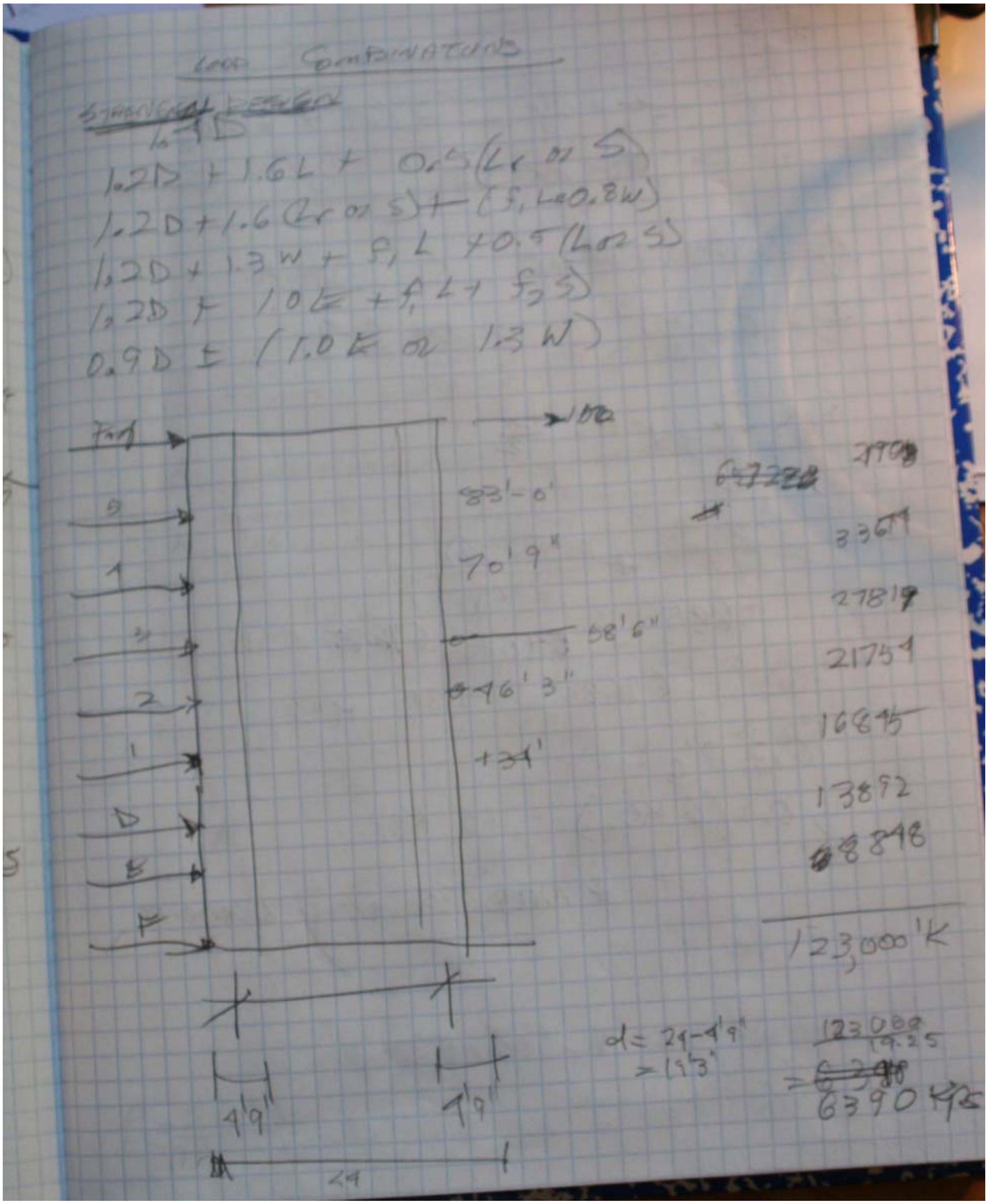
0.0115970051

Appendix 4


1915306.25 lb
75 60 ft
= 25.333 lb/ft²
+ DEAD LOAD
SHEAR WALL
5980 KIPS
5980 / 3 =
199 3.33 Kips
PU = ?
PU =
TRIB AREA = 1575
21'6"
15'9"
15'
15'
A_T = (30')(52.5')
FLOOR


LIVE
FLOOR
[DEAD]
5
4
3
2
1
D
E
F
x8
+ ROOF = 25 A_T
office (75 lb/sf)
4(75)(A_T)
office (80) 2(80)A_T
(80)
F 50 → PARKING
(50) A_T
x8
A_T = 1575
1575 (8.5/12)(150) + 2(24)(12.25)(150)
= 147656.25   88200
= 235856.25 lbs
= 235.856 KIPS x
DEAD = 1886.848 Kips
(25)(1575) = 39375
4(75)(1575) = 472500
2(80)(1575) = 252000
(50)(1575) = 78750
LIVE = 842.625 Kips
D + L = 2729.473
P_0 = 2729.473

Appendix 4


LOAD COMBINATIONS
1.4D
1.2D + 1.6L + 0.5(Lr or S)
1.2D + 1.6(Lr or S) + (f1L or 0.8W)
1.2D + 1.3W + f1L + 0.5(Lr or S)
1.2D + 1.0E + f1L + f2S
0.9D ± (1.0E or 1.3W)
83'-0"
70'9"
58'6"
46'3"
34'
123,000'K
d = 24-4'9"
= 19'3"
4'9"
4'9"
24
6390

Appendix 4

$C_v = \frac{P_u}{2} + \frac{M_u}{d} = \frac{2729}{2} + 6390 =$

$= 7754.5$

$A_g = (2)(24) = 48\ ft^2$

$I_g = \frac{(2)(24)^3}{12} = 2304\ ft^4$

$f_c = \frac{P_u}{A_g} + \frac{M_u \frac{h_w}{2}}{I_g}$

$= \frac{2729}{48} + \frac{1230(24/2)}{2304} =$

$5685 + 640 = 696.85$ KSF

$696.85 = 4.84$ KSI

$0.2\ f'_c$

$0.2\ (6\ KSi) - 1.2\ KSi$

* NEED BOUNDARY ELEMENT

Appendix 4

IF $V_u \geq 2 A_{cv}\sqrt{\ }$

NEED TWO CURTAINS REINF

$$\frac{2(12)(24)(24')(\sqrt{6000})}{1000} = 10.70K < V_u$$

NEED 2 CURTAINS

$$\rho_\ell, \rho_t = \frac{A_{s\ell}}{A_{cv}} \geq 0.0025$$

~~$A_{s\,req} = 0.0025\,(12)(24)^2 = 17.28\,in^2$~~

$$A_{s\,req} = 0.0025\,(12)(24) = 0.72\ in^2/ft$$

2 # 6 BARS/ft

$$2(0.44) = 0.88\ in^2/ft$$

$$\frac{0.72}{12} - \frac{0.88}{s} \Rightarrow 14.67$$

$$V_n = A_{cv}(\alpha_c\sqrt{f'_c} + \rho_t f_y)$$

$$\frac{h_w}{\ell_w} = \frac{100}{24'} > 2 \quad \therefore \quad \alpha_c = 2.0$$

$$V_n = A_{cv}(2.0\sqrt{6000} + 0.0031(60000))$$

$$\rho = \frac{2(0.44)}{(12)(24)} = 0.0031$$

$V_n = ((12)(24)(24))\left[2\sqrt{6000} + 0.0031(60000)\right]$

$V_n = \dfrac{6912(154.92 + 186)}{1000} = 2356$ kips

$(0.6)(2356) = 1413.86$ kips

NO GOOD

~~NEED ... THAT~~

$V_u = 1993.37$ kips

TRY 2 #7's

$2(0.60) = 1.20\ in^2/ft$

$\rho_t\ \dfrac{1.20}{12(24)} = 0.004167$

$V_n = 6912(154.92 + (0.004167)(60000))/1000$

$= 2798.96$

$0.60(2798.96)$

$= 1679.$ NFG ?

2 # 8's

$\dfrac{2(0.79)}{12(24)} = 0.005486$1

$V_n\ 6912\left[(154.92) + (0.005486\,1)(60000)\right] = 3346$

$\times 0.6 = 2007$

Appendix 4

2 #8's 12" O.C.

c) Check Boundary Element Capacity

$A_{ST} = 36\ (\#11)$

$= 36(1.56) = 56.16$ $\qquad \dfrac{56.16}{24(57)} = 0.04105 = \rho$

$\rho_{min} = 0.01 \quad \rho_{max} = 0.06$ ok

$$\phi P_n = 0.8\phi\left[0.85 f'_c (A_g - A_{ST}) + f_y A_{ST}\right]$$

$$= 0.8\,(0.7)\left[0.85(6)(1368 - 56.16) + 56.16(60)\right]$$

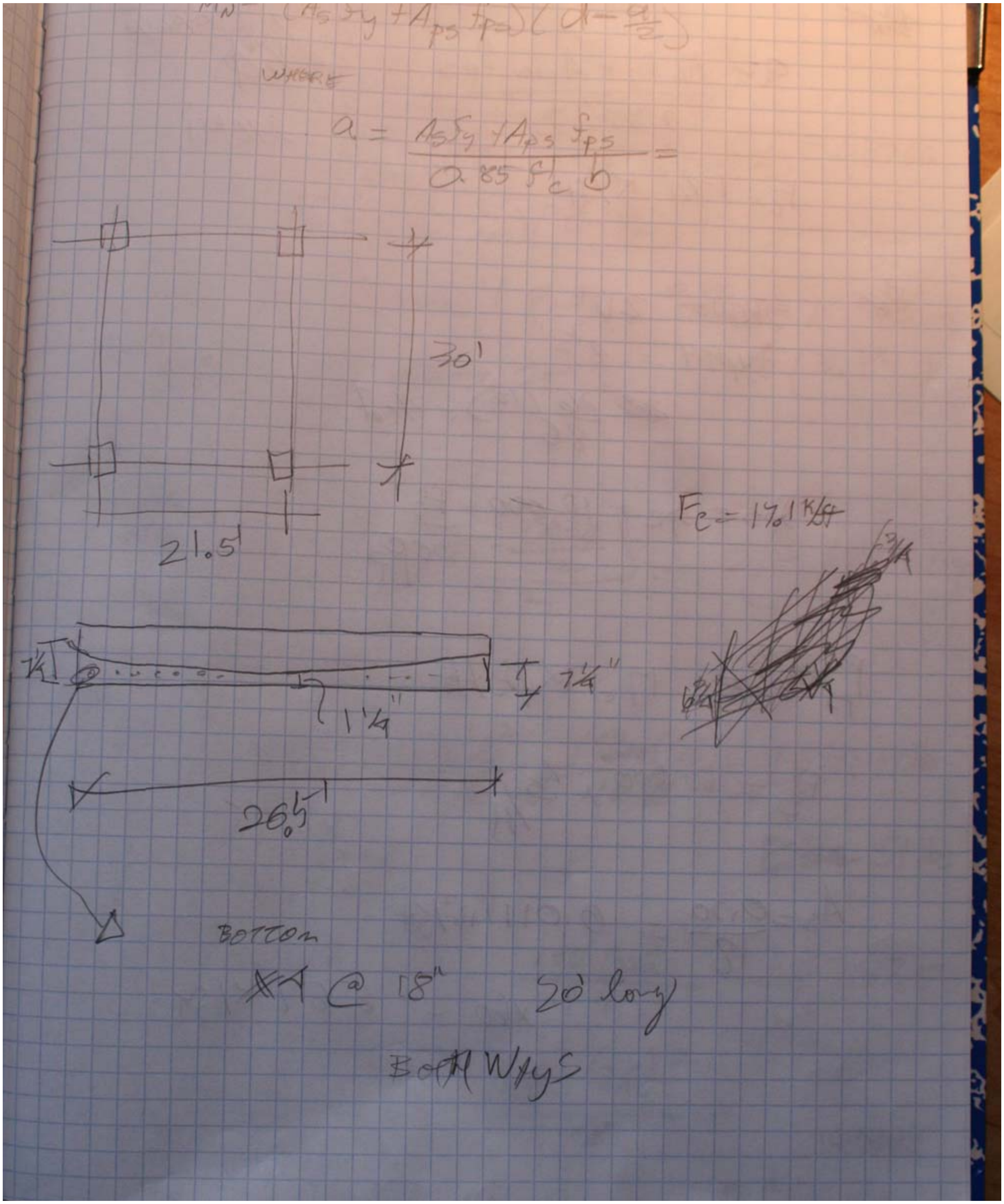
$$\left[6690 + 3369.6\right]$$

$$= 5648.5 \text{ kips}$$

∴ no good

→ over estimated loads

A trib > should be


WHERE
30'
21.5'
7¼"
1¼"
26.5'
BOTTOM
@ 18"
20' long
Both Ways

Appendix 4

~~M_N~~

$$a = \frac{A_s f_y + A_{ps} f_{ps}}{0.85 f'_c b}$$

7 wire STRAND
$\frac{1}{2}$" ϕ
270 ksi

$F_c = 17.1$ K/ft

~~8e~~ SPAN = 26'
Depth = $8\frac{1}{2}$"

$$\frac{26}{8.5}(12) = 36.7$$

$$f_{ps} = f_{pe} + 10{,}000 + \frac{f'_c}{300 \rho_p}$$

F_{ps} = 17.1 Kips/ft

$F_s = 60\,000 \times A_s/\text{ft}$

$$A_s = \frac{0.20}{18} = 0.0111 \text{ in}^2/\text{ft}$$

$\times 60 = 6.67$ K/ft

$$a = \frac{17.1 + 6.67}{0.85\, f'_c\, b} = \frac{23.77}{0.85\,(6000)(12)} = 0.376\text{ in}$$

$$M_n = (A_s\, S_y + A_{ps}\, S_{ps})\left(d - \frac{a}{2}\right)$$

$$M_n = (23.77\text{ kips})\left(7.25 - \frac{0.376}{2}\right) = 167.86\text{ in-kips}$$

$$= 13.99\text{ ft-kips}$$

MIDSPAN $d = 7.25$

$d = 1.25$

SUPPORT

$$\frac{(23.77)\left(1.25 - \frac{0.376}{2}\right)}{12} = 2.1036\text{ ft-kips}$$

DESIGN MOMENT

$$M = \frac{w\ell^2}{16} = \frac{100\,(26)^2}{16} = \frac{16900\text{ ft-lbs}}{2}$$

ft-lbs

75+75

$$\frac{100\,(26)^2}{16} = 6337.5$$

ok

$$\frac{150\,(26)^2}{16}$$