SENIOR THESIS

TECHNICAL REPORT 1: EXISTING CONDITIONS

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Executive Summary

The objective of Technical Report 1 is to gain an understanding of the Judicial Center Annex through investigation of its existing conditions. The structural system will be summarized with calculations performed to obtain both gravity and lateral loads. Any loads presented on the drawings are used as a point of reference to validate any assumptions made. The pertinent design codes are summarized in addition to the materials to be used in the construction of the building.

The gravity loads are further verified by three spot checks performed on an interior column, a post tensioned slab panel and a doubly reinforced beam to provide a range of the concrete members involved in the building's structural system.

Lateral load calculations performed in accordance with ASCE 7-05 to come up with initial values for both wind and seismic loading. The seismic load was found to be with 10% of the base shear presented on the structural drawings and to control by a factor of 2 over wind. Further lateral analysis will be performed in Technical Report 3.

This report includes a variety of figures and hand calculations to reinforce points and support any results that are spoken of during the course of the paper.

Building Introduction

The Judicial Center Annex (JCA) is a modern addition to the existing Montgomery County Judicial Center. Located on the corners of Maryland Avenue and East Jefferson Street in downtown Rockville, MD the JCA is set provide a bold statement through both its architecture and engineering. Construction on the addition began this past April and is projected to take two years to complete.

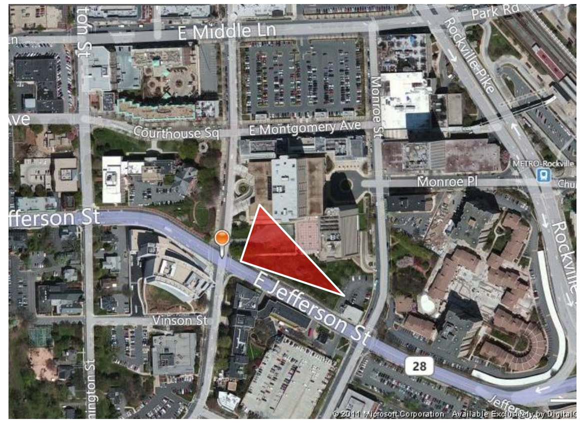


Figure 1: Site Location, Bing.com

The JCA will stand 114' tall at the crest of each of the four lanterns located on top of the building, so tall that local building codes needed waved for overall building height. Six stories rise above the ground, with garage and terrace levels located below grade, adding approximately 210,000 sq ft to the Judicial Center that will add 10 more courtrooms and administrative spaces among other spaces.

The project team, led by AECOM who provided both architectural and the majority of building engineering services, was able to achieve a unique look through both form and material. The East and West Elevations (Figure 2) are dominated by glazing, with the curtain wall that covers the East wrapping around the South corner. This curtain wall system is unique in that it uses glass stabilizing fins instead of traditional aluminum mullions, which enables an all glass look that when combined with the way the slab cantilevers out from the structure gives the illusion of the floors floating without structure. On the North the addition abuts against the original Judicial Center. The elements of the façade not covered in glass are sheathed in either a powder coated aluminum that has a reddish hue or architectural pre-cast panels that are more reminiscent of the exterior of the original building.

Figure 2: West Elevation

From the roof projects four lanterns which have a translucent linear glazing system allowing them to light up the night sky in a truly dramatic manner. The roof is also the site of two of the JCA's sustainable features that enabled it to achieve a LEED Gold Rating. The tops of each of the four lanterns are covered in photovoltaic panels, while green roofs cover much of the remaining roof.

Structural Overview

The JCA sits atop core-drilled concrete piers due to the rather poor soil conditions, all columns coming to bear atop a pier. The floor systems are post-tensioned slabs, with wide-shallow beams running one-way on the typical levels framing into cast-in-place concrete columns. The lateral system consists of five concrete shear walls, which rise continuously to the penthouse level, with some continuing to support the roof.

This building was designed as Occupancy III according to Sheet 1.S001. The reason for this is thought that the holding cells in the building subject it to the "Jail and detention facilities" clause or perhaps a courtroom has the ability for "more than 300 people to congregate." This Occupancy was assumed due to one of the previously mentioned reasons for purposes of coming up with importance factors in later calculations.

Foundations

Schnabel Engineering performed the geotechnical services on the JCA project. Reports indicated that for the purposes of shallow continuous wall footings the soil has a bearing capacity of 2000 psi, with any unsuitable conditions requiring excavation and replacement with lean concrete. Core-drilled piers ranging in diameter from 2.5' to 7' are located beneath every column and support much of the shallow wall footings. Grade beams are also used in several locations, specifically beneath the five shear walls. Using Grade beams beneath the shear walls, which not only have a larger self-weight as they rise continuously for most of the building height, but they will also exert additional compressive forces to into the foundations as they transfer the lateral loads of the building into the soil. In addition, typing into the Grade beams would help against uplift which will be investigated further in Technical Report 3. Grade beams vary from 24" to 42" in width and 36" to 72" in depth. The SOG is 5" thick and reinforced with WWF.

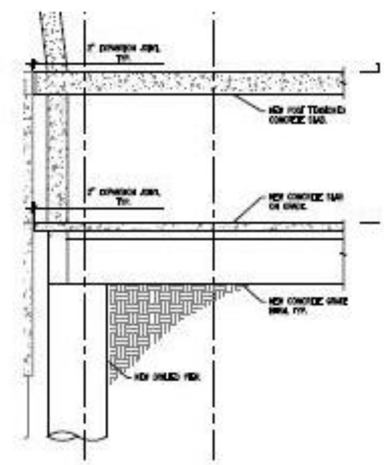


Figure 3: Column adjacent to existing Judicial Center resting on pier foundation

The garage level of the JCA is located 25' below grade. Though soil pressures on basement walls were not considered in this report they are a possible point of investigation in the future.

Floor Systems

As mentioned previously, the floor systems for the JCA utilize post-tensioning. The economy is achieved by greater span lengths being possible, with smaller slab depths. The typical floor system, which begins on the terrace level and extends to the 5th floor has both 8” and 9” slab depths, with wide-shallow beams running in the plan NS direction. These are denoted as continuous drop panels on the drawings and extend an additional 8” beneath the slab, with average width of 8’ that is not centered on the column lines. The beams are skewed towards the larger spans, which gives the idea that the added stiffness the beams provide allow for these slab depths to be achieved. In the slab gravity check which will be discussed later, ln/45, a typical slab span-to-depth ratio was not achieved for the longest spans of approximately 41’. It is here that it is thought the shallow beams are especially advantageous. The term continuous drop panel is thought to be utilized as the beams provided additional negative moment resistance around the columns. The bays are essentially uniform in parts of the building, with an alternating long/short/long span pattern. A small portion of the slab on the second level connecting to the existing building is lightweight concrete on metal deck on steel framing.

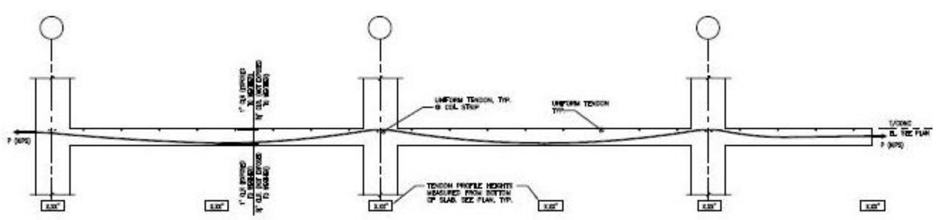


Figure 4: Section of PT slab showing tendon drape

The penthouse slab is 11” thick due to the larger loads present on this floor. There is an unreducible 150 psf mechanical live load present, as well as a 55 psf green roof dead load in several areas. The mechanical floor also features a floating four inch light weight concrete on metal deck isolation slab, to prevent mechanical equipment vibrations from affecting other parts of the building. The roof slab is 10” and features several large voids. This slab has post tensioned beams 36” x 24” typical for additional span stiffness in lieu of the wide-shallow beams.

Framing Systems

Cast-in-place columns rise from the garage level to the roof, with the four lanterns extending the extra fourteen feet with steel framing. The column concrete has a compressive strength of 7000 psi at the base, which is reduced to 5000 psi at level 2. Typical column sizes are 24”x24”

Lateral System

The lateral system of the JCA is comprised of five cast-in-place concrete shear walls, see Figure 5. The shear walls in the NS plan direction extend to the roof, while in the EW direction they reach the penthouse level. The walls extend continuously upward and feature large openings relying on link beams to maintain the load path from the various floor heights to the foundation. The walls are all 12” thick, and assuming a rigid diaphragm (reasonable for the thick concrete slabs), the walls will take load in proportion to their stiffness. Based upon their similar thicknesses, this stiffness will then be proportional to their length, meaning that in the EW direction shear walls 4 and 5 each take half the lateral force, while in the NS direction shear wall 1 takes half the load with shear walls 2 and 3 splitting the other half between them. These assumptions will be investigated more in depth through the usage of computer software in subsequent reports. Also worth investigation is how much of the load is transferred through frame action in the concrete slab and columns, and whether overturning will be an issue for the shear walls that are tied into grade beams.

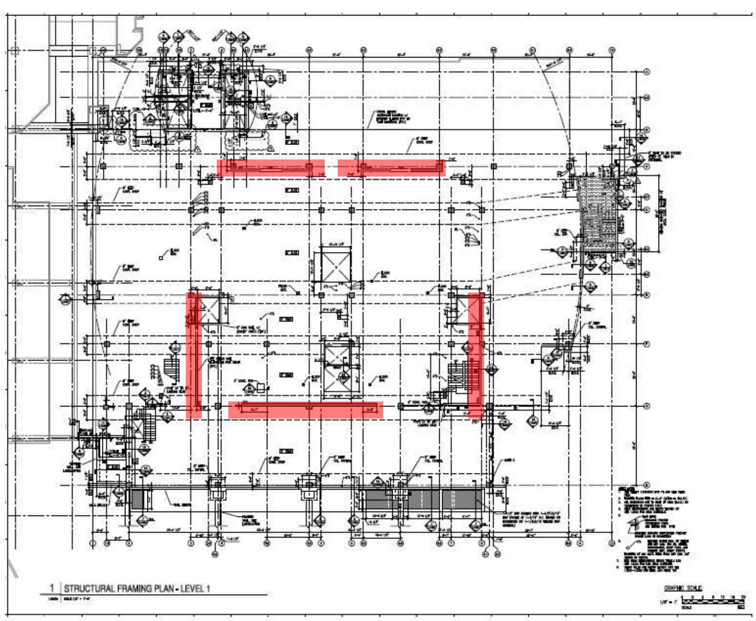


Figure 5: Shear Wall Layout

Roof Systems

The roof varies in height in several locations with the floor slabs described earlier in *Floor Systems*. The varying heights made snow drift a concern, and the large loads associated with the penthouse floor, which is the heaviest floor on the building, add a significant contribution to both seismic base shear and overturning. The green roof and pavers on the penthouse and upper roof levels lay overtop a hot applied fluid membrane.

Design Codes

The list of Major Codes and Standards on Sheet 1.S001 is as follows:

- 2009 International Building Code
- ACI 318-08
- AISC LRFD, 13th Edition, 2005
- AWS D1.1, D1.3, D1.4, Current Edition
- ASTM, Current Edition
- Steel Deck Institute Design Manual for Composite Deck, Form Decks and Roof Decks., 2007

These are the codes being used to complete the analyses performed in this report, with heavy usage of ASCE 7-05 (Minimum Design Loads).

Materials Used

Sheet 1.S001 was used as the reference for materials used in the construction of this project and summarized in Figure 6.

Concrete		
Usage	Weight	f'c (psi)
Column (Levels 2-Rf)	Normal	5000
Column (Levels G1-1)	Normal	7000
Floor Slab	Normal	5000
Wall Footings	Normal	3000
Beams	Normal	5000
Slab on Grade	Normal	4500
Walls, Piers, & Pilasters	Normal	5000
Drilled Piers	Normal	4000
LW Concrete Fill on Deck	Light	4000
Isolation Slab @ Penthouse	Light	4000

Steel		
Type	ASTM Standard	Grade
W Shapes	A992	
Plates, Angles, Channels	A36	
High-Strength Bolts	A325 or A490	
Anchor Rods	F1554	36
Tubes	A500	B
Pipes	A53 E or S	B
Reinforcing Steel	A615	60
Reinforcing Steel, Welded	A706	60
Roof Deck	A653	A - F
Floor Deck	A653	C, D, or E
Post-Tensioned Reinforcment	A416-96	

Masonry		
Type	ASTM Standard	F'm (psi)
CMU	C90	1500
Masonry Mortar	C270	
Grout	C476	
Aggregate	C404	

Figure 6: Summary of Materials Used

Gravity Loads

This section will describe how dead, live, and snow loads were calculated and compared to loadings given on the structural drawings. Three gravity checks were performed once the loadings were determined for an interior column, the typical long span for the post tensioned slab, and a doubly reinforced beam with full hand calculations available in Appendix A.

Dead and Live Loads

Dead Loads		
	Design	Student
Vegetated Roof	55	55
MEP/Celing	15	15
CMU Partitions	Actual Weight	91 pcf (Fully Grouted Assumption)

Figure 7: Summary of Dead Loads

The dead loads listed on 1.S001 shown in Figure 7 were used for the purposes of analyses. The non-load-bearing CMU walls were assumed to be fully grouted for the purposes of worst-case load calculations. The weight of the building was calculated neglecting voids in slabs and with an assumption of 10 psf for the steel lantern framing, which would not have much effect on the building weight were it too small an assumption. The total building weight which was used for the seismic calculations was in the order of 28000 kips.

Based upon ASCE 7-05 the 100 psf typical live load was found to be correct, possibly for different reasons than the designer decided for, and the 40 psf holding cell load was neglected in favor of using the 100 psf live load in all locations except for the mechanical penthouse and the roof loading.

Live Loads		
	Design	ASCE 7-05
Typical	100	80 (Corrider Above First Floor) + 20 (Partition) = 100
Holding Cells	40	-
Mechanical Penthouse	150	150
Roof	-	20

Figure 8: Summary of Live Loads

Snow Loads

The flat roof snow load was calculated via the method outlined in Chapter 7 of ASCE 7-05. A discrepancy arose as the importance factor, I, listed on the drawings had a value of 1.0, whereas the appropriate importance factor for an Occupancy III building is 1.1. This led to flat roof snow load value of 22 psf which differs from the calculated value of 23.1 psf. Curiously the design load is higher despite the lower importance factor which may be a result of a higher design ground snow load, though this isn't available on the drawings.

Flat Roof Snow Load		
pf = .7 CeCtIpg > 20*I		
Ce	1	ASCE 7-05 Tab. 7-2
Ct	1	ASCE 7-05 Tab. 7-3
pg	25	ASCE 7-05 Fig. 7-1
I	1.1	ASCE 7-05 Tab. 7-4
pf =	0	
20*I=	500	
pf =	22	

Figure 9: Snow Load Parameters and Flat Roof Calculation

The varying roof levels led to eight different drift calculations. The calculations can be see viewing Figure 10 and 11, with an accompanying hand check for one of the drifts performed in Appendix A.

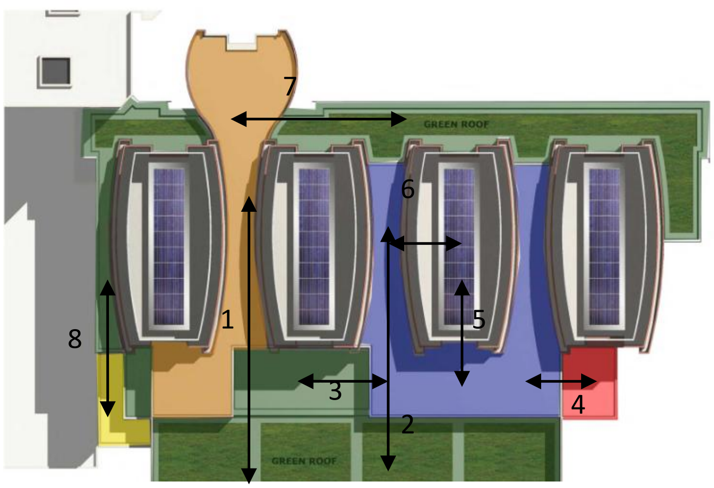


Figure 10: Rooftop Drift Diagram

Snow Drift		γ=	17.25						
	Lu	Ll	hc	hd Lee	hd Wind		hd (ft)	w (ft)	Max psf
Drift 1	130	50	16	3.79826	1.764815	3.79826	3.79826	15.19	65.52
Drift 2	93	30.33	18	3.238561	1.321269	3.238561	3.238561	12.95	55.87
Drift 3	70	50	18	2.810406	1.764815	2.810406	2.810406	11.24	48.48
Drift 4	70	20	21	2.810406	1.004234	2.810406	2.810406	11.24	48.48
Drift 5	70	20	14	2.810406	1.004234	2.810406	2.810406	11.24	48.48
Drift 6	38	12	14	2.016252	0.670866	2.016252	2.016252	8.07	34.78
Drift 7	21	147	16	1.385528	3.014862	3.014862	3.014862	12.06	52.01
Drift 8	83	24	52	3.06224	1.137649	3.06224	3.06224	12.25	52.82

Figure 11: Drift Spreadsheet

Gravity Checks

Column D4 Gravity Check

Column D4 is an interior column located in one of the more typical bays. The 24"x24" column extends the entire height of the building and was checked for strength considerations at the base where it has (8) #10 vertical bars and #3 ties at 16" on center. The column was cast with 7000 psi concrete at this level; though at level two it transitions to 5000 psi concrete and (8) #8 vertical bars. Loads were summed for each level and the column was assumed to be in a state of pure compression as it is in an interior column that is not intended to act as part of the lateral system.

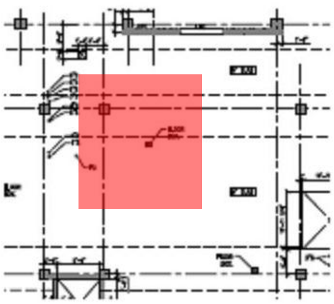


Figure 12: Column D4

The column's capacity was found to be more than adequate. It was calculated that it could carry approximately 2240 kips in a state of pure compression, while only having a demand of 1330 kips. It's possible that this means the assumption of pure compression was a poor one and that the column indeed sees moment through either the uneven spans or some incidental lateral load. The possibility of lateral load will be addressed in a later report when the system is explicitly modeled with computer software and lateral force distribution can be seen, see Appendix D for hand calculations.

Post Tensioned Slab Check

To investigate the post tensioned slab a Frame along line E was chosen. This represented a portion of the building with the most uniform bays, albeit with a long/short/long span pattern, and also the portion that had the largest spans and so would likely be the controlling factor in design of certain aspects.

The Equivalent Frame method was employed, as required by ACI 318-08 Chapter 18. In an effort to simplify the procedure dimensions were made uniform for the various spans, though only one bay was considered, and beams were centered on column lines. The first discrepancy came in the calculation of a proper slab depth given the span using h = ln/45. This resulted in a slab having to be 10 ½" to have a span to depth ratio where deflection would not be considered an issue. This was greater than the designed 9", though it has been heard of for a ratio of ln/50 to be used which gives a value of 9 ½". The thought is the wide-shallow beams allow the thinner slab thickness.

The analysis was carried out; finding the jacking force, P, to be equal to 583 kips. This seems consistent with the given jacking forces of 648 kips. The original jacking force was found to be 600 kips but was lowered as it over balanced the dead load, but the results are close enough to consider consistent. The number of tendons needed was found to be 24, though this number could not be found in the drawings. Drape values were almost identical between what were used in the hand calculations and what appear on the drawings. See Appendix E.

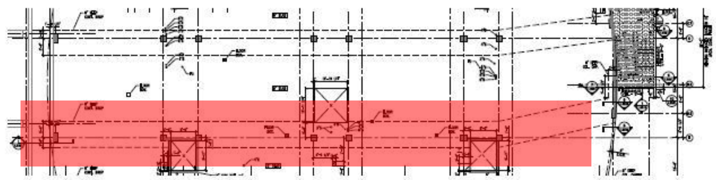


Figure 13: PT Slab Frame

Doubly Reinforced Beam Check

The final gravity spot check was performed on one of the few non-post tensioned beams to explore the different systems employed within the structure. Beam B5 was chosen, an exterior beam on the plan North side of the building that appears in levels 1-5. A non-load-bearing CMU wall sits atop the beam and two point loads can be seen to act on the beam due to the pre-cast façade being attached at these points.

The beam is 16”x24” with (3) #6’s top and bottom in the center of the span. A simply supported condition was assumed as no end bars were present in the beam schedule. The beam has an approximate span of 9.67’, and the 18 kip point load was assumed to act 1’ from the support as no dimension could be seen.

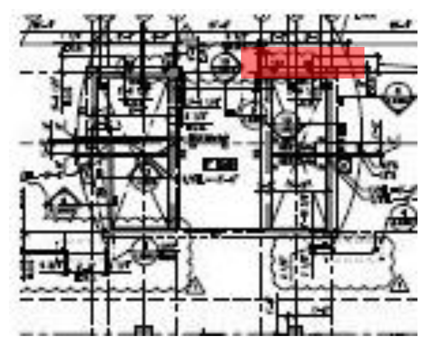


Figure 14: Beam Location

Assuming Case 2, where the bottom steel is yielding but the top is not, resulted in a moment capacity of 423 k-ft, far greater than the required 60 k-ft. Only flexure was considered, with the apparent overdesign possible as a result of either shear or torsion, but with a point load so close to the critical section shear is a good possibility and could be examined in subsequent reports. See Appendix F.

Lateral Loads

Wind (Appendix B) and seismic (Appendix C) loads were calculated using prescribed methods from ASCE 7-05. These preliminary hand calculations give some insight into the behavior of the building and the controlling factors, which will be further explored in Technical Report 3.

Wind Loads

Method 2 Main Wind Force Resisting System (MWRFS) procedure from ASCE 7-05 chapter 6 was used in the calculation of the wind forces the building will be subjected to. To simplify the calculations, the maximum roof height was assumed to be 100', though the lanterns extend to 114'. This is likely a good assumption on the grounds the lanterns themselves represent a small surface area which wind load can be applied to in addition to making calculations simpler. Additionally the floor plan was assumed rectangular and an idealized building width and length were determined to get values of L and B.

Calculations were performed with the aid of Microsoft Excel, with a hand check of both the velocity pressure and design wind pressure. The base shear in the NS direction was found to be 290 k and the EW direction had a base shear of 280 kips. As no wind pressures were listed to compare to, these values are assumed accurate.

The components and cladding wind pressures were not considered in this report but would need to be looked at going forward in the analysis as the wind loads are transferred through the cladding into the floor slabs and into the shear walls before being transferred to the foundations.

Design Wind Pressure N/S							
		Distance	Wind Pressure	Internal Pressure		Net Pressure	
				(+) Gcpi	(-) Gcpi	(+) Gcpi	(-) Gcpi
Windward	1st	0	7.86	3.61	-3.61	4.24	11.47
	2nd	14	7.86	3.61	-3.61	4.24	11.47
	3rd	29.5	9.59	3.61	-3.61	5.98	13.21
	4th	46	10.89	3.61	-3.61	7.28	14.50
	5th	62.5	11.85	3.61	-3.61	8.24	15.47
	Penthouse	79	12.76	3.61	-3.61	9.15	16.38
	Roof	100	13.65	3.61	-3.61	10.03	17.26
Leeward	All	-	-8.53	3.61	-3.61	-12.14	-4.92
Side Walls	All	-	-11.94	3.61	-3.61	-15.55	-8.33
Roof		0 - 50	-17.74	3.61	-3.61	-21.35	-14.13
		> 50	-14.19	3.61	-3.61	-17.80	-10.58

Design Wind Pressure E/W							
		Distance	Wind Pressure	Internal Pressure		Net Pressure	
				(+) Gcpi	(-) Gcpi	(+) Gcpi	(-) Gcpi
Windward	1st	0	7.86	3.61	-3.61	4.24	11.47
	2nd	14	7.86	3.61	-3.61	4.24	11.47
	3rd	29.5	9.59	3.61	-3.61	5.98	13.21
	4th	46	10.89	3.61	-3.61	7.28	14.50
	5th	62.5	11.85	3.61	-3.61	8.24	15.47
	Penthouse	79	12.76	3.61	-3.61	9.15	16.38
	Roof	100	13.65	3.61	-3.61	10.03	17.26
Leeward	All	-	-7.85	3.61	-3.61	-11.46	-4.23
Side Walls	All	-	-11.94	3.61	-3.61	-15.55	-8.33
Roof		0 - 50	-16.17	3.61	-3.61	-19.78	-12.56
		> 50	-14.94	3.61	-3.61	-18.55	-11.33

Figure 15: Design Wind Pressures

Wind Force (NS)								
		Trib Below		Trib Above		Story Force	Story Shear	Overturning Moment
	Height	Ht	Area	Ht	Area			
1st	0	0	0	7	1050.00	17.20	290.26	0.00
2nd	14	7	1050	7.75	1162.50	36.25	273.05	507.52
3rd	15.5	7.75	1162.5	8.25	1237.50	43.49	236.80	1283.00
4th	16.5	8.25	1237.5	8.25	1237.50	48.06	193.31	2210.64
5th	16.5	8.25	1237.5	8.25	1237.50	50.45	145.25	3152.83
Penthouse	16.5	8.25	1237.5	10.5	1575.00	59.88	94.81	4730.73
Roof	21	10.5	1575	0	0.00	34.92	34.92	3492.37
							Base Shear (k)	290.26
							Total Overturning Moment (k-ft)	15377.09

Wind Force (EW)								
		Trib Below		Trib Above		Story Force	Story Shear	Overturning Moment
	Height	Ht	Area	Ht	Area			
1st	0	0	0	7	1050.00	16.49	280.02	0.00
2nd	14	7	1050	7.75	1162.50	34.74	263.53	486.39
3rd	15.5	7.75	1162.5	8.25	1237.50	41.85	228.79	1234.70
4th	16.5	8.25	1237.5	8.25	1237.50	46.37	186.94	2132.96
5th	16.5	8.25	1237.5	8.25	1237.50	48.76	140.57	3047.29
Penthouse	16.5	8.25	1237.5	10.5	1575.00	57.96	91.81	4579.14
Roof	21	10.5	1575	0	0.00	33.85	33.85	3384.91
							Base Shear (k)	280.02
							Total Overturning Moment (k-ft)	15145.41

Figure 16: Design Wind Forces

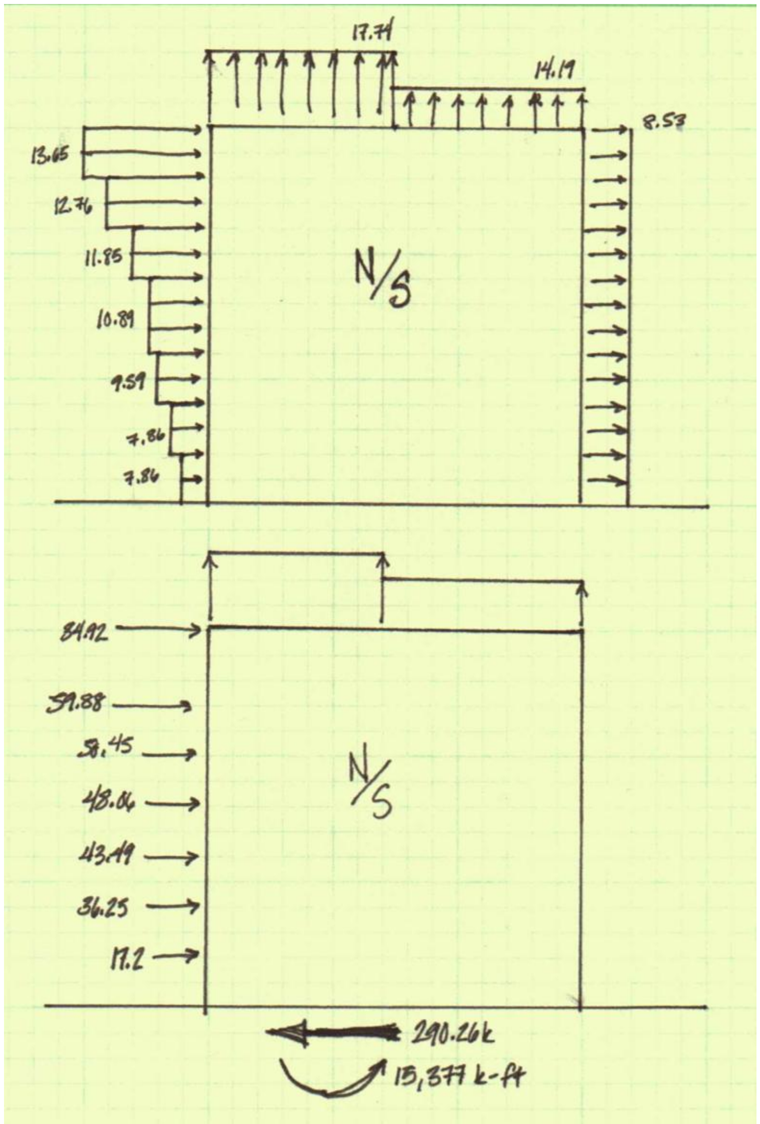


Figure 17: N/S Wind Pressure and Wind Force Diagrams

Seismic Loads

The seismic loads were calculated based upon the Equivalent Lateral Force Method outlined in ASCE 7-05 Chapters 11 and 12. The fundamental period was calculated via equation 12.8-7 in ASCE 7-05 and modified by the Cu coefficient found in Table 12.8-1 as the low base shear listed on the drawings did not seem achievable given the weight of the structure if the period was calculated via the method for shear walls. The full height of the building was also used which resulted in a base shear of 600 kips, within 10% of the 560 kips listed on the drawings lending weight to the earlier assumptions.

The seismic is seen to control by a factor of 2 over wind in this case, which may be the reason why only the seismic base shear is shown on the structural drawings. The inertial response of the building will be directly related to the mass, which means the majority of the seismic forces originate in the slabs which transfer the forces directly to the shear walls.

Seismic Forces						
Level	Story Ht (ft)	Story Weight (k)	Cvx	Story Force (k)	Shear Shear (k)	Overturning Moment (k-ft)
1	0	3584.8713	0	0	600.00	0.00
2	14	4024.472	0.04259	25.55	600.00	357.76
3	29.5	4108.9654	0.091818	55.09	574.45	1625.18
4	46	4132.2627	0.144067	86.44	519.36	3976.25
5	62.5	4121.3877	0.195177	117.11	432.91	7319.13
PentHouse	79	5698.8439	0.352365	211.42	315.81	16702.10
Roof	98	2466.4583	0.173983	104.39	104.39	10230.21
					Base Shear (k)	600.00
					Total Overturning Moment (k-ft)	40210.63

Figure 18: Seismic Forces

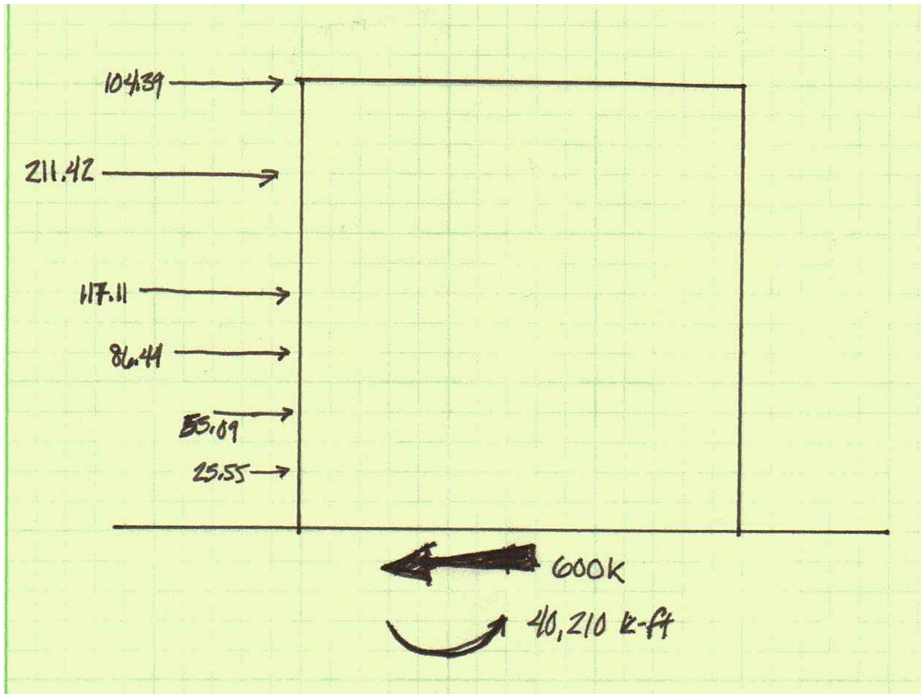


Figure 19: Seismic Force Diagram

Conclusion

Technical Report 1 gave an overview of the existing conditions of the Judicial Center Annex's structure. Various structural elements, including the foundations and floor slabs were summarized with the intent to give a good understanding of how the current structure is intended to be built and act.

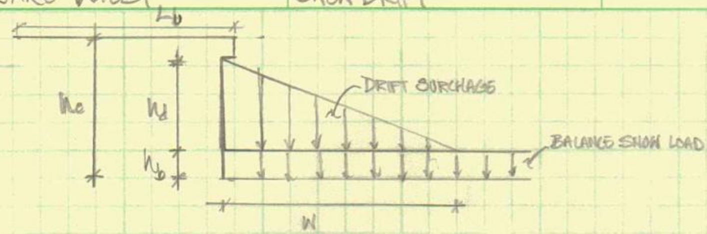
Gravity and lateral loads were also determined over the course of this report using both ASCE 7-05 and the structural drawings as resources. The dead, live, and snow loads were compiled and compared to the loads used with positive results. The lateral loads, calculated through methods outlined in ASCE 7-05 compared favorably to the numbers that were present on the drawings.

Gravity checks were performed on three members; a column, slab panel, and beam. These members were chosen as they represented a wide range of what is present in the JCA's structural system, and all three are different components. While the checks may have not considered every option they in general showed that the members were designed appropriately. The added challenge of the slab configuration combined with post-tensioning made for an interesting analysis.

Seismic loads were found to control by a factor of 2, and were found within 10% of the base shear reported on the structural drawings indicating the validity of the assumptions and methods used. This report presented many valid points, answering questions about the design and raising new ones to be investigated in further reports.

Appendix A: Snow Load

JAKE WIEST SNOW DRIFT



$h_d \le h_c,\ w = 4h_d$

$h_d > h_c,\ w = \frac{4h_d^2}{h_c}$ + $h_d = h_c$

$P_d = h_d \gamma$

$h_b = P_f/\gamma = 22/17.25 = 1.28'$

NOTE:
WINDWARD DRIFT
$h_d \rightarrow \frac{3}{4}h_d$ FROM FIG. 7-9

$\gamma = .13p_g + 14 < 30\,pcf$
$= .13(25) + 14$
$= 17.25\,pcf$

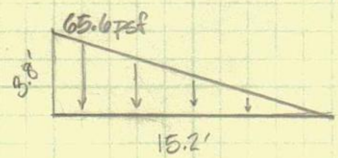
$h_d = .43\sqrt[3]{L_u}\,\sqrt[4]{p_g+10} - 1.5$

$L_u = 130$
$L_L = 50$

$h_d = .43\sqrt[3]{130}\,\sqrt[4]{25+10} - 1.5$
$= 3.80'$

$h_{d_{WINDWARD}} = .75(.43\sqrt[3]{50}\sqrt[4]{35} - 1.5)$
$= 1.76'$

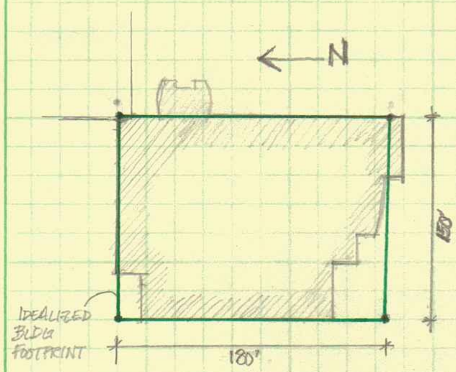
$h_d = 3.8' < h_c \quad w = 4h_d = 15.2'$



$\gamma = 17.25\,P/ft^3 \rightarrow 17.25 \times 3.8$
$= 65.6\,psf$

Appendix B: Wind Load

JAKE WIEST | WIND



*WHILE THE HIGHEST PT. ON THE BLDG IS 114', THE MAJORITY OF THE ROOF IS @ 100', WHICH WILL BE USED FOR SIMPLIFICATION

$V = 90$ mph FIG. 6-1
$I = 1.15$ TAB. 6-1
EXPOSURE B
$K_z = .99$ TAB 6-3
$K_{zt} = 1.0$
$K_d = .85$ TAB 6-4 MWFRS

VELOCITY PRESSURE

$q = .00256 K_z K_{zt} K_d V^2 I$

$= .00256(.99)(1.0)(.85)(90^2)(1.15)$

$= 20.1$ psf FOR $h = 100$ ft

(REST ON EXCEL)

T_a (FROM SEISMIC) $= .698$

$f = 1/T = 1/.698 = 1.4 > 1\text{ Hz}$

RIGID STRUCTURE

$G = 0.85$

$GC_{pi} = \pm 0.18$: FULLY ENCLOSED

DESIGN WIND PRESSURE

$P = qGC_p - q_i(GC_{pi})$

C_p

WINDWARD - $C_p = .8$
SIDEWALL $C_p = -.7$
LEE NORMAL 180' $150/180 = .83 = -0.5$
NORMAL 150' $180/150 = 1.2 = -0.46$

WINDWARD P @ 100'

$P = 20.1(.85)(.8) - 20.1(\pm .18)$

$= 13.7 \pm 3.6$ psf

(REST ON EXCEL)

ROOF C_p: $100/150 = .67$
$100/180 = .56$

0 to $h/2 = -1.04$
$-.948$
$> h/2 = -.832$
$-.846$

Wind Load Criteria		
Gcpi	0.18	ASCE 7-05 Fig. 6-5
Exposure	B	ASCE 7-05 6.5.6.3
V	90 mph	ASCE 7-05 Fig. 6-1C
I	1.15	ASCE 7-05 Tab 6-1
Kzt	1	ASCE 7-05 6.5.7.1
Kd	0.85	ASCE 7-05 Fig. 6-4

Figure 20: Wind Design Criteria

Velocity Presssure Coefficients (Kz) and Velocity Pressures (qz)			
	Height	Kz	qz
1st	0	0.570	11.55
2nd	14	0.570	11.55
3rd	29.5	0.696	14.11
4th	46	0.790	16.01
5th	62.5	0.860	17.43
Penthous	79	0.926	18.77
Roof	100	0.990	20.07

Figure 21: Velocity Pressure Coefficients and Velocity Pressures

Appendix C: Seismic Load

JAKE WIEST SEISMIC

- OCCUPANCY CATEGORY III – IMPORTANCE FACTOR = 1.25
- SITE CLASS – D

$F_a = 1.6$ (TABLE 11.4-1)

$F_v = 2.4$ (TAB. 11.4-2)

$S_s = .156$
$S_1 = .051$
(VALUES FROM USGS SEISMIC HAZARD CURVES + UNIFORM HAZARD RESPONSE SPECTRA) EST. VALUES FROM MAPS .16 & .05 RESPECTIVELY)

$S_{MS} = F_a S_s = .156(1.6) = .2496$

$S_{M1} = F_v S_1 = .051(2.4) = .1224$

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(.2496) = .1664$ SDC = A

$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(.1224) = .0816$ SDC = B ← CONTROLS

T. 12.8-2 ∴ $C_t = .02$
$x = .75$

$T_a = C_t h_n^x = .02(114)^{.75} = .698$ $T_L = 8s$ $T = C_u T_a = 1.7(.698) = 1.19$

$C_s = \frac{S_{DS}}{(R/I)} = \frac{.166}{(4/1.25)} = .052s < C_s = \frac{S_{D1}}{T(R/I)} = \frac{.0816}{1.19(4/1.25)} = .0214$

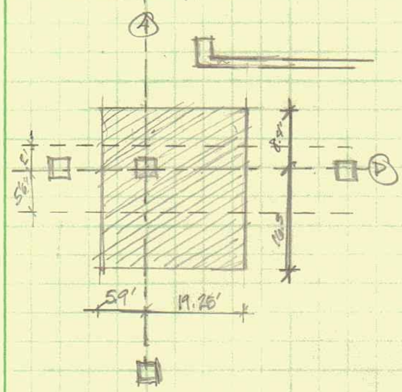
$C_s = .0214$

$V = WC_s = 28000 \times .0214 = 600k$

Appendix D: Gravity Check, Column

JAKE WIEST | GRAVITY CHECK

COLUMN D-4



TRIB AREA = 626'
INFL. AREA = 2504'

ASSUMED LL = 80psf + 20psf

DESIGN LL = 100psf TYP.
150psf MECH. P.HOUSE
40psf HOLDING CELLS

LOADING FOR TERRACE - 5th
SLAB = 9" - 150 × 9/12 = 112.5psf
SHALLOW BM = 150 × 8/12 × (7.5 × 25.2) = 18.9k
LL = 100psf

5th $LL_R = .25 + \frac{15}{\sqrt{2504}} = .55$

$P_D = 112.5(626) + 18.9 = 89.3k$
$P_L = 100(626) \times .55 = 34.4k$

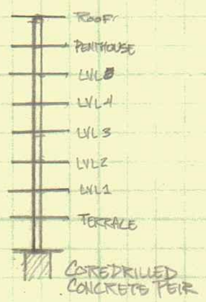
4th
$LL_R = .25 + \frac{15}{\sqrt{2 \times 2504}} = .46$

$P_D = 89.3k$
$P_L = 100 \times .46 \times 626 = 28.8k$

3rd
$LL_R = .25 + 15/\sqrt{3 \times 2504} = .42$

$P_D = 89.3k$
$P_L = .42 \times 100 \times 626 = 26.3k$

2ND - TERRACE
$LL_R = .40$
$P_D = 89.3k$
$P_L = .4 \times 100 \times 626 = 25k$



PENTHOUSE
SLAB → 11/12 × 150 = 137.5psf + $\underbrace{4/12}_{\text{FLOATING LW SLAB}}$ × 110 = 36.7psf
18.9k → SHALLOW BM
LL = 150psf

$P_D = 174.2 \times 626 + 18.9 = 127.9k$
$P_L = 150 \times 626 = 93.9k$

ROOF (UPPER ROOF NEGLECTED, SLAB OPENING NEGLECTED AS WELL TO BALANCE)
ROOF SNOW LOAD USED OVER ROOF LIVE

SLAB → 10/12 × 150 = 125psf
BM → 14/12 × 150 × (16/12 × 25.2) = 5.9k

$P_D = .125 \times 626 + 5.9 = 84.2k$
$P_L = 22 \times 626 = 13.8k$

$P_u = 1.2(748) + 1.6(272) = 1333k$

ASSUMING PURE COMPRESSION

D4 → 24×24, $f'_c = 7000psi$
(8) #10's, GR. 60

$\phi P_n = .85 f'_c (bh - \sum A_{si}) + \sum A_{si} f_{si}$

$= .65 \times .85(7000)(24 \times 24 - 8 \times 1.27) + 1.27 \times 60,000$

$= 2238k > 1333k$

$A_{STmin} = .01A_g = 5.76 in^2$

$A_{STmax} = .08A_g = 46 in^2$

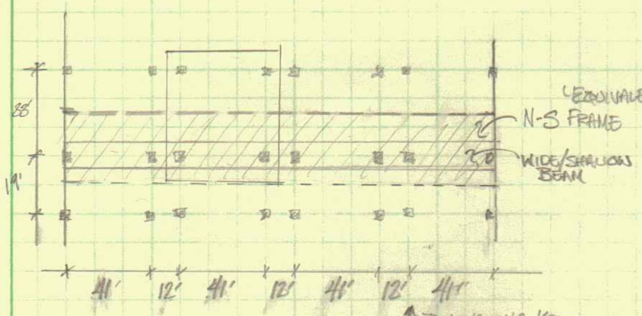
$A_{ST} = 10.2 in^2$ ✓ ACI 318-08 (10.9.1)

Appendix E: Gravity Check, Slab

JAKE WIEST | GRAVITY CHECK

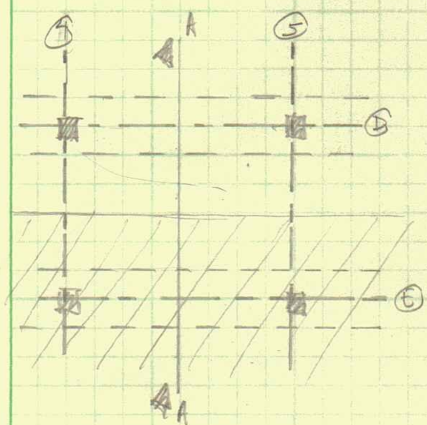
IDEALIZED SLAB PLAN (LEVELS TERRACE–5)

COLUMNS – 24"x24"
FLR TO FLR – 16'

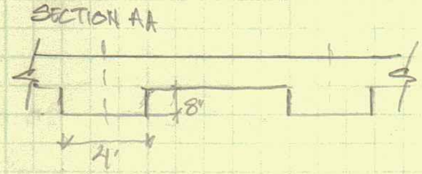


FOR SIMPLIFICATION DIMENSIONS MADE UNIFORM + BMS CTR'D ON COLUMN LINE

DIMENSIONS KEPT UNIFORM FOR SIMPLIFICATION PURPOSES



SECTION AA



SLAB THICKNESS

$h = \frac{l_n}{45} = \frac{39 \times 12}{45} = 10.4 \rightarrow 10\frac{1}{2}"$ DESIGNER USED 9"

IN SOME INSTANCES SPAN TO DEPTH FOR PT → $\frac{l_n}{50} = 9\frac{1}{2}"$, CLOSER TO 9"

USE 9" $\quad W_D = \frac{9}{12} \times 150 = 112.5$ psf

W_D of BM' $= \frac{8}{12} \times 150 = 100$ psf (OVER 8') $\rightarrow \frac{100 \times 8}{27} = 30$ psf

$W_L = 100$ psf

$\frac{LL}{DL} \; \frac{100}{142.5} = .7 \leq \frac{3}{4}$

FULL LIVE LOAD (NO PATTERNS)

APPROX FLEXURAL STIFFNESS OF COLUMNS

$K_c = \frac{4E_cI_c}{L-2t}$ $E_c = E_s = 1.0$ $I_c = \frac{24(24)^3}{12} = 27{,}648 \text{ in}^4$

$L = 16 \times 12$

$= \frac{4(27648)}{16\times12-2(9)} = 636\,E_c \text{ in-lb}$

TORSIONAL STIFFNESS OF WIDE/SHALLOW BM + SLAB

IN THIS CALC WIDE SHALLOW BM NEGL.

$C = (1-.63\,9/24)(9^3(24)/3) = 4454$

$K_t = \frac{9E_c\,4454}{(19\times12)[1-24/(19\times12)]^3} + \frac{9E_c\,4454}{(33\times12)[1-24/(33\times12)]^3} = 368\,E_c \text{ in-lb}$

SLAB STIFFNESS

$K_s = \frac{4E_{cs}I_s}{l_n - c/2}$ $I_s = I_{BM} + I_s - I_{s\,(\text{WHERE BM WAS})}$

$= \frac{(8\times12)(17^3)}{12} + \frac{(26\times12)(9^3)}{12} - \frac{(8\times12)(9^3)}{12}$

$= 52426 \text{ in}^4$

$K_{s_{12'\text{SPAN}}} = \frac{4E_c(52426)}{(12\times12)-24/2} = 1589\,E_c$

$K_{s_{41'}} = \frac{4E_c(52426)}{(41\times12)-24/2} = 437\,E_c$

DISTRIBUTION FACTORS

4 → ← 5

$D_f = \frac{K_s}{\Sigma K}$ $\Sigma K = K_{ec} + K_{sl} + K_{sr}$ $K_{ec} = \frac{1}{\Sigma K_c} + \frac{1}{K_t} = \frac{1}{2(636)} + \frac{1}{368} = 285\,E_c$

$D_F = \frac{437}{437+285+1589} = .189$

$\leftarrow 4 \quad 5 \rightarrow$

$D_F = \frac{1589}{437+285+1589} = .688$

SECTION PROPERTIES (ACCOUNTING FOR WIDE/ALUM BM)

$A = bh = 12 \times 18(9) + 8 \times 12(17) = 3576 \text{ in}^2$

$S = bh^2/6 = \frac{(12 \times 18)(9)^2}{6} + \frac{(8 \times 12)(17)^2}{6} = 7540 \text{ in}^3$

DESIGN PARAMETERS

AT TIME OF JACKING
$f'_{ci} = 3000$ psi

COMP: $= .6 f'_c = 1800$ psi
TENS $= 3\sqrt{f'_c} = 164$ psi

AT SERVICE
$f'_c = 5000$ psi

COMP: $.45 f'_c = 2250$ psi
TENS: $6\sqrt{f'_c} = 424$ psi

AVG. PRECOMPRESSION LIMITS

$P/A = 125$ psi MIN
$= 300$ psi MAX

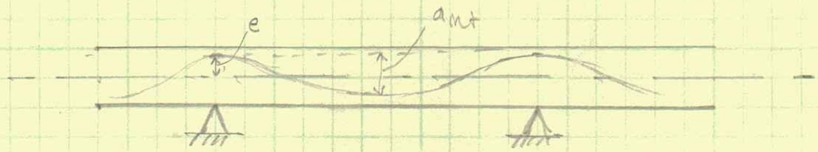
COVER REQ'D

RESTR: 3/4" BOTTOM
UNRESTR: 1.5" BOTTOM
3/4" TOP

TARGET LOAD BALANCE

$3/4\, w_D = 142.5(.75) = 107$ psf

TENDON PROFILE



LOCATION FROM BOTTOM
INT. SUPPORT: 16"
INT. SPAN: 1"

$a_{int} = 16-1 = 15"$

No e @ end

$a_{end} = (16+8.5)/2 - 1" = 11.25$

ASSUMED ENDSPAN

FORCE REQ'D TO BALANCE 75% OF DL.

$W_D = 107 \times 26' = 2.8$ klf

FORCE NEEDED IN TENDONS

$P = w_b L^2 / 8a_{end} = 2.8(41)^2 / 8(\frac{11.25}{12}) = 628$ k

ASSUMING
$d = 1/2"$, 7 WIRE-STRAND
$A = .153 in^2$
$f_{pu} = 270$ ksi
$f_{py} = 243$ ksi
$f_{pe} = 159$ ksi

$.153 \times 159 = 24.3$ k/tendon

$628/24.3 = 25.8$ TENDONS → 26 TENDONS

$P_{ACTUAL} = 631.8$ k

$W_B = \frac{631.8}{628}(2.8) = 2.82$ k/ft

$\frac{P_{ACTUAL}}{A} = \frac{631.8 \times 1000}{3576} = 177$ psi > 125 psi < 300 psi ∴ ok

INTERIOR
$P = 2.8(41^2) / (8 \times 16/12) = 442$ k < 632 k ✓

$W_b = 632(8)(16/12)/41^2 = 3.82$ klf

$W_D = 3.7$ klf < 3.82 NO GOOD

BALANCE 70% = $.75(142.5)(26) = 2.6$ klf

$P = 2.6(41^2)/8(11.25/12) = 583$ k

USE 24 TENDONS

$W_b = 583(8)(16/12)/41^2 = 3.52$ ✓

INTERIOR = $P = 2.6(41^2)/(8 \times 16/12) = 409 < 583$

$\frac{583 \times 1000}{3576} = 163$

FEM = $wl^2/12$

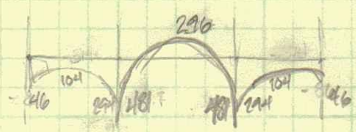
DL MOMENT

$\frac{3.7 \times 41^2}{12} = 518$ k-ft

$\frac{3.7 \times 12^2}{12} = 44.4$ k-ft

		4		5		
	.688	.688	.189	.189	.688	.688
FEM	+44.4	+44.4	-518	+518	-44.4	+44.4
DSTR.	+80.5 →	15.25			-5.25 ←	-80.5
		-443		443		
	152.5 ←	+305	+83.7	-83.7	-305 →	-157.5
	-104.9 →	-53	41.9	+41.9	53 ←	104.9
		+293.8		-293.8		
	101 ←	202.1	56	+56	+202.1 →	101
	-70 →	+35	28	-28	35 ←	-70
		-251		+251		
	86 ←	-172	47	-47	-172 →	-86
	-43 →	-28	-24	24	22 ←	+43
	-45.9	+294	-481	+481	-294	+45.7

$M^+ = \frac{3.7(41^2)}{8} - 480.7 = 296$ k-ft



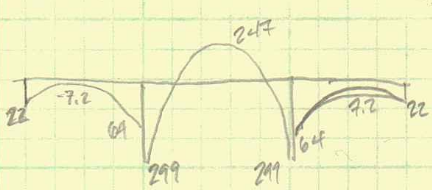
LL MOMENT

$\frac{2.6 \times 41^2}{12} = 364$ k-ft

$\frac{2.6 \times 12^2}{12} = 31.2$ k-ft

	4		5		
.688	.688	.189	.189	.688	.688
-31.2	+31.2	-364	+364	-31.2	+31.2
21.5 →	16.7			-10.7 ←	-21.5
	+322.1		+322.1		
+111 ←	+222	+60.9	-60.9	222 →	+111
76 →	-38	30.5	-30.5	+38 ←	+76
	276		-276		
-95 ←	-190	-52	52	190 →	95
48 →	28	26	-26	-24 ←	-46
-22	64	-299	299	-64	22

$M^+ = \frac{2.6(41^2)}{8} - 299 = 247$



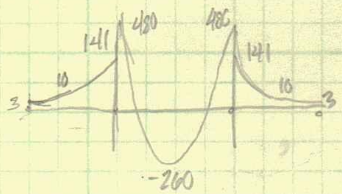
BALANCING MOMENT

$W_b = -3.52\ klf$

$\frac{wl^2}{12} = \frac{-3.52(41^2)}{12} = -493$

$\frac{-3.52(12^2)}{12} = -42$

	4		5		
.688	.688	.189	.189	.688	.688
-42	+42	-493	+493	-42	+42
29	15			-15	-29
	-436		436		
150	300	82	-82	-300	-150
-103	-52	-41	41	52	-103
	289		-289		
-100	-199	-55	55	199	100
69	35	28	-28	-35	-69
3	141	-480	480	-141	3



$M = \frac{3.52(41)^2}{8} - 480 = 260\ ft\,k$

STAGE 1 (DL + Pf) ONLY CONSIDERING 41' SPAN

MIDSPAN

$f_{top} = (-296 + 260)\frac{12 \times 1000}{7540} - 163 = -220$ COMP < 1800 ok

$f_{Bot} = (296 - 260)\frac{12 \times 1000}{7540} - 163 = -105$

SUPPORT

$f_{top} = (481 - 480)\frac{12 \times 1000}{7540} - 163 = -161$ COMP < 1800 ok

$f_{bottom} = (480 - 481)\frac{12 \times 1000}{7540} - 163 = -164$

STAGE 2 (DL + LL + Pf)

MIDSPAN

$f_T = (-296 - 247 + 260)\frac{12 \times 1000}{7540} - 163 = -613 < 2250\ psi$ ok

$f_B = (296 + 247 - 260)\frac{12 \times 1000}{7540} - 163 = 287 < 424$

SUPPORT

$f_T = (481 + 299 - 480)\frac{12 \times 1000}{7540} - 163 = 314 < 424\ psi$

$f_B = (-481 - 299 + 480)\frac{12 \times 1000}{7540} - 163 = -640 < 2250\ psi$ ok

Appendix F: Gravity Check, Beam

JAKE WIEST | GRAVITY CHECK

BEAM 85 16"x24" (3)#6 TOP & BOTTOM

SPAN = 9.67'

LL = 100psf × 5 = .5 klf

SLAB = 5' × $\frac{9"}{12}$ × 150 = .56 klf

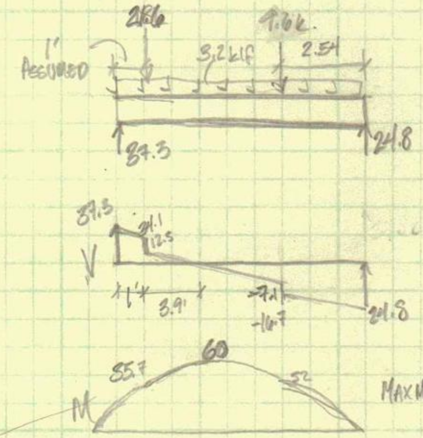
W_u = 1.2(.56 + .4 + 1) + 1.6(.5) = 3.2 klf

SELF WT = .4 klf

CMU WALL ≈ 16' TALL, 8" THICK ASSUME FULLY GROUTED

= 16 × $\frac{8}{12}$ × 94 = 1 klf

2 pt. LOADS, 18k & 8k



MAX MOMENT ≈ MIDSPAN = 60 k-ft

f'_c = 5000 psi

f_y = 60 ksi

ASSUME CASE 2 $\varepsilon_s > \varepsilon_y$ & $\varepsilon'_s < \varepsilon_y$

$.85 f'_c \beta_1 c + A'_s \cdot \frac{\varepsilon_u}{c}(c - d') \cdot E_s = A_s f_y$

$.85(5000)(.8)c + 1.32 \cdot \frac{.003}{c}(c - 1.625)(29000000) = 1.32(60{,}000)$

$3400c + \left[114840 - \frac{186615}{c}\right] = 79200$

$3400c^2 + 35640c - 186615 = 0$

$c = \frac{-35640 \pm \sqrt{35640^2 + 4(3400)(186615)}}{2(3400)}$

C = 3.83"

$\varepsilon_s = \frac{.003}{3.83}(22.4 - 3.83) = .015 > .005$ ✓

$$\varepsilon_s' = \frac{.003}{3.83}(3.83 - 1.75) = .00165 < .005 \checkmark$$

$$M_n = A_s' \, .003 \, \frac{c - d'}{c} E_s (d - d') + .85 f_c' \cdot a \cdot b \left(d - \frac{\beta_1 c}{2}\right)$$

$$= 1.32(.003)\left(\frac{3.88 - 1.75}{3.89}\right)(29000)(22.4 - 1.75) + .85(5)(3.88 \cdot .8)(16)\left(22.4 - \frac{.8 \cdot 3.83}{2}\right)$$

$$= 5634 \text{ k-in}$$

$$= 470 \text{ k-ft}$$

$$= .9(470) = 423 > 60 \text{ k-ft}$$