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Structural
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Koshland Integrated Natural Science Center
10/05/05
AE 481W

Structural Technical Report 1

Structural Concepts / Existing Conditions

Executive Summary

The purpose of this technical report is to descriptively explain the existing conditions, namely the structural system, of the Koshland Integrated Natural Science Center at Haverford College in Pennsylvania, through a number of analyses. The KINSC is four story science building that is a new edition to the Haverford campus. It is comprised of laboratory, classroom, and office spaces as well as numerous communal areas. The KINSC is directly connected to the two existing structures, Sharpless and Hillies Halls, but is very distinctive in its architecture and engineering.

This report is intended to give an introductory understanding to the buildings structural system. Included in this report are detailed descriptions of the foundation, the floor framing, the lateral force resisting system, the roof framing, as well as the design codes and standards used throughout the engineering of this structure. Also, in this report, the results of several spot checks throughout the building, by way of analysis, have been recorded and confirm that the sizes and reinforcing of the existing framing members are sufficient to carry the required loading. These spot checks include a typical precast concrete beam, a typical precast column, and a 10" hollow core precast plank floor system with a 2" topping. In addition, a shear wall was also analyzed to confirm that lateral support requirements were met. Following the body of the report, are Appendices provide copies of the calculations from the analysis as well as brief sketches of the framing system to allow for a better understanding.

Found within this report are also conclusions from all findings from analyses that were conducted. The primary structural system is precast concrete with the exception of the roof framing and foundation. However, the drawings and building information that was available, there are some key factors that are unknown, such as member reinforcement. Therefore, the spot check calculations could not be checked against existing conditions for validity. However, given the overall dimensions of most of the members, reasonable results were obtained and seem to be very convincing possibilities for what is actually found in the building.

Code Requirements (Existing)

The following design loads that were taken off of the drawings have been confirmed with the BOCA design code.

Gravity Loads:

Live Loads:

- Roof: Ground Snow Load - 30 psf
- Floor
  - Typical - 100 psf
  - Special (Lab Equip.) - Equip. Wt.
  - Library - 300 psf
  - Lobbies/Corridors/Entrances - 100 psf
  - Stairs - 100 psf
  - Mechanical Room - 125 psf
  - Storage - 125 psf

  (Drift as per BOCA 1610.7, Live Load Reduction as per BOCA 1608.0. No reduction at mechanical room floor or roof.)

Dead Loads

- Roof –
  - Ceiling - 5 psf
  - Mech., Elec., & Plumb. - 10 psf
  - Framing - 15 psf
  - Roofing & Insulation (flat roofs) 8 psf
  - Deck & Sheathing - 5 psf
  - Slate Roofing (sloped roofs) - 10 psf
- Floor –
  - Ceiling - 5 psf
  - Mech., Elec., & Plumb. - 10 psf
  - Framing - 15 psf
  - 2" Topping on 10" Spandeck - 91 psf
- Partitions –
  - 6" lightweight CMU wall - 30 psf (Surface Area)
- Exterior Wall –
  - Stucco (Stucco, conc., insulation, CMU) - 71 psf
  - Typical Stone Wall (field stone, insulation, CMU) - 140 psf
  - Battered Stone Wall (field stone, insulation, CMU) - 215 psf

Lateral Loads

- Wind (93 BOCA and ASCE 7-88)
- Seismic – per BOCA 1993 section 1612.0
- Active Soil Pressure
  - Cantilever Retaining Walls - 50 psf/ft

Design Codes and Design Standards

General:
- BOCA National Building Code/ 1993
- Building Code Requirements for Reinforced Concrete (ACI 318-319, 1992)
- AISC Manual of Steel Construction – LRFD (1st Edition)

Precast Structural Concrete
- PCI Manual on Design of Connections for Precast Pre-stressed Concrete
- PCI Design Handbook – Precast and Pre-stressed Concrete
- ACI 301, 318
- ANSI/AWS D1.1 – Structural Welding Code
- PCI MNL – 116, 119

Description of the Structural System

The design of the KINSC makes use of a variety of structural materials and methods of support. The foundation and four floors of the structure are framed with precast concrete columns and beams. However, the roof of the building incorporates steel columns in the framing. The roof of the science center is a bent-frame structure that is also framed with steel W shaped members. The exterior walls that enclose the entire 185,000 square feet of the building are a combination of precast concrete, various sizes of CMU block walls, and stone facades. The floor system for the building is a 10" hollow-core plank system with a 2" topping slab reinforced with prestressed steel. The structure also makes use of precast shear walls to help support the structure in lateral loading. To allow for an easier understanding of the structural system, in this report, the building has been separated into the East Wing, the Link, and the West Wing. See Appendix E for a typical floor layout of the KINSC.

East Wing

Foundation

The foundation floor of the East Wing of the KINSC is a 4 inch thick slab-on-grade that sits on a 4 inch layer of crushed stone. The slab is reinforced with WWF 6x6 throughout its entirety. The foundation walls are built up out of CMU blocks that vary in thickness from 8" to 12". The reinforcement is unknown. Also, along some sides of the building, retaining walls are used as a part of the foundation to support the structure from the lateral loads of the soil. These retaining walls have rectangular footings that range from 22" to 44" in depth and are typically 14' in total width. The reinforcement for these footings is typically #10 bars at 8" on the bottom and #6 bars at 12" on the top. Along the retaining walls are a number of 17" pilasters to help reinforce the wall in bending. The retaining walls are heavily reinforced. The exterior foundation walls that are not considered retaining walls are supported by strip footings that range in width from 2'-8" to 10'-6". The precast piers that are supporting the first floor, are sitting on square footings that are approximately 12’ in width and 2-2” deep. These footings typically have 11-#10 bars as reinforcement in both directions.

Framing

Floors 1, 2, 3, and 4 above grade are framed similarly in terms of spans and member sizes. In the East Wing, precast concrete beams are used to support the floors above. The interior beams are sized at 20”x12” while the exterior beams are sized at 12”x24”. The beams have a typical span of 21’-0” throughout the length of the building. The exterior walls of the East Wing are constructed of 8” and 10” CMU blocks with 10” precast concrete panels or quarried stone façade on the outside of the walls. The columns used to support the floors are also precast concrete. The interior columns are sized at 18”x30” while the exterior columns are 22”x22”. The columns span a height of 13’-0” from floor to floor. The floor is a system of hollow-core precast planks that are 10” deep with a 2” slab topping. The slab is reinforced with fibrous reinforcing. The steel reinforcement of the framing members is unknown.

Shear Walls

The KINSC makes use of precast concrete shear walls to resist lateral forces on the building. There are two shear walls on each floor, located in the East Wing that run in the East to West direction. The shear walls span 31’-5”. The overall dimensions and the reinforcement in the shear walls are unknown. Also, due to the fact that no other shear walls are identified on the plans in either direction, it is assumed that the exterior walls are contributing some area to act as shear walls in both directions.

West Wing

Foundation

The foundation of the West Wing is found to be very similar to the foundation of the East Wing with a few exceptions. The foundation floor is a 4 inch slab on grade that sits on top of a 4 inch layer of crushed stone and is reinforced with WWF 6x6 throughout. The foundation walls are constructed of 8” to 12” thick CMU block. The reinforcement is unknown. The retaining walls used in the West Wing rest on rectangular footings that are between 22” to 26” in depth and typically 6’-6” wide. The reinforcement for the footings is typically #8 bars at 8 inches at the bottom, and #6 bars at 12” at the top. To help with lateral load from the soil, 22” pilasters are used. The rest of the foundation walls sit on strip footings that range in width from 5’ to 10’-6”. The precast piers supporting the first floor sit on square footings that are typically 12’ in width and 2’-2” in depth. The reinforcement is 11-#10 bars in each direction.

Framing

The structural framing of the West Wing is practically identical to the framing of the East Wing. Interior beams are sized at 20”x12” while the exterior beams are 12”x24”. All -

beams have a typical span of 21'-0". The precast columns are sized at 18"x30" for the interior, and 22"x22" for exterior locations. All columns have a floor to floor span height of 13'-0". The floor system is also the 10" hollow core precast planks with 2" topping, reinforced with fibrous reinforcing. The wall thicknesses and materials remain the same as that of the East Wing. The reinforcing of the framing members is unknown.

Shear Walls

The KINSC makes use of precast concrete shear walls to resist lateral forces on the West Wing of the building as well. There are two shear walls on each floor, located in the West Wing that run in the East to West direction. The shear walls span 26'-8" and 31'-2". The overall dimensions and the reinforcement in the shear walls are unknown. Also, due to the fact that no other shear walls are identified on the plans in either direction, it is assumed that the exterior walls are contributing some area to act as shear walls in both directions.

Link

Foundation

The foundation floor of the Link is a 4" slab on grade that sits on top of a 4" layer of crushed stone. It is reinforced with WWF 6x6 reinforcing. The exterior foundation walls are 20" cast-in-place concrete walls with #5 longitudinal bars at 12" and #9 traverse bars at 12" for reinforcement. The foundation walls are supported by strip footings that range from a width of 6'-4" to 9'-0" and are 1'-8" deep.

Framing

The framing for the Link consists of all bearing walls ranging in thickness from 8" to 14". The reinforcement for the bearing walls is unknown. The floor system is a 10" hollow core plank system with a 2" topping. It is reinforced with fibrous reinforcing throughout. No beams or columns are used in this section of the building because of the small overall spans.

Shear Wall

In this section of the building, there is only one shear wall located on each floor. The shear wall runs in the North to South direction and spans 16'-6". The shear wall is constructed of 10" CMU blocks, but the reinforcement is unknown. It is assumed that because this is the only shear wall that is called out on the drawings, then some of the exterior wall area is used as shear wall area in both directions.

Central Stair (Stair No. 3)

The central atrium which is the main stairwell for the building is constructed of precast concrete panels of 1'-4" thickness, HSS6x4x5/16 tube columns, TS 8x3 beams. The precast panels as well as the precast stairs are heavily reinforced. The stairs are cantilevered out of the exterior wall as they spiral upward. The other means of egress are the elevators where the shafts are framed out with 8" CMU block. In addition to the CMU elevator shaft, the elevator and stairwell located in the West Wing are framed out with steel beams of sizes W10x12 and W12x16 predominantly.

Roof

The roof to the KINSC is framed with structural steel members. The columns supporting the bent-frame steel roof are standard W-shape steel columns consisting of W10x30, W10x33, and W10x45 in size. These columns are positioned directly above the precast concrete columns on the floors below. On the East Wing, the framing between the bent-frames are typically W14x22 members on the exterior spans with W12x22 and W12x26 members spanning the long direction of the building in the interior span. The most typically bent-frame is comprised of W14x34 members, both angled and horizontal. The other bent-frames on the East Wing are constructed with members that are very similar in size and weight to this bent-frame. As for the West Wing, the roof framing is similar. The steel members framing between the bent-frames are typically W14x22 members for the exterior spans with W12x16 members spanning the long direction of the building for the interior spans. The steel beams that run along the exterior walls at both wings are sized at W16x50.

Spot Check Results

Beams

A typical precast concrete beam from the first floor in the East Wing was chosen for the spot check. From the drawings, it appears that the interior beams spanning the length of the building are cantilevered and have several typical interruptions that occur between spans. Since the floor layout still requires support where these interruptions take place, the assumption made is that the interruption in the beam span is a designation for a cast-in-place concrete simple span beam. Assuming this is the case, the reactions from the simple beam were applied to the cantilevered portion of the precast beam to account for the balancing of flexural moments. The precast beam has a center span of 21' and has a cantilever on each side. The length of the cantilever is not shown, therefore it was measured by scale of the drawing to be 1'-3". The reinforcement in the beam is unknown. Therefore the results from the analysis could not be checked for result accuracy. For the entire set of calculations, see Appendix A

- 20"x12" precast concrete beam
- Center span of 21'-0", cantilever of 1'-3" on each side
- Reaction from simple span C.I.P. beam: P = 54.8 k
- Calculated distributed load: wu = 7.058 klf
- Reactions at supports: P = 137.7 k
- @ midspan:
  - M_u = 388.9 'k

- Required tension steel: 2 layers of 7 – 0.6" dia. strands; As = 3.03 sq. in.
- Required compression steel: 6 - #11 bars; As = 9.36 sq. in.
- ΦM_n = 416.4 'k > M_u OK for flexure

- @ supports:
  - M_u = 74'k
  - Required tension steel: 7 – ½" dia. strands; As = 1.07 sq. in.
  - ΦM_n = 168 'k > M_u OK for flexure

Hollow Core Plank Floor System

The typical flooring for the KINSC was designed to be a 10" hollow core plank system with 2" topping. However, no reinforcement of the hollow core planks were provided in the drawings that were available for review, therefore some assumptions had to be made. Also, it was noted in the drawings that SpanDeck is the type of hollow core planks used. However, while referencing the PCI Manual for the Design of Hollow Core Slabs, a 10" section could not be found from SpanDeck, therefore, a 10" section with 2" topping from Dy-Core was used as a similar section. For entire calculations, see Appendix B.

- 10" hollow core planks w/2" topping
- Assume 4' panels
- Span = 31'-5"
- w_u = 1.2(DL) + 1.6(LL) = 1.36 klf
- Assume simple span: M_u = (w_u*l2)/8 = 142.9'k
- Use 6 – 0.6" dia. prestressed tendons
- ΦM_n = 180.3 'k > M_u = 142.9'k   OK for flexure

Columns

The typical interior precast concrete column supporting the interior precast beam that was checked, located on the first floor, was chosen for a spot check. Although the column is dimensioned as 22"x22" with a height of 13', no steel reinforcement was shown in the drawings provided. Therefore, an assumption had to be made. The load is acting at the center of the column cross section, therefore there is no eccentricity causing any moment in the column. It was assumed that the reinforcement in the column consisted of 4 - #8 bars giving an area of steel of 3.16 square inches.

- Dimensions: 22" square column
- Span: 13' floor to floor
- Assume 4 - #8 bars for reinforcement; As = 3.16 sq. in.
- Reaction from Beam Check: P = 137.7 kips (per floor)
- Total Axial Load: P_u = 551 k
- ΦPn = 1006 k; Design is sufficient

The reason for the over-designing of the column may be due to the limited deflection that is allowed for the concrete column for story drift before the concrete cracks. Another reason could be for connection purposes, providing a better fit for beams to columns. See Appendix C for calculations.

Shear Wall

The design of the KINSC incorporates the use of precast concrete shear walls to resist the lateral forces acting on the structure. Shear walls are located on every floor of the building. A shear wall in the East Wing at the southern end of the building was chosen to be checked. The thickness and reinforcement of the shear wall is unknown. It was assumed to be 12" thick. Tributary areas were used to dissipate the lateral loads to each of the resisting shear walls. It was found that the shear wall requires hold downs to resist a force of 213 kips. See Appendix D for entire calculations.

- Tributary width: 73'-3"
- Lateral forces
  - Roof – 58 k
  - 4^{th} floor – 197 k
  - 3^{rd} floor – 291 k
  - 2^{nd} floor – 340 k
- M_{OT} = 10548 'k
- M_R = 3899 'k
- T = 213 k

Load Design

Wind vs. Seismic

The following tables show the findings from the wind analysis and seismic analysis of the 4-story KINSC, as per ASCE 7-02. The lateral force resisting system in the building is comprised of shear walls that have the same tributary area, whether dealing with wind or seismic loading. With this in consideration, and with the results from the tables below, it was concluded that the seismic loading was the controlling lateral force for the building.

The following tables/diagrams:

Wind factors and tabulations
Wind loading diagrams
Seismic factors and tabulations
Seismic loading diagrams

Wind – ASCE 7 – 02 Sec. 6

Wind Analysis

Simplified Method - ASCE 7 - 02 Sec. 6.4

Wind Load Factors			
mean building height (must be < 60'):	h (ft.) =	53.19	
Basic Wind Speed:	V (mph) =	75	From General Notes on Plans
Building Category:	Category	III	Table 1-1
Importance Factor:	I =	1.15	Table 6-1
Exposure Category:	Category	B	Sec. 6.5.6
Ht.& Exposure Adjustment Coeff.:	λ =	1.178	Fig. 6-2; by interpolation

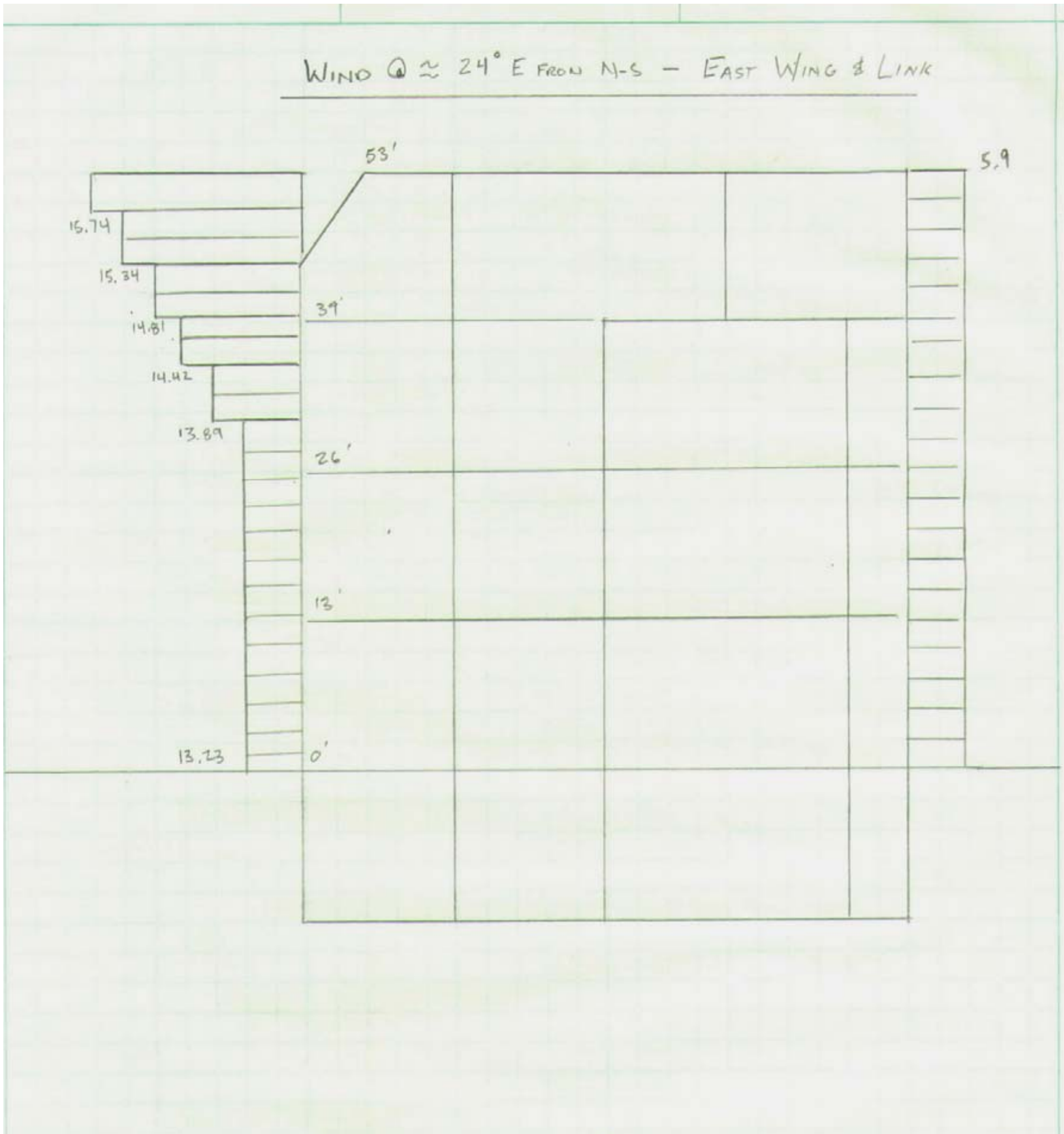
Zone	p_{s30}	
A	11.5	Horizontal Pressures
B	-5.9	
C	7.6	
D	-3.5	
E	-13.8	Vertical Pressure
F	-7.8	
G	-9.6	
H	-6.1	

$p_s = \lambda * I * p_{s30}$
I = 1.15
p_{s30} = 11.5 - (-5.9)
λ: see below

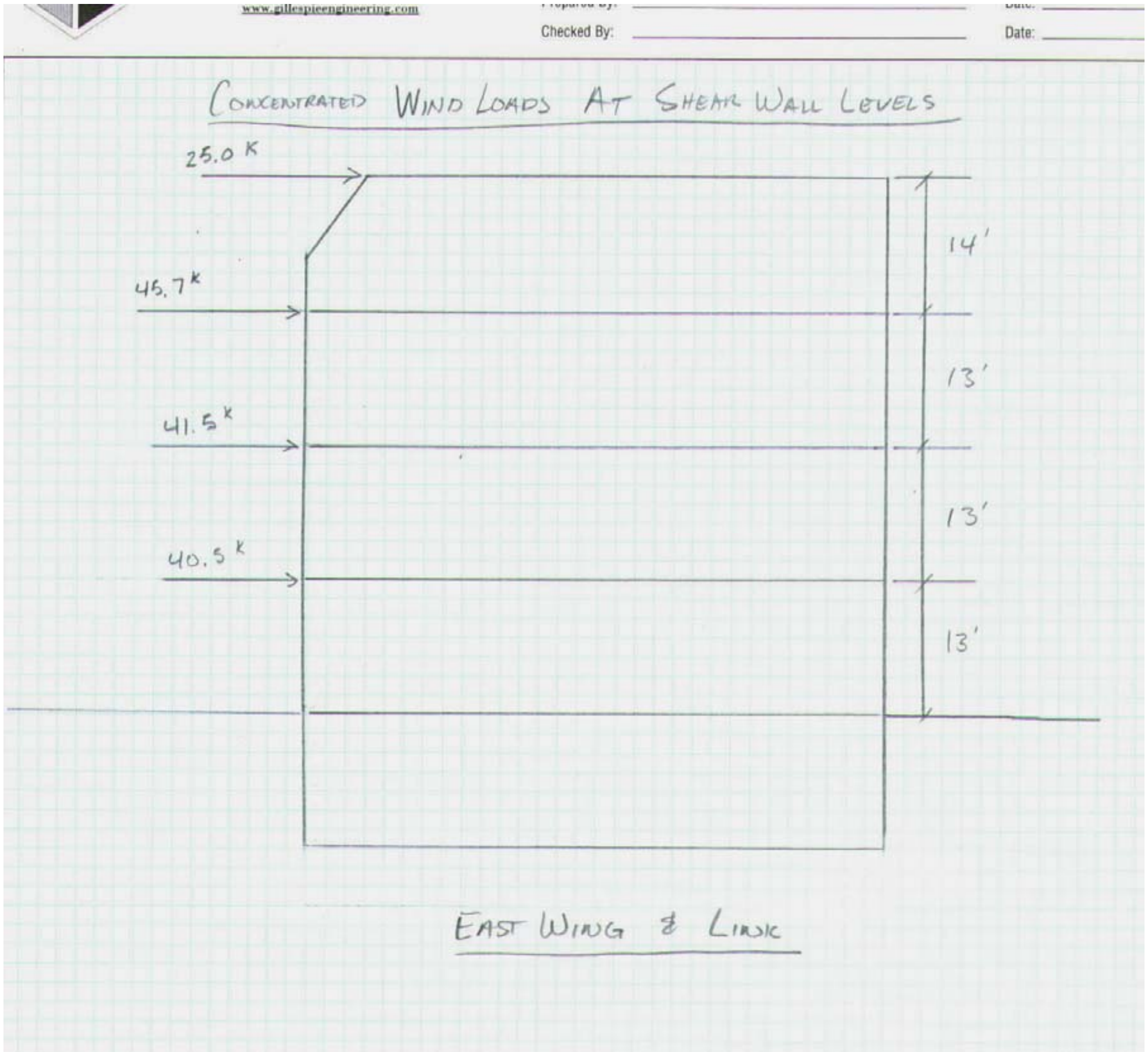
height	λ	I	$p_s = \lambda * I * p_{s30}$ (psf)
15	1.00	1.15	19.13
20	1.00	1.15	19.13
25	1.00	1.15	19.13
30	1.00	1.15	19.13
35	1.05	1.15	19.79
40	1.09	1.15	20.32
45	1.12	1.15	20.71
50	1.16	1.15	21.24
55	1.19	1.15	21.64
60	1.22	1.15	22.03

Level	plf of width
Roof	151.97
4th	277.88
3rd	252.44
2nd	248.69
1st	0
Basement	------

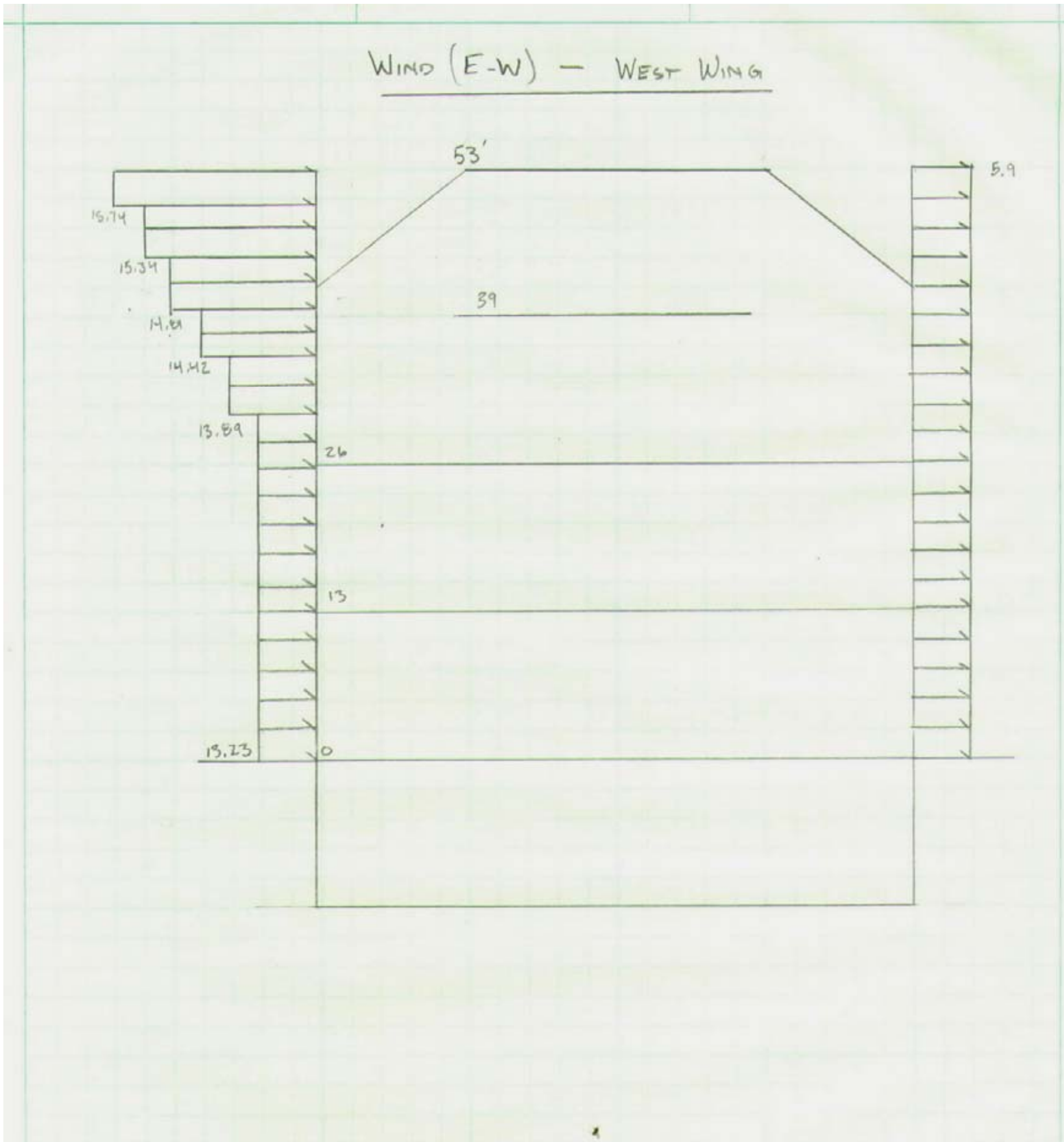
East Wing & Link: Wind Loads (psf) Controlling Direction



East Wing & Link: Concentrated Wind Loads in Controlling Direction



West Wing: Wind Loads (psf) Controlling Direction



Seismic – ASCE 7 – 02 Sec. 9:

Building Information

Building Location		Haverford, PA
# of stories		4
inner story ht.		13
Bldg. height		53
Seismic Use Group		III
Importance Factor		1.15
Site Classification		B
0.2s Acceleration		0.35
1.0s Acceleration		0.08
Site Class Factor:		
	F_a	1.00
	F_v	1.00
Adjusted Accelerations		
	S_{ms}	0.35
	S_{m1}	0.077
Spectral Response Accelerations		
	S_{DS}	0.233
	S_{D1}	0.051
Seismic Design Category		**B**

Seismic Analysis

East Wing & Link

Vertical Distribution of Seismic Forces (N-S)

$k_{N\text{-}S} = 1 + (T_{N\text{-}S} - 0.5)/(2.5 - 0.5) =$ 0.946

Level, x	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x	V_x	M_x
	(kips)	(ft.)			(kips)	(kips)	(ft-kips)
Roof	1278	53	54776	0.17	87		4604
4	4092	39	131144	0.408	208	87	8111
3	4092	26	89349	0.278	142	295	3684
2	4092	13	46365	0.144	74	437	956
1						510	
	Σ= 13554		Σ= 321634	Σ= 1	Σ= 511		Σ= 17355

Vertical Distribution of Seismic Forces (E-W)

$k_{E\text{-}W}$ = 1+ ($T_{E\text{-}W}$ 0.5)/(2.5 - 0.5) = 0.946

Level, x	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x	V_x	M_x
	(kips)	(ft.)			(kips)	(kips)	(ft-kips)
Roof	1278	53	54776	0.17	58		3069
4	4092	39	131144	0.408	139	58	5408
3	4092	26	89349	0.278	94	197	2456
2	4092	13	46365	0.144	49	291	637
1						340	
	Σ= 13554		Σ= 321634	Σ= 1	Σ= 340		Σ= 11570

West Wing

Vertical Distribution of Seismic Forces (N-S)

$k_{N\text{-}S}$ = 1+ ($T_{N\text{-}S}$ - 0.5)/(2.5 - 0.5)
= 0.946

Level, x	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x	V_x	M_x
	(kips)	(ft.)			(kips)	(kips)	(ft-kips)
Roof	892	53	38211	0.164	61		3223
4	2993	39	95917	0.411	153	61	5954
3	2993	26	65349	0.28	104	213	2704
2	2993	13	33911	0.145	54	317	702
1						371	
	Σ= 9871		Σ= 233388	Σ= 1	Σ= 372		Σ= 12583

Vertical Distribution of Seismic Forces (E-W)

$k_{E\text{-}W}$ = 1+ ($T_{E\text{-}W}$ 0.5)/(2.5 - 0.5)
= 0.946

Level, x	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x	V_x	M_x
	(kips)	(ft.)			(kips)	(kips)	(ft-kips)
Roof	892	53	38211	0.164	41		2149
4	2993	39	95917	0.411	102	41	3969
3	2993	26	65349	0.28	69	142	1803
2	2993	13	33911	0.145	36	212	468
1						248	
	Σ= 9871		Σ= 233388	Σ= 1	Σ= 248		Σ= 8389

Snow Loads

Snow Loads – ASCE 7- 02 Sec. 7.3

Snow Load Analysis

Flat Roof Snow Loads - ASCE 7 - 02 Sec. 7-3

$p_f = 0.7*C_e*C_t*I*p_g$

$p_{f,min} = 20*I$ (For $p_g > 20$ psf)

p_g (psf) =	30	(As per General Notes; see structural dwgs.)
C_e =	0.7	(As per General Notes; see structural dwgs.)
C_t =	1	(Table 7-3)
I =	1	(As per General Notes; see structural dwgs.)

p_f (psf) =	14.7	$p_{f,min}$ (psf) =	20	<= ***Controls***

Appendix A

Beam Spot Check

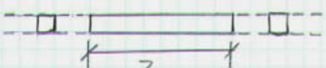
Precast Concrete Beam

210 Arnold Avenue, Suite F
Point Pleasant Beach, New Jersey 08742
Voice: (732) 892-6979 Fax: (732) 892-6976
www.gillespieengineering.com

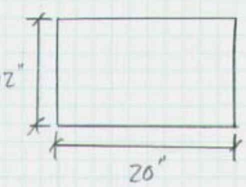
Project No.: ______
Subject: ______ Sheet ___ of ___
Prepared By: ______ Date: ______
Checked By: ______ Date: ______

BEAM SPOT CHECK

WHEN VIEWING THE TYPICAL FLOORS ON THE DRAWINGS, IT WAS NOTED THAT THERE ARE SEVERAL SIMILAR INTERRUPTIONS IN THE BEAM SPANS. IT IS ASSUMED THAT THE INTERRUPTION DESIGNATES LOCATIONS FOR SIMPLE-SPAN CAST-IN-PLACE CONCRETE BEAMS TO BE POURED. THE ACTUAL SPAN IS NOT NOTED.



ASSUMED SPAN OF 18'-6" (MEASURED OFF THE DRAWINGS)



$A = \left(\frac{31.5'}{2} + \frac{15.67'}{2}\right)(18.5')$
$= 417.8\ ft^2$

$A_I = 2(A) = 2(417.8) =$
$= 835.65\ ft^2$

DL: - CEILING = 5 psf
- MEP = 10 psf
- FRAMING = 15 psf
- 10" Hollow Core PLANKS w/ 2" TOPPING = 91 psf
- PARTITIONS = 30 psf

151 psf

Self wt = $\left(\frac{20}{12}\right)\left(\frac{12}{12}\right)(150\ pcf) = 250\ plf$

LL: Typical = 100 psf

$L = L_o\left(0.25 + \frac{15}{\sqrt{A_I}}\right) = 100\left(0.25 + \frac{15}{\sqrt{835.7}}\right) = 76.9\ psf > 0.5L_o \therefore OK$

$W_u = 1.2(151(18.5') + 250) + 1.6(76.9(18.5'))$
$= 5.93\ klf$

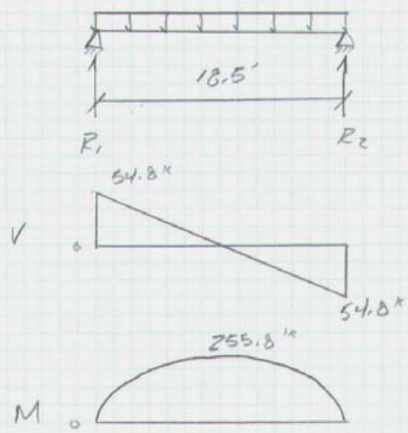
Precast Concrete Beam

210 Arnold Avenue, Suite F
Point Pleasant Beach, New Jersey 08742
Voice: (732) 892-6979 Fax: (732) 892-6976
www.gillespieengineering.com

Project No.:
Subject:
Prepared By:
Checked By:

Sheet ____ of ____
Date:
Date:

$W_u = 5.93$ klf



$$R_1 = R_2 = \frac{(5.93)(18.5)}{2} = 54.8^k$$

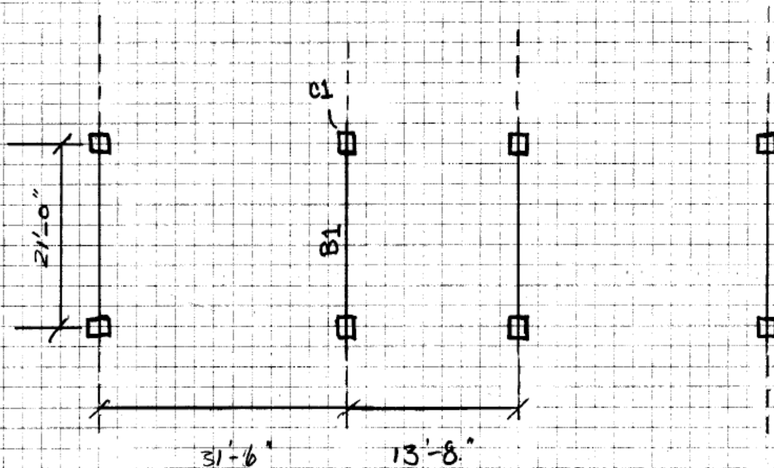
$$M_u = \frac{w l^2}{8} = \frac{5.93\,(18.5')^2}{8} = 255.8^{'k}$$

THE END REACTIONS ON THIS BEAM MUST BE APPLIED TO THE CANTILEVERED ENDS OF THE TYPICAL BEAM BEING CHECKED.

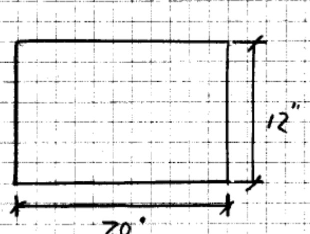
• 2ND FLOOR: TYPICAL BAY

BEAM SPOT CHECK

trib w = 22.586'



B1



D.L. ⇒ CEILING = 5 psf
MEP = 10 psf
FRAMING = 15 psf
2" TOPPING ON
10" HOLLOW CORE PLANKING
= 91 psf
PARTITIONS = 30 psf
151 psf

$A = \left(\frac{31.5'}{2} + \frac{13.67}{2}\right)(21') = 474.3\ ft^2$

$A_T = 2(A) = 2(474.3) = 948.57\ ft^2$

Self wt. = $\left(\frac{20}{12}\right)\left(\frac{12}{12}\right)(150\ pcf) = 250\ plf$

LL ⇒ Typ. = 100 psf

$L = L_o\left(0.25 + \frac{15}{\sqrt{A_T}}\right) = 100\left(0.25 + \frac{15}{\sqrt{948.57}}\right) = 73.7\ psf > 0.5L_o \therefore OK$

$W_u = 1.2\left(151\,(22.585) + 250\right) + 1.6\left(73.7\,(22.585)\right) = 7055.6\ plf = 7.056\ klf$

4392.4

Precast Concrete Beam

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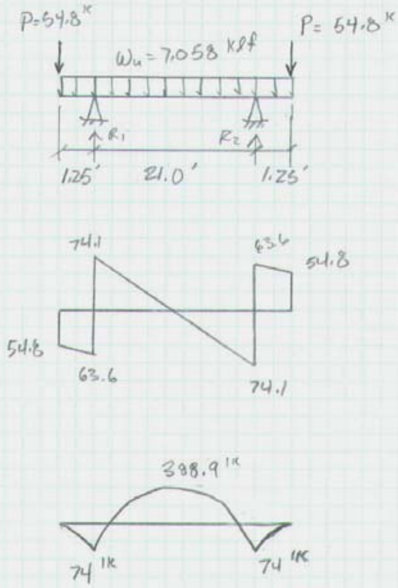

$$R_1 = R_2 = \frac{7.058(23.5') + 2(54.8)}{2}$$

$$= 137.7^k$$

@ MIDSPAN: ASSUME: 1.) PRESTRESSED TENDONS (BONDED) ARE USED FOR REINFORCEMENT

2.) $f_{pu} = 270$ Ksi , $f'c = 5000$ psi

3.) $f_{pe} = 0.55\, f_{pu} = 148.5$

4.) $d_p = 10.67''$

5.) Assume 7 - $\frac{1}{2}''\phi$ bars , "low lax"

PRESTRESSED FLEXURE FLOW-CHART

$A_{ps} = 7(0.153'') = 1.071\ in^2$

$\rho_p = \frac{A_{ps}}{bd^2} = \frac{1.071}{(20)(10.67)^2} = 0.00047$

$\frac{L}{d_p} = \frac{21'\text{-}0(12)}{10} = 25.2 \leq 35$

$\therefore K = 100$

$C = 60$

$f_{ps} = f_{pu}\left[1 - \frac{\gamma_p}{\beta_1}\left(\rho_p \frac{f_{pu}}{f'c}\right)\right]$

$\gamma_p = 0.28$

$\beta_1 = 0.80$

$= 270\left[1 - \frac{0.28}{0.8}\left(0.00047\left(\frac{270}{5}\right)\right)\right]$

$= 267.6$ Ksi

Precast Concrete Beam

$$C = \frac{A_{ps} f_{ps}}{0.85 f'_c \beta_1 b} = \frac{1.071(267.6)}{0.85(5)(0.8)(20)} = 4.21''$$

$$d_e = \frac{A_{ps} f_{ps} d_p}{A_{ps} f_{ps}} = d_p = 10.67''$$

$$a = \beta_1 c = 0.8(4.21'')$$
$$= 3.37''$$

$$\frac{c}{d_e} = \frac{4.21}{10.67} = 0.394$$

$$\phi = 0.37 + \frac{0.20}{(c/d_e)} = 0.37 + \frac{0.20}{0.394}$$
$$= 0.88$$

6.74

$$\phi M_n = \phi[A_{ps} f_{ps}(d_p - a/2)]$$
$$= 0.88\left(1.071(267.6)\left(10.67 - \frac{3.37''}{2}\right)\right)$$
$$= 188.84 < M_u = 388.97 \therefore \text{No Good}$$

Try 14 – 0.6" ϕ bars (two rows $\Rightarrow d = 9.5''$)

& Try 6 – #11 bars for compression reinforcement

$A_s' = 9.36 \text{ in}^2$ $\quad A_s = 3.03$

$f_{ps} = 261.8$ ksi

$$C = \frac{3.03(261.8) - 9.36(60)}{0.85(5)(0.8)(20)}$$
$$= 3.4$$

$$d_e = 9.5$$

$$a = \beta_1 c = 0.8(3.4) = 2.72''$$

$$\frac{c}{d_e} = \frac{3.4}{9.5} = 0.357 \longrightarrow \phi = 0.9$$

$$\phi M_n = 0.9\left(3.03(267.6)\left(9.5 - \frac{2.72}{2}\right) - 9.36(60)\left(1.5 - \frac{2.72}{2}\right)\right)$$
$$= 495.0^{'k} - 78.624 = \boxed{416.4^{'k} > M_u = 388.9^{'k} \therefore \text{ok}}$$

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@ Supports   Assume: $d_p = 10''$

Try 7 - $\frac{1}{2}''\phi$ strands, low-lax

$A_{ps} = 7(0.153) = 1.071 \text{ in}^2$

$\frac{L}{d_p} = \frac{23.5'(12)}{10''} = 28.2 \leq 35$

$\therefore K = 100$

$c = 60$

$\rho_p = \frac{1.071}{(20)(10.0)^2} = 0.00054$

$\gamma_p = 0.28$

$\beta_1 = 0.80$

$f_{ps} = f_{pu}\left[1 - \frac{\gamma_p}{\beta_1}\left(\rho_p \frac{f_{pu}}{f'_c}\right)\right]$

$= 270\left[1 - \frac{0.28}{0.8}\left(0.00054\left(\frac{270}{5}\right)\right)\right]$

$= 267.2 \text{ ksi}$

$c = \frac{A_{ps} f_{ps}}{0.85 f'_c \beta_1 b} = \frac{1.071(267.2)}{0.85(5)(0.8)(20)} = 4.21''$

$d_e = d_p = 10''$

$a = \beta_1 c = 0.8(4.21'')$
$= 3.37''$

$c/d_e = \frac{4.21}{10''} = 0.421$

$\phi = 0.37 + \frac{0.2}{(c/d_e)} = 0.37 + \frac{0.2}{0.421} = 0.85$

$\phi M_n = \phi\left[A_{ps} f_{ps}(d_p - a/2)\right]$

$= 0.85\left[1.071(267.2)(10 - \frac{3.37}{2})\right]$

$= 168.5'^k > M_u = 74'^k \quad \therefore \text{ OK}$

Use 7 - $\frac{1}{2}''\phi$ strands

Appendix B

Floor System Spot Check

Hollow Core Plank System

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HOLLOW - CORE PLANK FLOOR SYSTEM - SPOT CHECK

$d = 10''$ SPAN $= 31'-6'' = 31.5'$

$d_{TOPPING} = 2''$ CLEAR SPAN $= 29'$

ON PLANS, IT IS SPECIFIED THAT A 10" SPANDECK HOLLOW-CORE SYSTEM IS USED. AS PER MANUAL FOR THE DESIGN OF HOLLOW CORE SLABS, 2ND EDITION - PCI, THERE IS NO 10" DEEP SECTION OF SPANDECK.

ALTERNATE SYSTEM USED: DY-CORE
Section: 4'-0" x 10"
Weight: 81 psf

LOADS

DL = CEILING = 5 psf
MEP = 10 psf
FRAMING = 15 psf
PARTITIONS = 30 psf
60 psf
Self wt = 91 psf
151 psf

Assume:
1.) $f'_c = 5000$ psi
2.) low lax tendons
3.) $f_{pu} = 270$ ksi
4.) $f_{pe} = 0.55 f_{pu} = 148.5$

LL = Typ. = 100 psf

$w_u = 1.2(151\ psf(4')) + 1.6(100\ psf(4'))$
$= 1364.8$ plf
$= 1.36$ klf

$M_u = \dfrac{1.36(29)^2}{8} = 142.9$ 'k

Try (6)-0.6"ϕ PRETENSIONED STRANDS

$A_{ps} = 6(0.216) = 1.296\ in^2$

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$$f_{ps} = f_{pu}\left[1 - \frac{\gamma_p}{\beta_1}\left(\rho_p \frac{f_{pu}}{f'_c}\right)\right]$$

$\gamma_p = 0.28$ for "low-lax"

$\beta_1 = 0.80$

$d_p = 9.0"$

$\rho_p = \frac{A_{ps}}{bd_p^2} = \frac{1.296}{20(9)^2} = 0.0008$

$$= 270\left[1 - \frac{0.28}{0.8}\left(0.0008\,\frac{270}{5}\right)\right]$$

$= 265.9$ ksi

$$\omega_p = \frac{\rho_p f_{ps}}{f'_c} = \frac{0.0008(265.9)}{5} = 0.0425 < 0.36\beta_1 = 0.288 \therefore \text{ok}$$

$$a = \frac{A_{ps} f_{ps}}{0.85 f'_c b} = \frac{1.296\ (265.9)}{0.85(5)(20)}$$

$= 4.05"$

$$\phi M_n = \phi\left[A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right)\right] = 0.9\left[1.296(265.9)\left(9 - \frac{4.05}{2}\right)\right]$$

$= 2163.6$ in-k

$= 180.3'^{k} \simeq 142'^{k}$ ∴ OK BY JUDGEMENT

USE $6 - \frac{1}{2}"\phi$ STRANDS in 10" HOLLOW-CORE SYSTEM w/ 2" TOPPING

Appendix C

Precast Concrete Column Check

Precast Concrete Column

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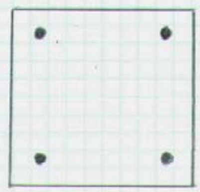
COLUMN SPOT CHECK



$h_t = 13'\text{-}0''$ LOCATION: EAST WING 1ST FLOOR

$A_C = 22 \times 22 = 484 \text{ in}^2$

TRY 4 - #8 bars
↳ $A_s = 3.16 \text{ in}^2$

REACTIONS FROM BEAMS ⇒ $R = 137.7^k$

$P = 137.7^k$ per floor $\cdot$ (4 floors)
$= 551^k$

$$\phi P_n = 0.85\,\phi\,[0.85 f'_c (A_g - A_{st}) + f_y A_{st}]$$
$$= 0.85\,(0.65)\,[0.85(4)(484 - 3.16) + 60(3.16)]$$
$$= 1006.0^k \quad > \quad P_u = 551^k$$
$$\therefore \text{ ok}$$

4-#8 bars would be sufficient to carry the applied loads.

Appendix D

Shear Wall Check

Precast Concrete Shear Wall

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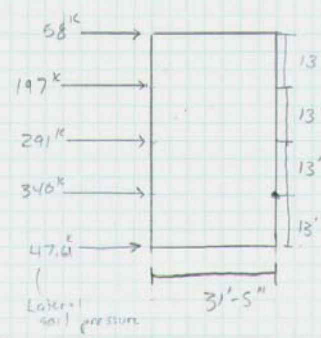
SHEAR WALL SPOT CHECK



* Seismic (E-W) CONTROL OVER WIND

trib w of shear wall

$W_{trib} = 20.75' + 21 + 21 + \frac{21}{2} = 73.25'$

length of shear wall

$L_{s.w.} = 31'-5"$

LATERAL PT LOADS

ROOF = 58 k

4^{TH} = 197 k

3^{rd} = 291 k

2^{nd} = 340 k

LATERAL SOIL PRESSURE
= 50 psf

$V_{SOIL} = 50\,psf(13')(73.25')$
$= 47.61^k$

OVERTURNING M:

$M_o = 340(0) + 291(13) + 197(26) + 58(39) - 47.61(13')$
$= 11,167'^k - 618.93'^k$
$= 10548.1'^k$

Assume $W_{s.w} = 12"$, width of shear wall not shown on drawings.

RESISTING Moment

$M_R = \left[150\,pcf\,(1')(31.4167)(52')\right]\left(\frac{31.4167}{2}\right)$
$= 3849'^k$

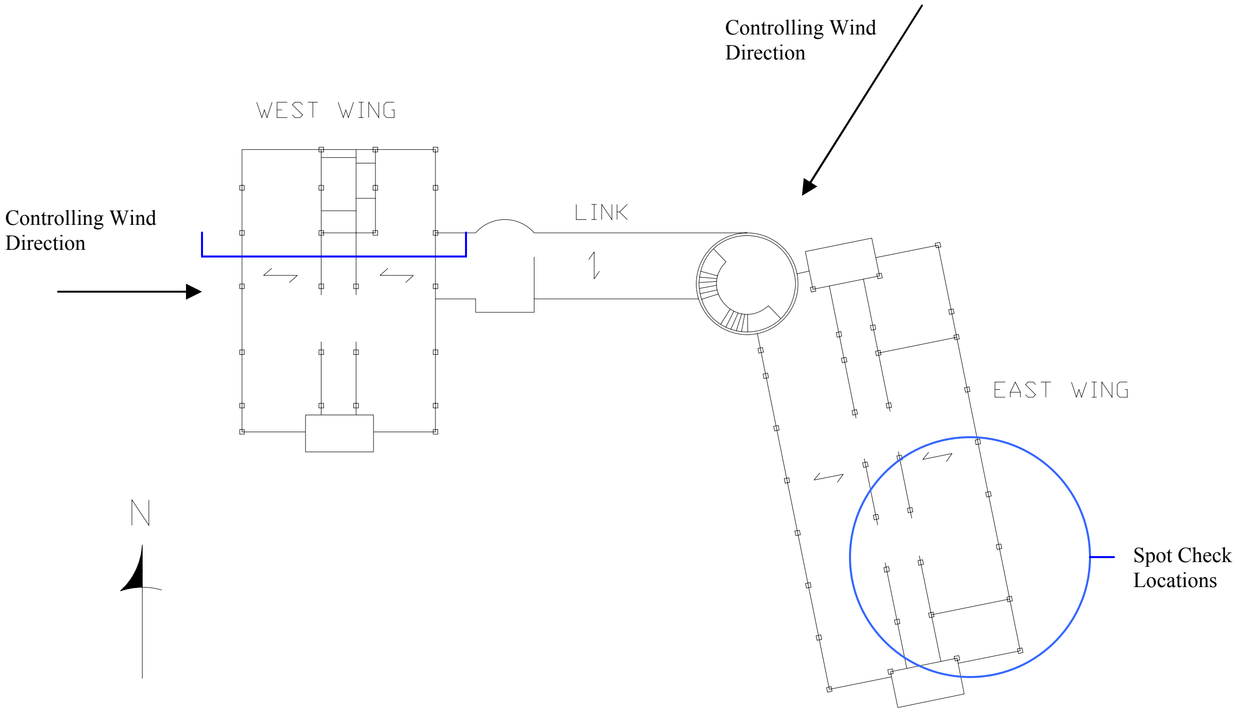
$T = \frac{10548.1 - 3849}{31.4167'} = 213.2^k$ = FORCE AT THE ENDS OF THE SHEAR WALLS

* ACTUAL WIDTH & REINFORCEMENT FOR SHEAR WALLS UNKNOWN.

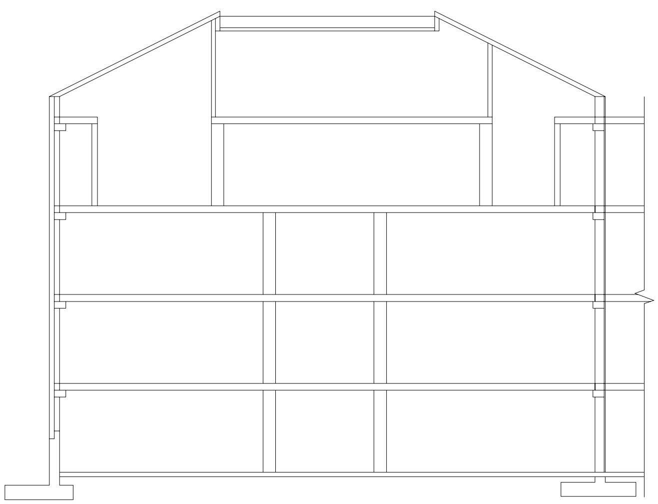
Appendix E

Building Sketches

Building Plan



Building Section – West Wing



Beam B1/ Column C1/Shear Wall – Spot Check Locations