Ingleside AT KING FARM

701 King Farm Blvd.
Rockville, MD 20852

Technical Assignment # 1

Structural Concepts and Existing Conditions Report

29 September 2008

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Senior Thesis 2008-2009

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EXECUTIVE SUMMARY:

Purpose

This building existing conditions and structural concepts reports introduces the physical existing conditions of Ingleside at King Farm and also its design concepts. This report illustrates the relevant design codes and regulations it adheres to through structural analysis to confirm the structural strength and serviceability of the building.

The Building

Ingleside at King Farm is a 7-story concrete continuous care retirement center that is 103 feet high. The main structural system is a two-way post-tension flat plate this is 8 inches for typical floors with no beams or drop panels. However, there are drop panels located in the sub grade level where columns hold up the courtyard and the 12 inch concrete lab on grade. The layout of the number of tendons varies for the post-tension slabs. The only lateral system is made of sever reinforced concrete shear walls. The vertical supports are mainly 30" x 18" reinforced concrete columns. Loads are transferred down through these columns to spread-footings and then to soil with a bearing capacity of 5000 psi. The building is a mixed use, thus allows for many load patterns and serviceability requirements. Although it the building's design incorporates some green features, it has not LEED certification.

The Structural Analysis

Although the building was designed to older version of model building codes and standards, the most up to date versions are used for this report's structural and serviceable analysis and future ones. Method 2 of the ASCE 7 was used for wind loads and the equivalent lateral force procedure was used for the seismic loads. Analysis of the building had two main challenges. One challenge is the four expansion joints in the building. Thus the building has to be divided into 5 individual sections for lateral force analysis. Using the building's symmetry, only three sections is sufficient to carry out the analysis.

The other main challenge is the irregular column grid, which made it difficult to find a location to perform a spot check. Equivalent frame analysis is preformed for a column strip with two columns there were within the 10 percent offset as specified by ACI.

After analyzing for punching shear for an exterior column, it did not match the design value. Perhaps a further study on the distribution of the moments for the next report is needed using a different method. A spot check of the same exterior column for axial loading although exceeded the standards, was found to provide more than the adequate strength. Using the equivalent frame analysis to spot check the post-tension slab, the results were sufficient for flexural strength. Much of the discrepancies in the calculations are due to the different assumptions of loads and the application of different codes.

INGLESIDE AT KING FARM
STRUCTURAL CONCEPTS AND EXISTING CONDITIONS REPORT

701 King Farm Blvd.
Rockville, MD 20852

INTRODUCTION

This report contains information of the physical existing conditions, design concepts, and load requirements of the structure of Ingleside at King Farm. It provides an overview of all the structural components of the building including, but not limited to: general floor framing, structural slabs, lateral resisting system, foundation system, bracing elements, expansion joints, secondary structural equipment support and the supporting structure of the exterior building envelop system. This report also discusses the design codes involved in the building's design and the confirmation of the code's application through structural analysis of the building's structural strength and serviceability.

BACKGROUND OF BUILDING

Ingleside at King Farm is owned by the Ingleside Presbyterian Retirement Community and was designed by Cochran, Stephenson & Donkervoet, Inc. (CSD). The building is being constructed under a guaranteed max price of $97 million, which covers construction only with a CM contract by general contractor Turner Construction Company of Baltimore, MD and construction manager Turner-Konover of Rockville, MD. Morabito Consultants, Inc. is serving as the engineering firm. Construction of the 103 feet, seven- story and 790,000 square footage post-tension concrete building began on November 1, 2006. It is expected to end on January 15, 2009.

The building site is located between a residential and commercial zone. The building itself is a mixed-use continuous care retirement center designed with several roof gardens, independent living units, assisted living units, and nursing units for the top seven floors. In addition, the first floor consist of many public servicing areas including but not limited to a theater, Olympic size swimming pool and a market place is the first floor. All the floor plans are identical with the exception of the first floor having an extended floor area for the swimming pool and market place.

Due to the building site's proximately of 0.30 miles from King Farm Farmstead Park Historic District, the architectural design is rather conservative (by choice) and is designed in context with the existing buildings in the community. There is no unique style to describe its architecture. Although it resembles the Victorian style it follows the Architectural Design Guidelines for the Exterior Rehabilitation of Buildings in Rockville's Historic Districts adopted in 1977.

There is uniformity both in the proportion of the facades along the street and in the spacing of buildings. Rhythm and harmony is well encouraged. Windows are all proportional and are evenly spaced apart. Keystones, dormers, lintels, and wrought iron shutters are used to give dept to windows and doors.

The building envelope consists of three primary wall assemblies. The exterior façade at the base consist of 16x24 cast stones giving it a more solid and rustic feeling than the rest of the building. It is followed by an air space, ½" sheathing, masonry veneer ties at 16" O.C. 6" steel studs at 16" O.C. 6" batt insulation at R of 19 and 5/8" foil face gypsum board. The mid section of the building is similar to the base except masonry brick is used in place of the cast stone. The portion atop the veneer brick section consists of light-beige stucco with a reinforcing mesh behind it in place of masonry. The top floor of the building consists of a sloped roof assembly that is a dark colored shingle mansard roof style with a defined soffit line. Behind the singles are plywood roof sheathing and 4" metal studs. The roof membrane is a 3" rigid insulation on 1 ½" x 20 gauge galvanized metal deck supported by either 26 k12 or 28 k12 joists depending on the roof loads. There are also low roof assembly areas consisting of 8" post tension slab with a membrane roof water proofing system.

There are two areas of the building that is not considered in calculations or analysis for this report. They are the Pool House and the Market Place, their names as indicated in the construction documents. The Pool House addition is under a different contract, thus it is not constructed yet. When it is constructed, it will be two stories high. The Market Place is part of the contract and is not yet constructed. The construction documents that were obtained has no detailed information about the layout of the space, nor the building envelope for that area. When constructed, the Market Place will be one store high.

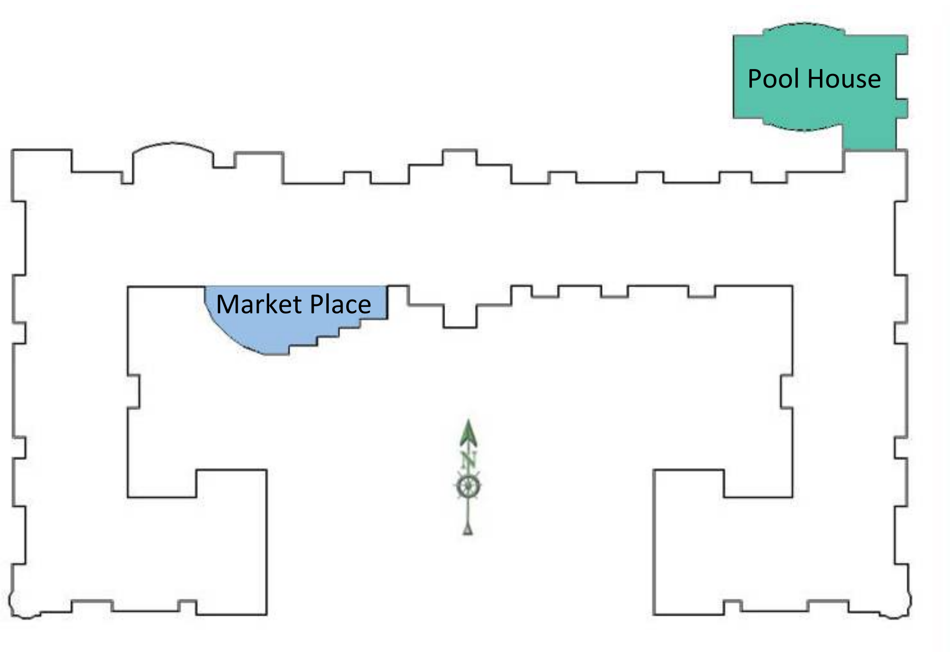


Figure 1: Shows First Floor of Building

There are four expansion joints built into the building. One of the primary reasons for these expansion joints is to make construction of the entire building simpler and faster. By dividing it into sections, while one section is left alone so that the concrete can cure, another section can be worked on. Another reason is due to the large square footage of the building. **Figure 2** shows a typical floor plan from floor two to floor six. Floor seven's building envelope is a sloped mansard roof assembly. Some parts of the building's roof line ends at level six and some at level seven. The building will have to be divided at the expansion joints for lateral loads analysis. Symmetry of the building can be utilized to simplify calculations. In fact, the building has a symmetrical column grid and lateral resistance system.

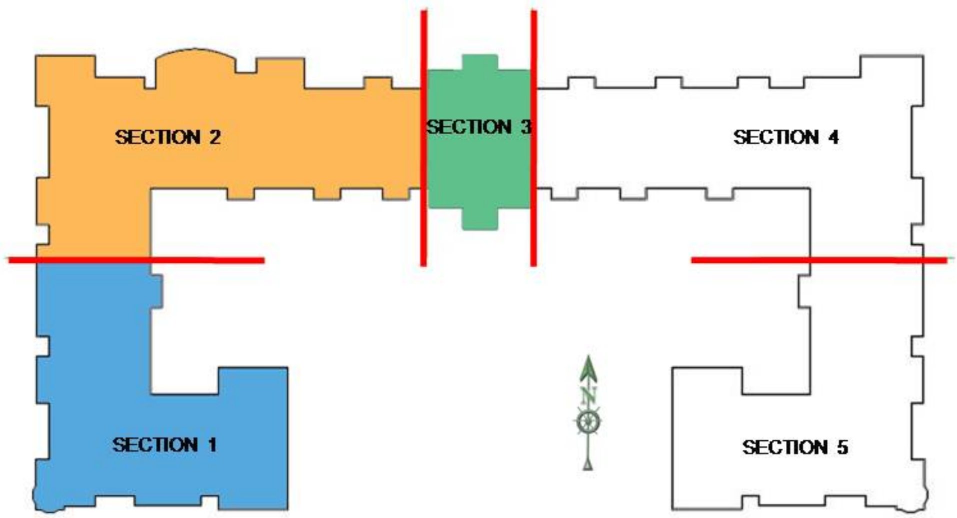


Figure 2: Shows typical floor plan for levels 2-6 and expansion joints indicated by red lines.

STRUCTURAL SYSTEM DISCUSSION

Foundation

The sub level of the building is mainly used as a parking garage and also houses most of the building's mechanical rooms. The loads from above are transferred down by 30" x 18" reinforced concrete columns with 10 #8 bars to spread footings. Beneath the spread footings is 3 feet of compact fill and then soil with a bearing capacity of 50 ksi. The 30" x 18" reinforced columns extends all the way to either the 6^{th} or 7^{th} floor. The structural slab in the foundation and sub level parking garage is a 5" concrete slab on grade reinforced with 6" x 6" W2.9 / W2.9 welded wire fabric over a vapor barrier and a 4" porous fill. It utilizes standard weight concrete with a 28 day minimum compressive strength of 4000 psi.

Typical Floor Frame

Ingleside at King Farm's primary structural system is a two-way flat plate post-tension concrete structure with 270 ksi unbounded ½ diameter 7 wire tendons. The post-tension concrete slabs are 8 inches thick for typical floors with a compressive strength of 4500 psi. All Concrete used in this building's construction is normal weight. There are no drop panels or beams supporting these slabs. The only drop panels in the building are found on the sub level columns holding up the 12 inch thick slab where the court yard rests. The drop panels are 10 inch thick and the 12 inch thick slab has a compressive strength of 6000 psi.

Shear Wall

Ingleside at King Farm has eleven shear walls to resist lateral load. Seven of the walls are ordinary reinforced concrete shear walls located at stairwells and elevator shafts with #4 horizontal reinforcing bars and #8 vertical reinforcing bars. All these walls have a compressive strength of 5000 psi with #5 reinforcing bars. The remaining four reinforced concrete shear walls have boundary elements and are 15 feet in length; two in east/west direction and two in north/south direction.

Columns

The building contains over 140 reinforced columns, which are either 18” x 30” or 12” x 30”. Due to the building’s irregular column grid, some columns are not accounted for in the column schedule. The column schedule also does not account for the 6” x 6” x 3/8” steel tubular columns that are located in section two of the building where a majority of the public areas are found. Those Tubular columns support the gravity loads of areas whose roof line is at the first floor or second floor level. **Figure 3** shows the column layout of the sub level parking garage. Notice how the only columns that are in perfect alignment with many consistent continuous bays are those columns that support only the loads of the court yard above it. All the other reinforced concrete columns extend to either the 6th or 7th floor.

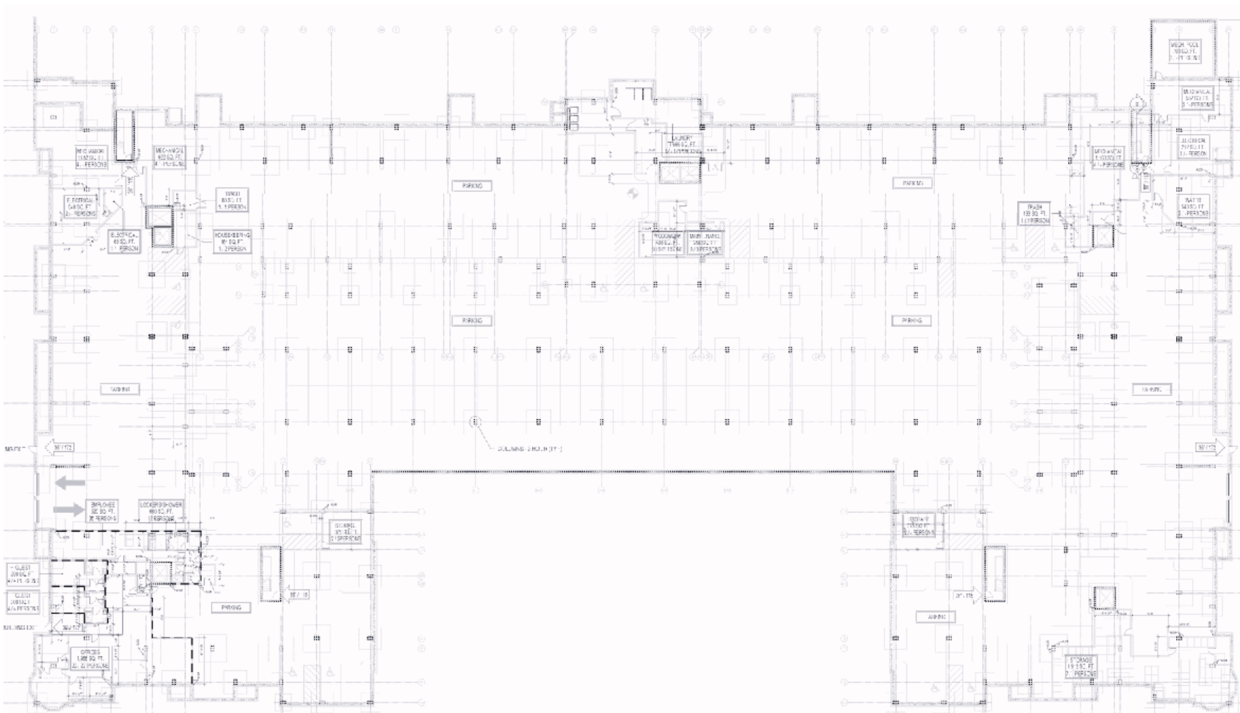


Figure 3: Sub Level Parking Garage

Other Structural Elements

Several structural elements that have not been analyzed for this report, but they will be at a later time. They include structural components for the canopies, building envelope supports and load paths into the structural slabs, the steel joists and tubular steel members supporting the roof and roof up lift. An analysis of these structural members for structural strength and serviceability shall be done for the future, and as well as how the various systems work together.

CODES AND STANDARDS

Codes and Standards in Original Design	Codes and Standards used for this Report
IBC 2003	International Building Code 2006
ASCE 7-98: Minimum Design Loads For Buildings and other Structures.	American Institute of Steel Construction 13th Edition
Rockville, MD City Codes: Local amendments	ASCE 7-05: Minimum Design Loads For Buildings and other Structures.
	American Concrete Institute: Building Code Requirements for Structural Concrete 318 - 05
	Post-Tensioning Institute (PTI) 1st edition

STRUCUTRAL MATERIAL SUMMARY

Structural Steel	
All Rolled Shapes	ASTM A-500, GRADE B (Fy=46 ksi)
Structural Concrete	
Structural Slab Supporting Court Yard	6 ksi Normal wt.
Slab on Grade/Foundation	4 ksi Normal wt.
Floor Slab	4.5 ksi Normal wt.
Columns	5 ksi Normal wt.
Cast in Place Walls	5 ksi Normal wt.
Shear Walls	5 ksi Normal wt.
Reinforcements	
Deformed Bars	ASTM A615, GRADE 60
Welded Wire Fabric	ASTM A18
Post-Tension Tendons	ASTM A-416-74 (270 ksi)

BUILDING DESIGN LOAD DISCUSSION:

Gravity Loads

Static and dynamic loads acting on the building were determined in order to analyze the structural behavior of the building. Information regarding the building's weight, code compliant loadings and material specifications were provided and referenced from the construction documents, specifications, AISC 13th edition, ASCE 7 - 05, and IBC 2006. The table below summarizes the type of gravity loads and the system it applies to.

Floor System Loads			
Load Type	Material / Usage	Load	Reference
Dead Load	Normal Weight Concrete	150 pcf	ACS 318
	Cold-formed, light gauge steel stud walls with insulation and 5/8" gypsum board	5 psf	WDG
	Brick Masonry	40 psf	AISC 13th ed.
	Partition Walls	15 psf	Engineer's Judgment
	Miscellaneous	10 psf	Engineer's Judgment
Live Load	Lobbies and Common Spaces	100 psf	ASCE 7 - 05
	Theater Stage	100 psf	ASCE 7 - 05
	Corridors	100 psf	ASCE 7 - 05
	Living Units	40 psf	ASCE 7 - 05
	Balconies	60 psf	ASCE 7 - 05
	Parking Garage	40 psf	ASCE 7 - 05
	Retail Spaces	100 psf	ASCE 7 - 05

Roof and Terrace System Loads			
Load Type	Material / Usage	Load	Reference
Dead Load	Normal Weight Concrete	150 pcf	ACS 318
	Steel	by shape	AISC 13th ed.
	Steel Deck	2 psf	USD
	Green Roof	100 psf	ASCE 7 - 05
	Ballast, insulation, and waterproofing membrane	8 psf	AISC 13th ed.
	Miscellaneous	10 psf	Engineer's Judgment
Live	Assembly Spaces	100 psf	ASCE 7 - 05
	Roof	30 psf	ASCE 7 - 05
Snow	Ground Snow Load	25 psf	ASCE 7 - 05 & IBC 2006
	Terrain Category	B	ASCE 7 - 05 & IBC 2006
	Ce Exposure	1	ASCE 7 - 05 & IBC 2006
	Ct Thermal Factor	1	ASCE 7 - 05 & IBC 2006
	Importance Factor	1	ASCE 7 - 05 & IBC 2006
	Flat Roof Snow	30 psf	ASCE 7 - 05 & IBC 2006

The miscellaneous gravity loads consist of lighting, plumbing, telecommunication, ACT, ductwork and anything that is not regarded as a live load. Because the building's roof is a mansard roof, snow drift will accumulate in the lower flat roof areas. The drift loads are not determined for this report.

Lateral Loads

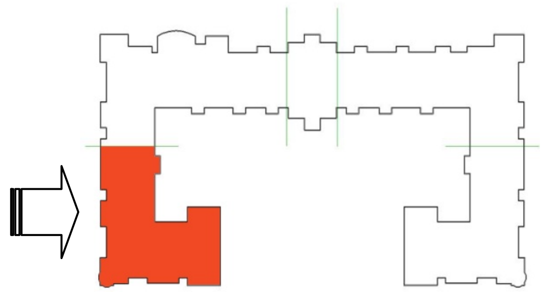
The main lateral force resisting system of Ingleside at King Farm consists of special and ordinary reinforced concrete shear walls located primarily around emergency exit stairwells and elevator shafts. They are also symmetrically placed within the 103 feet tall building. Due to 4 expansion joints within the building, it has to be divided into five sections. However, the building's symmetry can be utilized so that analysis of wind pressure effects of only three of these sections is sufficient enough. The first floor plan varies slightly from the other floors due to the pool room and the market place. These two areas can be ignored because the pool is under a different contract and is not built yet, and the square footage of the market place is less than 10 percent of the overall floor area of the building section it is located in. Each section is then individually analyzed for wind pressure, shear forces and overturning moments. Two of the building sections are analyzed for wind pressures in two main directions (west-east, north-south and vice versa), while the other one of the three building sections requires wind analysis at a 45degree angle.

The table below contains the parameters for building section 1 using the analytical method 2 for MWFRS

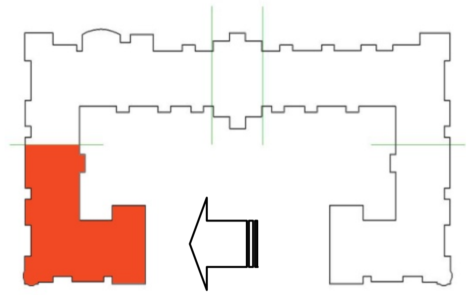
Building Section I Parameters for Method 2 Analysis	
Basic Wind Speed: V	90 mph
Wind Directionality Factor: K_d	0.85
Importance Factor: I	1
Exposure Category	B
Velocity Pressure Coefficient: K_z	Case 2
Topographic Factor: K_{zt}	1
Gust Effect Factor: G	0.85
Enclosure Classification:	Enclosed
External Pressure Coefficient: C_p	0.8 (windward); -0.5 W-E, -0.5 E-W (Leeward); -0.5 SW-NE, -0.5 NE-SW (Leeward)

The wind pressure in pounds per square inch (psf) at each story or level were calculated using parameters from the above table and equations provided by ASCE 7 - 05 for a rigid structure. They are summarized in the tables below.

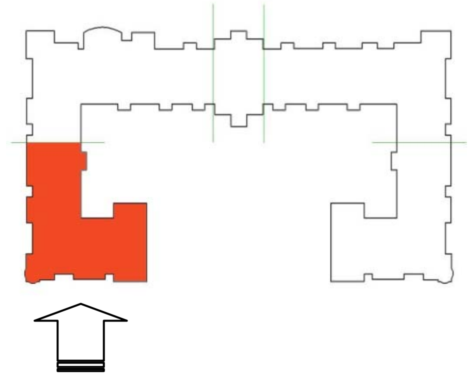
Wind Pressures (West-East)				
Floor	Height from Ground (ft)	Windward (psf)	Leeward (psf)	Total (psf)
Ground	0	6.83	-6.19	13.02
2	14	6.83	-6.19	13.02
3	24	7.43	-6.19	13.62
4	34	8.68	-6.19	14.86
5	44	9.35	-6.19	15.54
6	54	9.90	-6.19	16.09
roof	66	10.48	-6.19	10.48



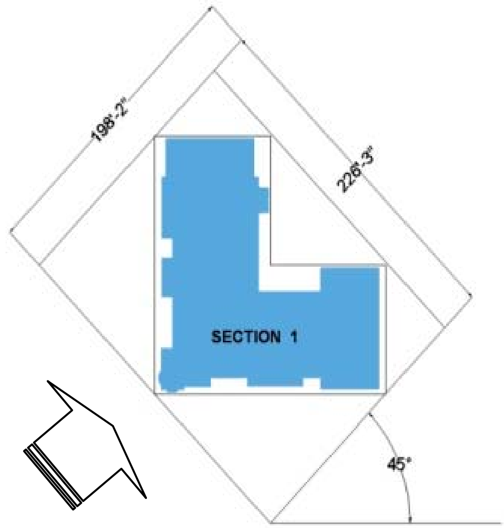
Wind Pressures (East-West)				
Floor	Height from Ground (ft)	Windward (psf)	Leeward (psf)	Total (psf)
Ground	0	6.83	-6.55	13.38
2	14	6.83	-6.55	13.38
3	24	7.43	-6.55	13.98
4	34	8.68	-6.55	15.22
5	44	9.35	-6.55	15.90
6	54	9.90	-6.55	16.45
roof	66	na	-6.55	6.55

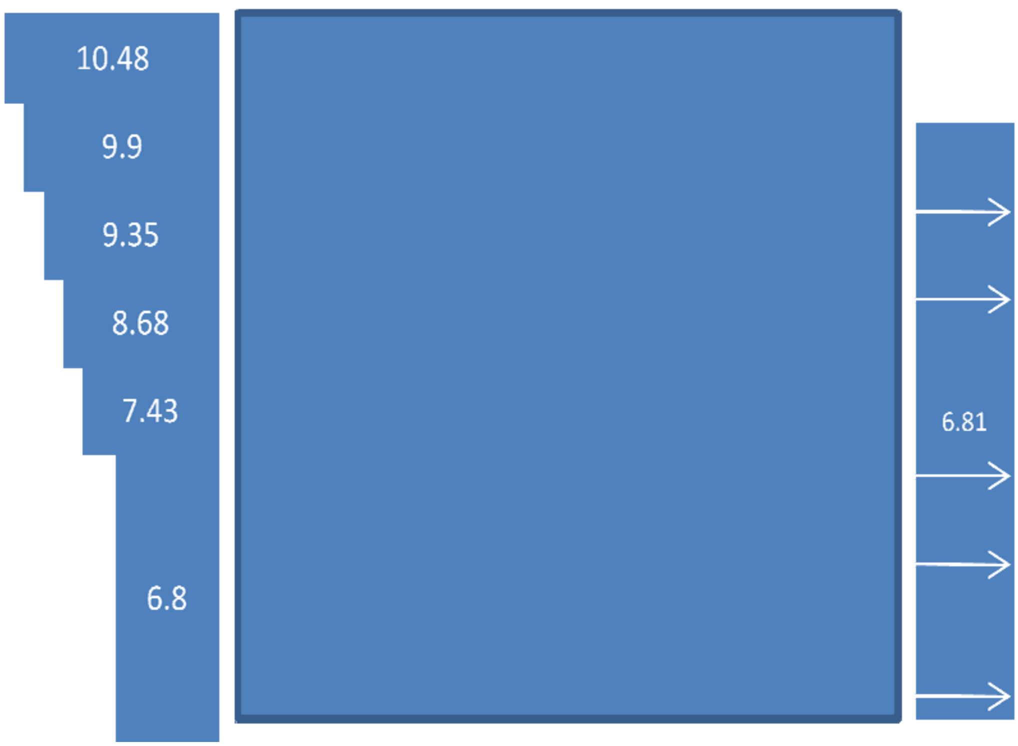


Wind Pressures (South-North)				
Floor	Height from Ground (ft)	Windward (psf)	Leeward (psf)	Total (psf)
Ground	0	6.83	-6.55	13.38
2	14	6.83	-6.55	13.38
3	24	7.43	-6.55	13.98
4	34	8.68	-6.55	15.22
5	44	9.35	-6.55	15.90
6	54	9.90	-6.55	16.45
roof	66	10.48	-6.55	17.02



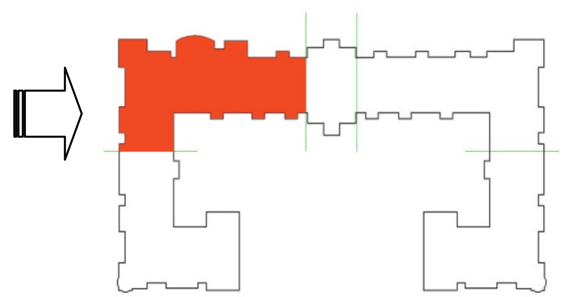
Wind Pressures (SW-NE) Orthogonal @ 45 Degrees				
Floor	Height from Ground (ft)	Windward (psf)	Leeward (psf)	Total (psf)
Ground	0	6.83	-6.81	13.64
2	14	6.83	-6.81	13.64
3	24	7.43	-6.81	14.24
4	34	8.68	-6.81	15.49
5	44	9.35	-6.81	16.16
6	54	9.90	-6.81	16.71
roof	66	10.48	-6.81	17.28



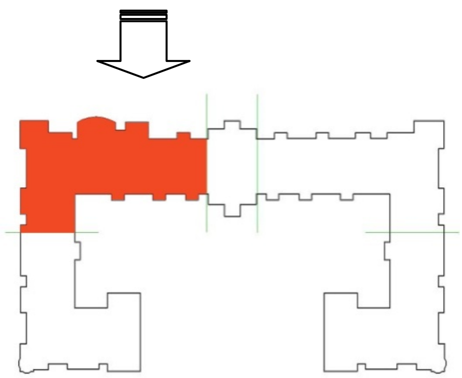


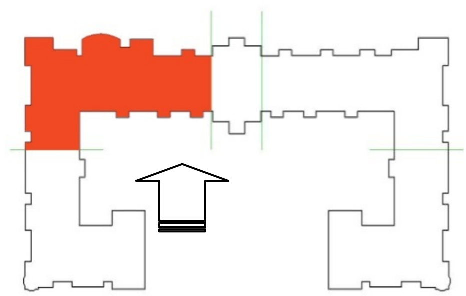
Building Section II Parameters for Method 2 Analysis	
Basic Wind Speed: V	90 mph
Wind Directionality Factor: K_d	0.85
Importance Factor: I	1
Exposure Category	B
Velocity Pressure Coefficient: K_z	Case 2
Topographic Factor: K_{zt}	1
Gust Effect Factor: G	0.85
Enclosure Classification:	Enclosed
External Pressure Coefficient: C_p	0.8 (windward); 0 W-E, 0E-W (Leeward); -0.5 S-N, -0.5 N-S (Leeward);

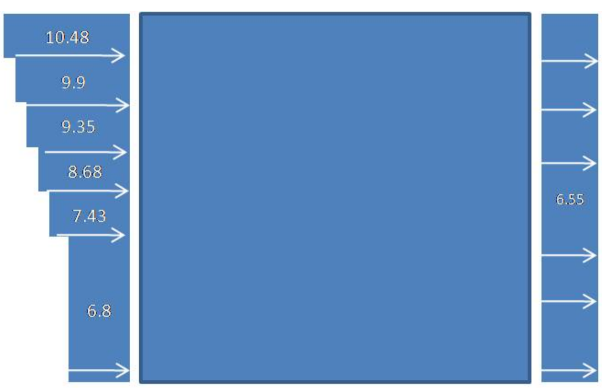
Wind Pressures (West-East)				
Floor	Height from Ground (ft)	Windward (psf)	Leeward (psf)	Total (psf)
Ground	0	6.83	0	6.83
2	14	6.83	0	6.83
3	24	7.43	0	7.43
4	34	8.68	0	8.68
5	44	9.35	0	9.35
6	54	9.90	0	9.90
roof	66	10.48	0	10.48

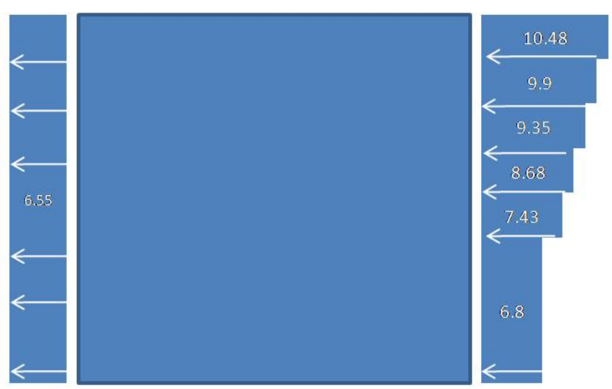


Wind Pressures (South-North) or (North-South)				
Floor	Height from Ground (ft)	Windward (psf)	Leeward (psf)	Total (psf)
Ground	0	6.83	-6.55	13.38
2	14	6.83	-6.55	13.38
3	24	7.43	-6.55	13.98
4	34	8.68	-6.55	15.22
5	44	9.35	-6.55	15.90
6	54	9.90	-6.55	16.45
roof	66	10.48	-6.55	17.02



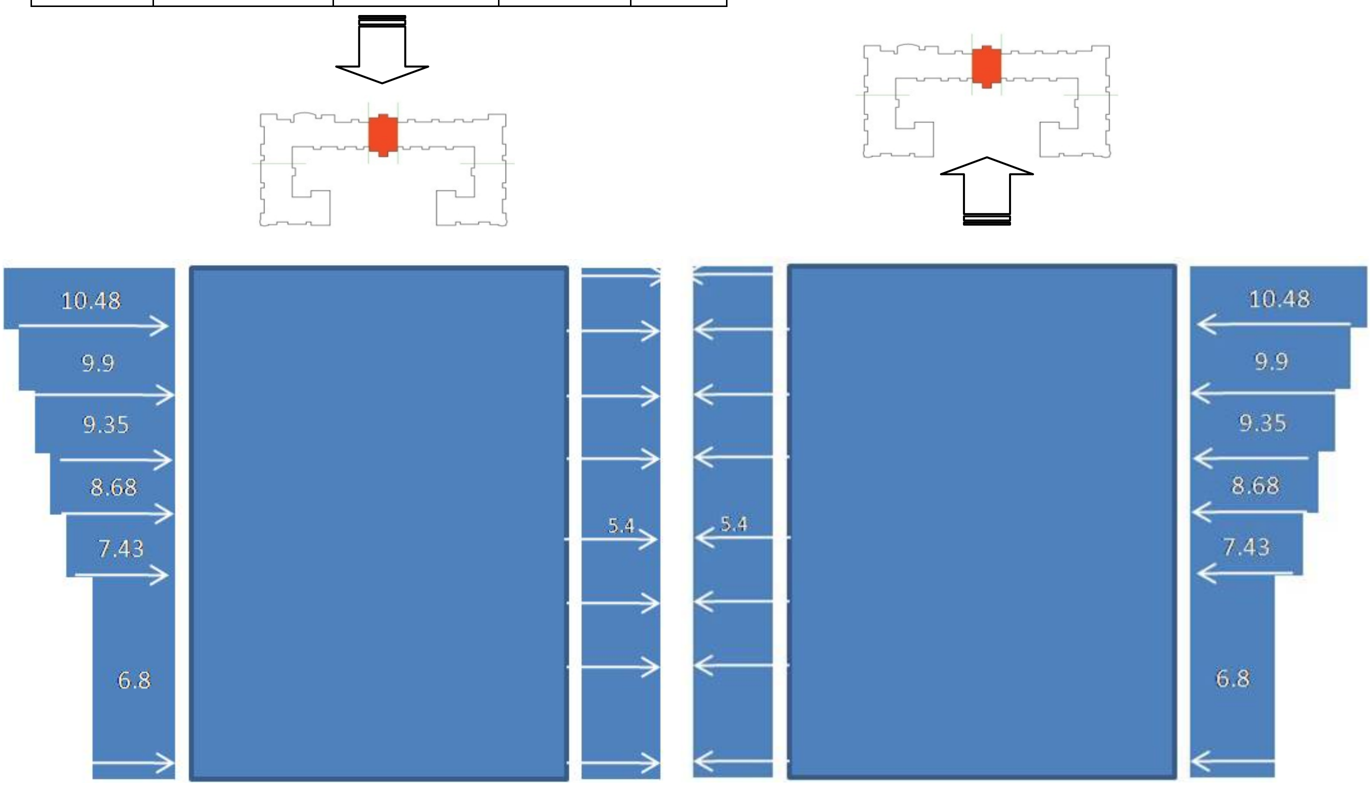






Building Section III Parameters for Method 2 Analysis	
Basic Wind Speed: V	90 mph
Wind Directionality Factor: K_d	0.85
Importance Factor: I	1
Exposure Category	B
Velocity Pressure Coefficient: K_z	Case 2
Topographic Factor: K_{zt}	1
Gust Effect Factor: G	0.85
Enclosure Classification:	Enclosed
External Pressure Coefficient: C_p	0.8 (windward); -0.4 S-N, -0.4 N-S (Leeward);

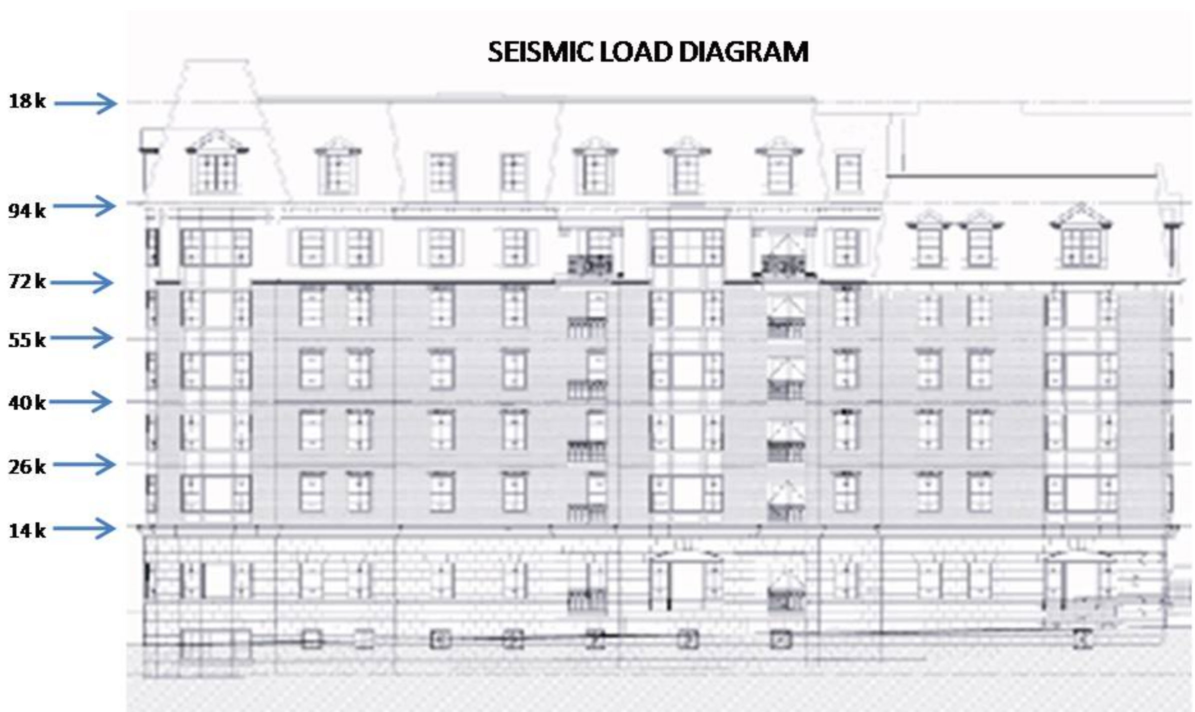
Wind Pressures (South-North) or (North-South)				
Floor	Height from Ground (ft)	Windward (psf)	Leeward (psf)	Total (psf)
Ground	0	6.83	-5.24	12.07
2	14	6.83	-5.24	12.07
3	24	7.43	-5.24	12.67
4	34	8.68	-5.24	13.92
5	44	9.35	-5.24	14.59
6	54	9.90	-5.24	15.14
roof	66	10.48	-5.24	15.71



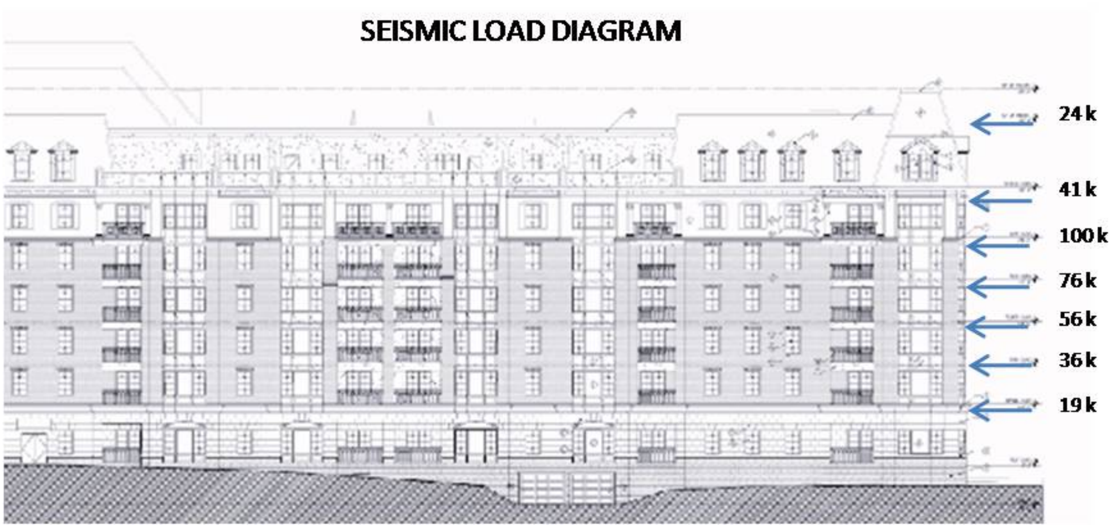
Similar to the wind load analysis, seismic loading is also preformed on each of the three individual building sections

Building Parameters for Seismic Analysis of BUILDING SECTION I, II, III	
Short Period MCE Spectral Response Acceleration: S_s	15.6%
1 sec. Period MCE Spectral Response Acceleration: S_1	5.1%
Site Class	A
Site Coefficients: Fa and Fv	1.0
Short Period Design Spectral Response Acceleration: S_{DS}	0.10
1 sec Period Design Spectral Response Acceleration: S_{D1}	0.03
Seismic Use Group	I
Importance Fact: I	1.0
Response Modification Factor: R	6.0
Fundamental Period: T	0.93
Seismic Response Coefficient: C_s	0.02

Seismic Load Distributed per Floor (SECTION I)								
Floor	Story Height From Ground (ft)	Story Height For Each Floor	Tributary Height (ft)	Story Weight (kips)	wh^k	Cvx	Fx (kips)	Overturning Moment (ft-kips)
Ground	0.00	14.00	0.00	2570.00	0.00	1.00	319	13956
2	14.00	10.00	12.00	2442.00	60288.42	0.04	14	13956
3	24.00	10.00	10.00	2442.00	116046.84	0.08	26	10200
4	34.00	10.00	10.00	2442.00	177180.52	0.13	40	7279
5	44.00	10.00	10.00	2442.00	242358.63	0.17	55	4689
6	54.00	12.00	11.00	2506.00	318972.22	0.23	72	2571
7	66.00	16.50	16.00	2582.00	419383.31	0.30	94	971
Roof	82.50	8.25	6.00	384.00	81795.04	0.06	18	110



Seismic Load Distributed per Floor (SECTION 2)								
Floor	Story Height From Ground (ft)	Story Height For Each Floor	Tributary Height (ft)	Story Weight (kips)	wh^k	Cvx	Fx (kips)	Overturning Moment (ft-kips)
Ground	0.00	14.00	0.00	3128.00	0.00	1.00	352	13975
2	14.00	10.00	12.00	2992.00	73866.90	0.05	19	13975
3	24.00	10.00	10.00	2992.00	142183.51	0.10	36	9848
4	34.00	10.00	10.00	2992.00	217086.04	0.16	56	6701
5	44.00	10.00	10.00	2992.00	296943.91	0.22	76	4014
6	54.00	12.00	11.00	3060.00	389487.23	0.28	100	1987
7	66.00	16.50	16.00	982.00	159502.10	0.12	41	774
Roof	82.50	8.25	6.00	440.00	93723.48	0.07	24	144



Seismic Load Distributed per Floor (SECTION III)								
Floor	Story Height From Ground (ft)	Story Height For Each Floor	Tributary Height (ft)	Story Weight (kips)	wh^k	Cvx	Fx (kips)	Overturning Moment (ft-kips)
Ground	0.00	14.00	0.00	3128.00	0.00	1.00	390	20907
2	14.00	10.00	12.00	2992.00	73866.90	0.03	13	20907
3	24.00	10.00	10.00	2992.00	142183.51	0.06	25	16291
4	34.00	10.00	10.00	2992.00	217086.04	0.10	38	12644
5	44.00	10.00	10.00	2992.00	296943.91	0.13	52	9309
6	54.00	12.00	11.00	3060.00	389487.23	0.17	68	6422
7	66.00	16.50	16.00	782.00	127016.94	0.06	22	3937
Roof	103.00	20.50	6.00	3561.00	993272.84	0.44	173	1038

SEISMIC LOAD DIAGRAM

WIND LOAD CALCULATIONS IN DEPT

Wind: (West-East) SECTION I													
Floor	Height for each foor	Height from Ground (ft)	Tributary Height	Kz Windward	qz Windward	kh Leeward	qh Leeward	Windward (psf)	Leeward (psf)	Total (psf)	Story Force (kip)	Story Shear (kips)	Overturning Moment
Ground	14	0	0	0.57	10.05	0.57	10.05	6.83	-6.19	13.02	0.00	133.80	4031.37
2	10	14	12	0.57	10.05	0.57	10.05	6.83	-6.19	13.02	25.00	133.80	4031.37
3	10	24	10	0.62	10.93	0.62	10.93	7.43	-6.19	13.62	21.79	108.80	2550.78
4	10	34	10	0.72	12.76	0.72	12.76	8.68	-6.19	14.86	23.78	87.01	1571.71
5	10	44	10	0.78	13.75	0.78	13.75	9.35	-6.19	15.54	24.86	63.23	820.51
6	12	54	11	0.83	14.56	0.83	14.56	9.90	-6.19	16.09	28.31	38.37	312.53
roof	0	66	6	0.87	15.40	na	na	10.48	na	10.48	10.06	10.06	60.34

Wind (East-West) SECTION I													
Floor	Height for each foor	Height from Ground (ft)	Tributary Height	Kz Leeward	qz Leeward	kh Windward	qh Windward	Windward (psf)	Leeward (psf)	Total (psf)	Story Force (kip)	Story Shear (kips)	Overturning Moment
Ground	14	0	0	0.57	10.05	0.57	10.05	6.83	-6.55	13.38	0.00	133.08	3890.05
2	10	14	12	0.57	10.05	0.57	10.05	6.83	-6.55	13.38	25.69	133.08	3890.05
3	10	24	10	0.62	10.93	0.62	10.93	7.43	-6.55	13.98	22.36	107.39	2421.57
4	10	34	10	0.72	12.76	0.72	12.76	8.68	-6.55	15.22	24.36	85.02	1459.51
5	10	44	10	0.78	13.75	0.78	13.75	9.35	-6.55	15.90	25.43	60.66	731.06
6	**12**	54	11	0.83	14.56	0.83	14.56	9.90	-6.55	16.45	28.95	35.23	251.58
Roof	0	66	6	0.87	15.40	na	na	na	-6.55	6.55	6.29	6.29	37.71

Wind: (South-North) SECTION I													
Floor	Height for each foor	Height from Ground (ft)	Tributary Height	Kz Windward	qz Windward	kh Leeward	qh Leeward	Windward (psf)	Leeward (psf)	Total (psf)	Story Force (kip)	Story Shear (kips)	Overturning Moment
Ground	14	0	0	0.57	10.05	0.57	10.05	6.83	-6.55	13.38	0.00	136.85	4112.54
2	10	14	12	0.57	10.05	0.57	10.05	6.83	-6.55	13.38	25.69	136.85	4112.54
3	10	24	10	0.62	10.93	0.62	10.93	7.43	-6.55	13.98	22.36	111.16	2598.81
4	10	34	10	0.72	12.76	0.72	12.76	8.68	-6.55	15.22	24.36	88.80	1599.04
5	10	44	10	0.78	13.75	0.78	13.75	9.35	-6.55	15.90	25.43	64.44	832.88
6	**12**	54	11	0.83	14.56	0.83	14.56	9.90	-6.55	16.45	28.95	39.00	315.69
roof	0	66	6	0.87	15.40	0.87	15.40	10.48	-6.55	10.48	10.06	10.06	60.34

Wind: (Southwest-Northeast & Vice Versa) SECTION I													
Floor	Height for each foor	Height from Ground (ft)	Tributary Height	Kz Windward	qz Windward	kh Leeward	qh Leeward	Windward (psf)	Leeward (psf)	Total (psf)	Story Force (kip)	Story Shear (kips)	Overturning Moment (ft-kips)
Ground	14	0	0	0.57	10.05	0.57	10.05	6.83	-6.81	13.64	0.00	172.24	5166.87
2	10	14	12	0.57	10.05	0.57	10.05	6.83	-6.81	13.64	32.44	172.24	5166.87
3	10	24	10	0.62	10.93	0.62	10.93	7.43	-6.81	14.24	28.22	139.81	3262.13
4	10	34	10	0.72	12.76	0.72	12.76	8.68	-6.81	15.49	30.69	111.59	2005.16
5	10	44	10	0.78	13.75	0.78	13.75	9.35	-6.81	16.16	32.02	80.90	1042.74
6	12	54	11	0.83	14.56	0.83	14.56	9.90	-6.81	16.71	36.42	48.88	393.86
roof	0	66	6	0.87	15.40	0.87	15.40	10.48	-6.81	10.48	12.46	12.46	74.73

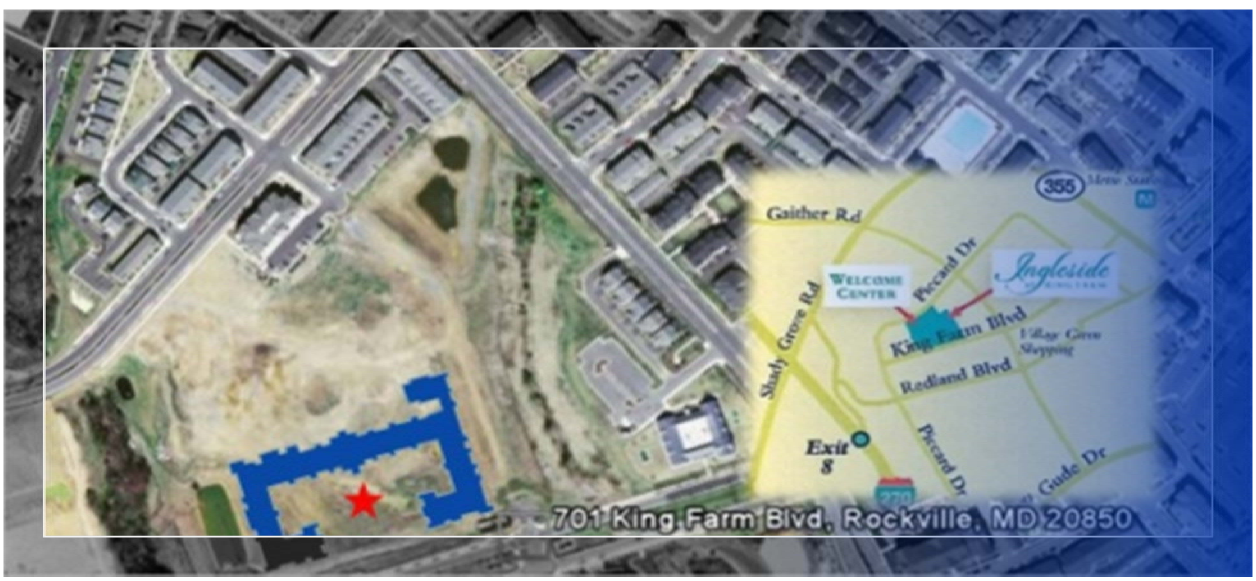
Wind: (South-North) SECTION II													
Floor	Height for each foor	Height from Ground (ft)	Tributary Height	Kz Windward	qz Windward	kh Leedward	qh Leeward	Windward (psf)	Leeward (psf)	Total (psf)	Story Force (kip)	Story Shear (kips)	Overturning Moment
Ground	14	0	0	0.57	10.05	0.57	10.05	6.83	-6.55	13.38	0.00	205.27	6168.81
2	10	14	12	0.57	10.05	0.57	10.05	6.83	-6.55	13.38	38.53	205.27	6168.81
3	10	24	10	0.62	10.93	0.62	10.93	7.43	-6.55	13.98	33.55	166.74	3898.22
4	10	34	10	0.72	12.76	0.72	12.76	8.68	-6.55	15.22	36.54	133.19	2398.56
5	10	44	10	0.78	13.75	0.78	13.75	9.35	-6.55	15.90	38.15	96.65	1249.33
6	12	54	11	0.83	14.56	0.83	14.56	9.90	-6.55	16.45	43.42	58.50	473.53
roof	0	66	6	0.87	15.40	0.87	15.40	10.48	-6.55	10.48	15.08	15.08	90.51

Wind: (West-East) SECTION II													
Floor	Height for each foor	Height from Ground (ft)	Tributary Height	Kz Windward	qz Windward	kh Leeward	qh Leeward	Windward (psf)	Leeward (psf)	Total (psf)	Story Force (kip)	Story Shear (kips)	Overturning Moment
Ground	14	0	0	0.57	10.05	0.57	10.05	6.83	0.00	6.83	0.00	50.83	1646.55
2	10	14	12	0.57	10.05	0.57	10.05	6.83	0.00	6.83	8.20	50.83	1646.55
3	10	24	10	0.62	10.93	0.62	10.93	7.43	0.00	7.43	7.43	42.63	1077.58
4	10	34	10	0.72	12.76	0.72	12.76	8.68	0.00	8.68	8.68	35.20	688.42
5	10	44	10	0.78	13.75	0.78	13.75	9.35	0.00	9.35	9.35	26.52	379.79
6	12	54	11	0.83	14.56	0.83	14.56	9.90	0.00	9.90	10.89	17.18	161.30
roof	0	66	6	0.87	15.40	0.87	15.40	10.48	0.00	10.48	6.29	6.29	37.71

Wind: (South-North) SECTION III													
Floor	Height for each foor	Height from Ground (ft)	Tributary Height	Kz Windward	qz Windward	kh Leedward	qh Leeward	Windward (psf)	Leeward (psf)	Total (psf)	Story Force (kip)	Story Shear (kips)	Overturning Moment
Ground	14	0	0	0.57	10.05	0.57	10.05	6.83	-5.24	12.07	0.00	47.15	1431.35
2	10	14	12	0.57	10.05	0.57	10.05	6.83	-5.24	12.07	8.69	47.15	1431.35
3	10	24	10	0.62	10.93	0.62	10.93	7.43	-5.24	12.67	7.60	38.46	908.95
4	10	34	10	0.72	12.76	0.72	12.76	8.68	-5.24	13.92	8.35	30.86	562.32
5	10	44	10	0.78	13.75	0.78	13.75	9.35	-5.24	14.59	8.75	22.51	295.44
6	12	54	11	0.83	14.56	0.83	14.56	9.90	-5.24	15.14	9.99	13.76	114.06
roof	0	66	6	0.87	15.40	0.87	15.40	10.48	-5.24	10.48	3.77	3.77	22.63

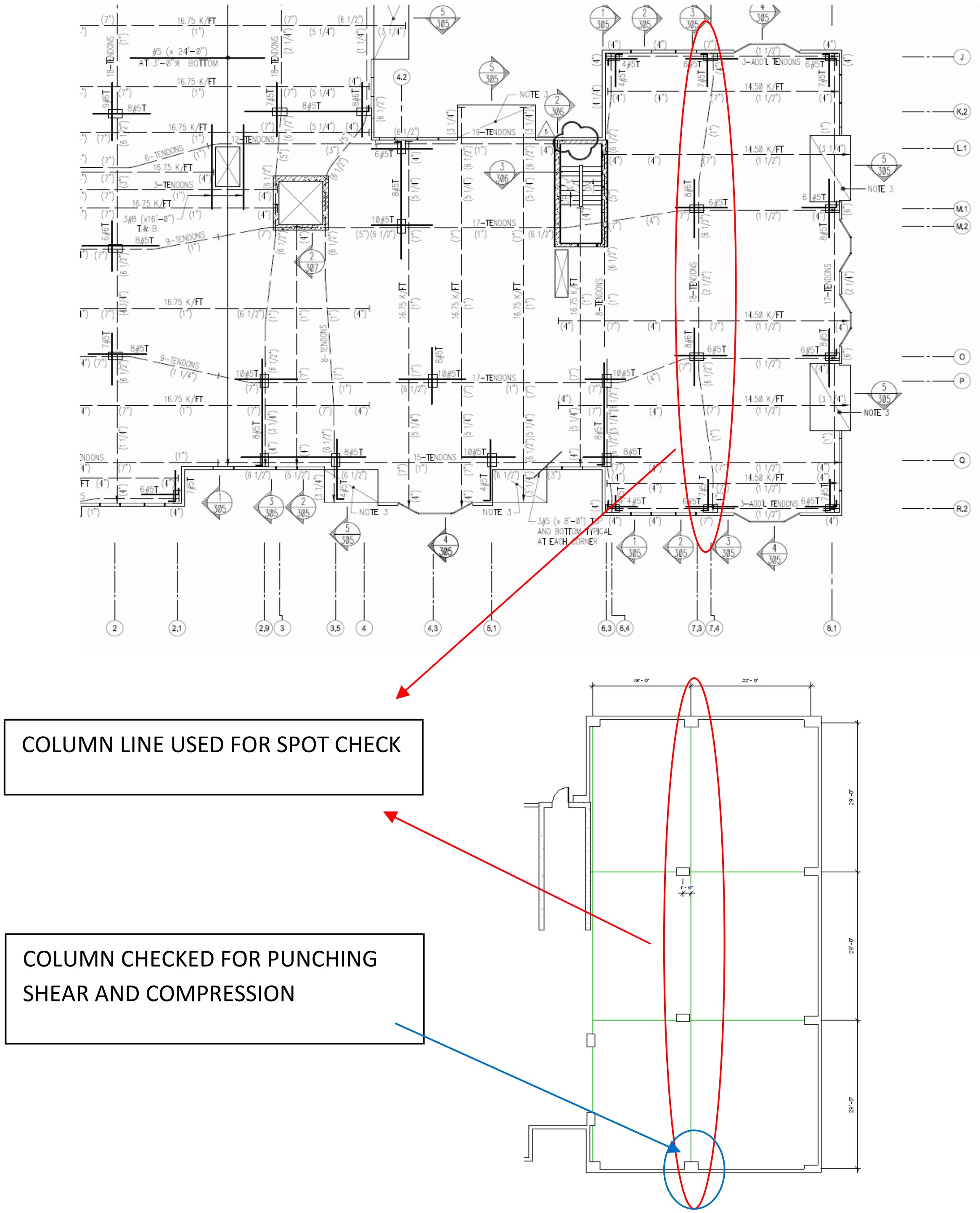
Appendix A
Photos

PANARAMAS



LOCATION

Appendix B
Structural Plans


COLUMN LINE USED FOR SPOT CHECK
COLUMN CHECKED FOR PUNCHING SHEAR AND COMPRESSION

Appendix C
Calculations

Stephen Dung Tat The Pennsylvania State University Architectural Engineering	Thesis: **Ingleside at King Farm**	Spot Check Calculations

LOADS:

FRAMING DEAD LOAD = SELFWEIGHT
PARTITION WALLS = 15 PSF
SUPERIMPOSED D.L. = 10 PSF (M/E, misc.)
LIVE LOAD = 40 PSF (RESIDENTIAL)

MATERIALS:

CONCRETE: NORMAL WEIGHT 150 PCF
f'_c = 4,500 psi
f'_{ci} = 3,000 psi

REBAR: f_y = 60,000 psi

PT: UNBONDED TENDONS
1/2 ϕ, 7 wire strands, $A_{ps} = 0.153\ in^2$
f_{pu} = 270 ksi

ESTIMATED PRESTRESS LOSSES = 15 ksi (ACI 18.6)
REDUCED EFFECTIVE STRESS $f_{se} = 0.7(270\ ksi) - 15\ ksi = 174\ ksi$ (ACI 18.5.1)
EFFECTIVE FORCE $P_{eff} = A \cdot f_{se} = (0.153)(174\ ksi) = 26.6$ kips/tendon

DETERMINE PRELIMINARY SLAB THICKNESS

$L/h = 45$
LONGEST SPAN = 29'

$h = (29')(12'')/45 = 7.73''$

$\therefore$ USE 8.0" SLAB THICKNESS

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LOADING

D.L. = SELF WEIGHT = 8"(150 PCF) = 100 PSF
SI.D.L. & PARTITION WALLS = 25 PSF
LL_o = 40 PSF

IBC 2006, 1607.9.1 allows for LL reduction

EXTERIOR BAY: $A_T = (20')(29') = 580\ ft^2$
$K_{LL} = 1$
$LL = LL_o\left(0.25 + \frac{15}{\sqrt{1 \times 580}}\right) = LL_o(0.873)$
$LL = 35$ PSF

INTERIOR BAY: $A_T = (20')(29') = 580\ ft^2$
$K_{LL} = 1$
$LL = 35$ PSF

DESIGN OF SOUTH-NORTH INTERIOR FRAME

- USE EQUIVALENT FRAME METHOD. ACI 13.7 (EXCLUDING 13.7.7.4-5)

  $LL/DL = 35/125 = 0.28 < 3/4$

  ∴ NO PATTERN LOADING REQUIRED (A.C.I. 13.7.6)

- CALCULATE SECTION PROPERTIES

  TWO-WAY SLAB MUST BE DESIGNED AS CLASS U (ACI 18.3.3)
  GROSS CROSS-SECTIONAL PROPERTIES ALLOWED (ACI 18.3.4)

  $A = bh = (20')(12")(8") = 1920\ in^2$

  $S = \frac{(20')(12")(8")^2}{6} = 2560\ in^3$

- IGNORE COLUMN STIFFNESS IN EQUATIONS FOR SIMPLICITY OF HAND CALCULATIONS.

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• SET DESIGN PARAMETERS

ALLOWABLE STRESSES : CLASS U (ACI 18.3.3)

AT TIME OF JACKING (ACI 18.4.1)

f'_{ci} = 3,000 PSI

COMPRESSION = $0.60(f'_{ci}) = 0.6(3{,}000) = 1{,}800$ PSI

TENSION = $3\sqrt{f'_{ci}} = 3\sqrt{3000} = 164$ PSI

AT SERVICE LOADS (ACI 18.4.2(a) and 18.3.3)

f'_c = 4,500 PSI

COMPRESSION = $0.45f'_c = 0.45(4500) = 2025$ PSI

TENSION = $6\sqrt{f'_c} = 6\sqrt{4500} = 403$ PSI

AVERAGE PRECOMPRESSION LIMITS :

P/A = 125 PSI min. (ACI 18.12.4)
= 300 PSI MAX.

TARGET LOAD BALANCES :

CODES DO NOT PRESCRIBE LIMITATIONS FOR THESE PERCENTAGES BUT WILL NEED TO DESIGN TO APPROPRIATE BALANCING LOADS TO LIMIT SLAB DEFLECTIONS AND CRACKING.

COMMON LOAD-BALANCING PERCENTAGES ARE IN THE 65-PERCENT TO 80-PERCENT RANGE AND IS KEPT CONSISTENT BENTEEN SPANS

AVERAGE OF 70-PERCENT OF DEAD LOAD (SELFWEIGHT) FOR SLABS SHALL BE USED IN THIS CALCULATION.

$0.65_{W_{DL}} = 0.65(100 \text{ PSF}) = 65$ PSF

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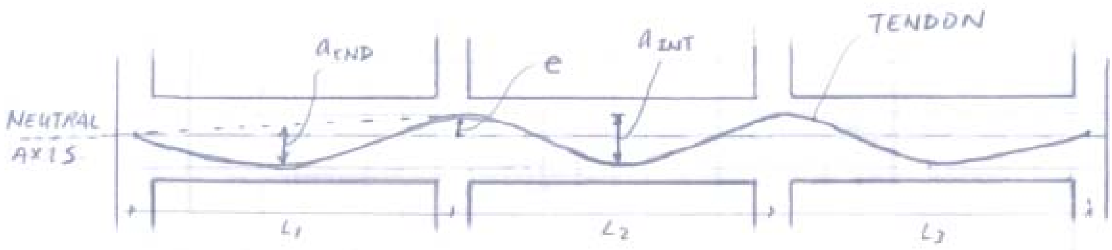
• TENDON PROFILE



* FIGURE NOT DRAWN TO SCALE

TENDON ORDINATE	TENDON CENTER OF GRAVITY LOCATION
EXTERIOR SUPPORT - ANCHOR	4.0"
INTERIOR SUPPORT - TOP	7.0"
INTERIOR SPAN - BOTTOM	1.0"
END SPAN - BOTTOM	1.75"

- LOCATION IS MEASURED FROM BOTTOM OF SLAB
- e : ECCENTRICITY ; IS THE DISTANCE FROM THE CENTER TO TENDON TO THE NEUTRAL AXIS, WHICH VARIES ALONG THE SPAN

$$a_{INT} = 7.0" - 1.0" = 6.0"$$
$$a_{END} = (4.0" + 7.0")/2 - 1.75" = 3.75"$$

• REQUIRED PRESTRESS FORCE TO BALANCE 75% OF SELFWEIGHT DL

- DUE TO REDUCED TENDON DRAPE AT a_{END} AND THE EQUIVILENT LENGTH SPANS, THE END SPAN WILL GOVERN THE MAXIMUM REQUIRED POST-TENSION FORCE.

- DESIGNER SPECIFIED EQUIVALENT BALENCING LOAD : ω_b

$$\omega_b = 0.65\,\omega_{DL} = 0.65(100\text{ psf})(20\text{ ft}) = 1300\text{ plf} = 1.30\text{ k/ft}$$

- FORCE NEEDED IN TENDONS TO COUNTERACT THE LOAD IN END BAY

$$P = \omega_b L^2 / 8 a_{END}$$
$$= (1.30)(29)^2 / [8(3.75/12)]$$

$$P = 437.3\text{ KIPS}$$

Stephen Dung Tat The Pennsylvania State University Architectural Engineering	Thesis: **Ingleside at King Farm**	Spot Check Calculations

- CHECK PRECOMPRESSION ALLOWANCE
  - DETERMINE NUMBER OF TENDONS TO ACHIEVE 437.3 K

$\# \text{ TENDONS} = 437.3 / (26.6 \text{ k/TENDON})$
$= 16.6$
$\therefore$ USE 16 TENDONS

- ACTUAL FORCE FOR BANDED TENDONS

$P_{ACTUAL} = 16\,(26.6\text{ k}) = 425.6\text{ K}$

- BALANCED LOAD FOR THE END SPAN ADJUSTED

$w_{b_{END\,SPAN}} = (425.6/437.3)(1.4\text{ k/ft}) = \boxed{1.26\text{ k/ft}}$

- DETERMINE ACTUAL PRECOMPRESSION STRESS

$P_{ACTUAL}/A = (425.6)(1000)/(1920\text{ in}^2) = \boxed{221.7\text{ psi}}$

$221.7 > 125$ psi min ✓
< 300 psi max ✓

- CHECK INTERIOR SPAN FORCE

$P = (1.3\text{ k/ft})(29\text{ ft})^2 / [8\,(6''/12'')]$
$= 273.3 < 452.2\text{ k}$

$\therefore$ LESS FORCE IS REQUIRED IN THE CENTER BAY

$w_{b_{MID\,SPAN}} = (452)(8)(6''/12'')/(29\text{ ft})^2 = \boxed{2.02\text{ k/ft}}$

$w_b / w_{DL} = 101\%$ Acceptable ✓

$\therefore P_{eff} = 452$ kips

$w_{b_{AVG}} = (w_{b\,END\,SPAN} + w_{b\,INT\,SPAN} + w_{b\,END\,SPAN})/3$

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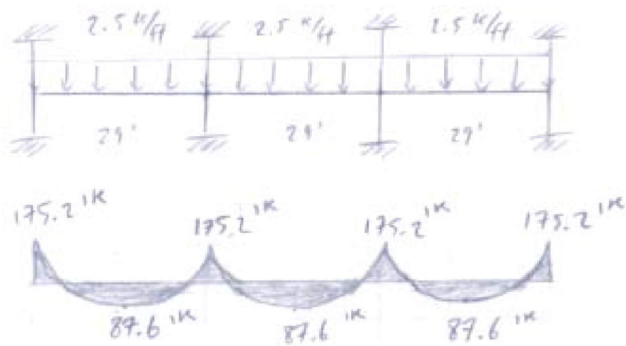
• CHECK SLAB STRESSES

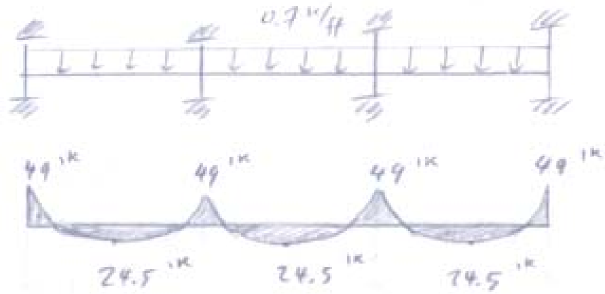
DEAD LOAD MOMENTS



$w_{DL} = 125\text{ psf}(20')/1000$

$w_{DL} = 2.5\text{ k/ft}$

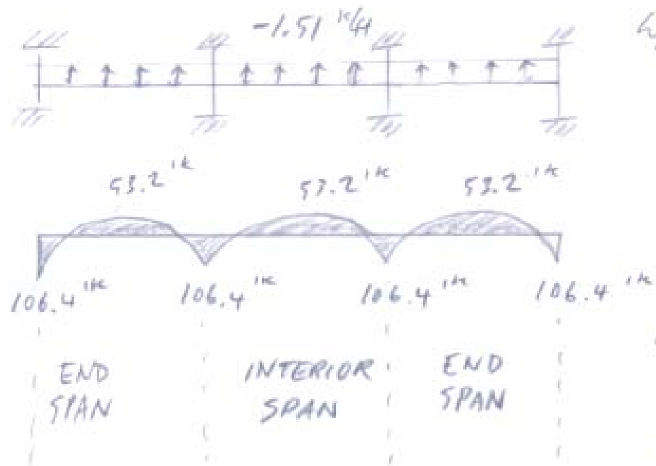
LIVE LOAD MOMENTS



$w_{LL} = 35\text{ psf}(20')/1000$

$w_{LL} = 0.7\text{ k/ft}$

TOTAL BALANCING MOMENTS, M_{bal}



$w_{b_{AVG}} = (w_{b\,END} + w_{b\,INT} + w_{b\,END})/3$

$= \dfrac{1.26 + 2.02 + 1.26}{3}$

$= -1.51\text{ k/ft}$

IMPORTANT: BE MINDFUL OF THE TERMS "INTERIOR SPAN" AND "MID SPAN" IN CALCULATIONS ON NEXT FEW PAGES.

Stephen Dung Tat **The Pennsylvania State University** **Architectural Engineering**	Thesis: **Ingleside at King Farm**	Spot Check Calculations

• STAGE 1 : STRESSES IMMEDIATELY AFTER JACKING (DL+PT) (ACI 18.4.1)

- MID SPAN STRESSES

$f_{top} = (-M_{DL} + M_{bal})/S - P/A$

$f_{bot} = (+M_{DL} + M_{bal})/S - P/A$

* · INTERIOR SPAN OR END SPAN

$f_{top} = [(-87.6 + 53.2)(12)(1000)]/(2560) - 221.6$
$= -382.9 < 0.60 f'_{ci} = 1800$ psi . . . . . ✓ ok

$f_{bot} = [(87.6 - 53.2)(12000)]/(2560) - 221.6$
$= -60.34 < 0.60 f'_{ci} = 1800$ psi . . . . . ✓ ok
$< 0.30 f'_{ci} = 164$ psi (END SPAN) ✓ ok

\+ · SUPPORT STRESSES

$f_{top} = (+M_{DL} - M_{bal})/S - P/A$

$f_{bot} = (-M_{DL} - M_{bal})/S - P/A$

$f_{top} = [(175.2 - 106.4)(12000)]/(2560) - 221.6$
$= 100.9 < 3\sqrt{f'_{ci}} = 164$ psi . . . . . ✓ ok

$f_{bot} = [(-175.2 + 106.4)(12000)]/(2560) - 221.6$
$= -544.2 < 0.60 f'_{ci} = 1800$ PSI . . . . ✓ ok

• STAGE 2 : STRESSES AT SERVICE LOAD (DL + LL + PT) (ACI 18.3.3, 18.4.2)

· MID SPAN STRESSES

$f_{top} = (-M_{DL} - M_{LL} + M_{bal})/S - P/A$

$f_{bot} = (+M_{DL} + M_{LL} - M_{bal})/S - P/A$

* · INTERIOR SPAN OR END SPAN

$f_{top} = [(-87.6 - 24.5 + 53.2)(12000)]/(2560) - 221.6$
$= -497.9 < 0.45 f'_c = 2250$ Psi . . . . . . ✓ ok

$f_{bot} = [(+87.6 + 24.5 - 53.2)(12000)]/(2560) - 221.6$
$= 54.6 < 6\sqrt{f'_c} = 424$ Psi . . . . ✓ ok

*NOTE : THE MID SPAN STRESSES FOUND AT BOTH INTERIOR OR END SPAN IS THE SAME BECAUSE OF HAVING THE SAME BENDING MOMENTS : $wL^2/24$

\+ NOTE : SUPPORT STRESSESS ARE FOUND AT COLUMN

Stephen Dung Tat The Pennsylvania State University Architectural Engineering	Thesis: **Ingleside at King Farm**	Spot Check Calculations

· SUPPORT STRESSES

$f_{top} = (+M_{DL} + M_{LL} - M_{bal})/S - P/A$

$f_{bot} = (-M_{DL} - M_{LL} + M_{bal})/S - P/A$

$f_{top} = [(+175.2 + 49.1 - 106.4)(1200)]/(2560) - 221.6$
$= 330.8 < 6\sqrt{f'c} = 424$ psi .... ✓ ok

$f_{bot} = [(-175.2 - 49.1 + 106.4)(1200)]/(2560) - 221.6$
$= -774.2 < 0.45f'_c = 2250$ psi ...... ✓ ok

∴ ALL STRESSE ARE WITHIN THE PERMISSIBLE CODE LIMITS.

ULTIMATE STRENGTH

$M_1 = P(e)$ IS THE PRIMARY POST-TENSIONING MOMENTS

$e = 0''$ at the exterior support

$e = 3.0''$ at the interior support (neutral axis to the center of tendon).

$M_1 = 425.6k(0'')/12 = 0$

SECONDARY POST-TENSIONING MOMENTS M_{sec}

$M_{sec} = M_{bal} - 0 = 106.4 - 0 = 106.4$

$M_{sec} = 0^{'k}$ at the interior supports

TYPICAL LOAD COMBINATION FOR ULTIMATE STRENGTH DESIGN

$M_u = 1.2\,M_{DL} + 1.6\,M_{LL} + 1.0\,M_{sec}$

AT MIDSPAN: $M_u = 1.2(87.6) + 1.6(24.5) + 106.4 = 250.7^{'k}$

AT SUPPORT: $M_u = 1.2(-175.2) + 1.6(-49.1) + 106.4 = 182.4^{'k}$

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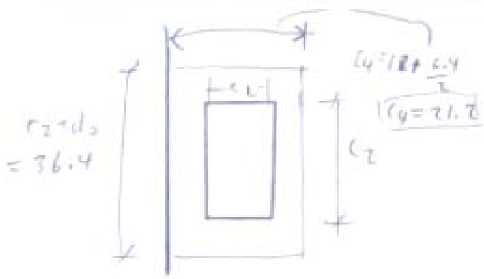
SHEAR CAPACITY IN EXTERIOR EDGE COLUMN (30"x18")



$C_1 = 18 \qquad d_0 = 0.8(8) = 6.4"$

$C_2 = 30$

$C_4 = 21.2$

C

$$A_c = (b_1 + 2b_2) \times d_0 = (2C_1 + C_2 + 2d_0)d_0$$
$$= (36 + 30 + 12.8)12.8$$
$$= 1008.64\ in^2$$

$$J_c = \frac{(C_1 + d_0/2)d_0^3}{6} + \frac{2d_0(C_3^3 + C_4^3)}{3} + d_0(C_2 + d_0)C_3^2$$

$$C_3 = \frac{d_0\left(C_1 + \frac{1}{2}d_0\right)^2}{A_c} = \frac{6.4(18 + .5(6.4))^2}{1008.64} = 2.85"$$

$$J_c = \frac{(18 + 6.4/2)6.4^3}{6} + \frac{2(6.4)(2.85^3 + 21.2^3)}{3} + 6.4(30 + 6.4)2.85^2$$

$$J_c = 926.2 + 40752.1 + 1892.2 = 43570\ in^3$$

$$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\frac{C_1 + d_0/2}{C_2 + d_0}}} = 0.337$$

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$$V_u = V_{ext}\left(\tfrac{1}{2}\right)\left(\tfrac{29}{2}\right) = M_{u\,(support)}$$

$$V_u = \frac{(182.4)(4)}{29} = 25.2\ k$$

$$M_u = M_{u\,(support)} = 144.32^{\ 'k}$$

$$v_u = \begin{cases} \dfrac{V_u}{A_c} + \dfrac{\gamma_v M_u c_4}{J_c} = \dfrac{25.2 \times 1000}{1008.64} + \dfrac{0.337(144.3)(21.2)(12000)}{43570} \\[2ex] \dfrac{V_u}{A_c} + \dfrac{\gamma_v M_u c_3}{J_c} = \dfrac{25.2 \times 1000}{1008.64} + \dfrac{0.337(144.3)(2.85)(12000)}{43570} \end{cases}$$

$$v_u = \begin{cases} 24.9 + 13.39(21.2) = \boxed{308} \leftarrow \text{governs} \\ 24.9 + 13.39(2.85) = 63.0\ psi \end{cases}$$

$$b_o = (2c_1 + c_2 + 2d_o) = (2\times 18 + 30 + 2\times 6.4) = 78.8$$

$\alpha_s = 30$ for edge column

$$\beta_c = 30/18 = 1.667$$

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$$v_c = \text{smallest of} \begin{cases} \left(2+\frac{4}{\beta_c}\right)\sqrt{f'_c} = 4.399\sqrt{f'_c} \\ \left(2+\frac{\alpha_s d_o}{b_o}\right)\sqrt{f'_c} = 4.437\sqrt{f'_c} \\ \boxed{4\sqrt{f'_c}} \Leftarrow \text{governs} \quad (f'_c = 5000\ \text{psi}) \end{cases}$$

$$v_c = 283 \text{ psi}$$

$$\phi v_c = 0.75 \times 283 = 212.3 \text{ psi}$$

$$v_u = 308 \text{ psi} > 212.3 \text{ psi}$$

$\therefore$ NG FAILS IN PUNCHING SHEAR

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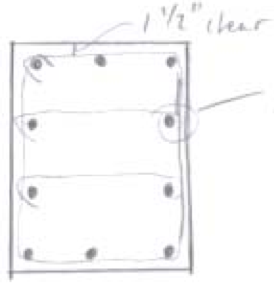
COLUMN DESIGN



- SIZE : 30" x 18"
- FIRST FLOOR (EXTERIOR EDGE COLUMN)
- (ACI 318)
- $f'c = 5000$ psi $\quad f_y = 60$ ksi

$$\phi P_n = (0.8)\phi[0.85 f'c (A_g - A_{st}) + f_y A_{st}] \quad \text{(ACI 10.3.6.2)}$$

$$= (0.8)0.65[0.85(5)[(30 \times 18) - (10 \times 0.79)] + 60(10)(0.79)]$$

$$= 1422.4 \text{ KIPS}$$

TRIB AREA PER FLOOR :

$$A_{TRIB} = (20' \times 29/2) = 290 \text{ ft}^2$$

DEAD LOADS :

CONCRETE SLAB 150 PCF (8"/12)	= 100 PSF	5 floors
PARTITIONS	= 15 PSF	5 floors
MEP	= 10 PSF	5 floors
ROOF GARDEN	= 100 PSF	× (1 floor)
EXTERIOR WALL (BRICK & CHANNELS)	= 45 PSF	× (5 floors)
JOISTS	= 5 PSF	× (ROOF)
TOTAL	= 955 PSF	

LIVE LOADS

RESIDENCES	40 PSF	× (5 floors)
TOTAL	= 200 PSF	

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$P_u = 1.2\,DL + 1.6\,LL$

$= 1.2(955\text{ PSF} \times 290\text{ ft}^2) + 1.6(200\text{ PSF} \times 290)$

$P_u = 332\text{ K} + 29.8\text{ K}$

$P_u = 362\text{ KIPS}$

$\phi P_n = 1422.4\text{ KIPS} > P_u$ ✓ ok