

Trump Taj Mahal Hotel

Atlantic City, New Jersey



Technical Report Number One

Investigation of the Structural System and Existing Conditions

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Executive Summary

The proceeding report is a technical description of structural concepts and existing conditions for the Trump Taj Mahal Hotel, currently being constructed in Atlantic City, New Jersey. The first technical section of the report provides a listing of structural codes and material strengths that The Harman Group, the structural engineers who designed the structure of the building, has specified in their design. These sections are followed by detailed descriptions of the structural systems of the tower. Descriptions of the foundation system, columns, floor systems, miscellaneous framing, and lateral force resisting systems are included with various diagrams, plans and illustrations to aid explanations. The next section provides a detailed analysis of gravity (including self weight), snow, wind, and seismic loads per ASCE 7-05 and IBC 2006. Calculations of these loads can be found in the appendix of the report. The final section provides commentary on the spot checks performed on one of the shear walls (Level 23), the filigree flat slab system, an in-slab beam, and a gravity column. All analyses calculations can be found in the appendix of this report.

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I. Introduction

Atlantic City is known as the “Las Vegas” of the east coast. It is home to some of the largest and finest hotels, resorts, and casinos, as well as one of the largest boardwalks in the world. Donald Trump came to Atlantic City with a vision to create one of the world’s finest casinos along with Atlantic City’s most luxurious hotels. At the 900 block of the Atlantic City boardwalk in 1990, Trump unveiled the first Taj Mahal Hotel, unprecedented in craftsmanship and opulence. Its stern use of iconic architecture, rich with lights and signage, matches that of the rest of Atlantic City.

The Trump Taj Mahal Hotel Tower at 1000 Boardwalk resembles a powerful type of iconic architecture, signifying the power and wealth of Donald Trump along with the luxury you can expect from such a hotel. Such iconic characteristics that are clearly expressed on the building include large, bold signage (Both the Taj Mahal running down the east and west sides of the building and Trump across the top of the building.), a unique and pure geometric plan that rivals its neighboring predecessor, and it’s overwhelming height as compared to the neighboring buildings along the ocean front skyline. The facade of the building is constructed with mostly modern materials, comprised of a reflective glass curtain wall, metal panels, and architectural pre-cast concrete panels.

The new Taj Mahal Hotel will serve as an expansion to its older and neighboring hotel tower that was built in the early 1990s. It will provide an additional 786 guest suites, ranging from spacious single rooms to deluxe 3 bay super suites. The tower will have 732,000 square feet of usable space and will soar 435 feet, 40 stories, into the air, making it an icon in the view of the Atlantic City skyline.

II. Codes

The following codes were referenced in the design of the structural system of the Trump Taj Mahal Hotel. The same codes and references for gravity, wind, and seismic loads have been used for the load analysis portion of this report.

Building Code:

New Jersey State Uniform Construction Code (IBC 2000)

Loads:

Minimum Design Loads for Buildings and Other Structures, ASCE 7-02
American Society of Civil Engineers

Comment: Standards of ASCE 7-02/7-05 are referenced by IBC

Structural Concrete:

ACI 318-02 – Requirements for Structural Concrete
American Concrete Institute

Manual of Standard Practice, 27th Edition, March 2002
Concrete Reinforcing Steel Institute

Structural Steel:

Steel Construction Manual, 13th Edition
American Institute of Steel Construction

Detailing for Steel Construction
American Institute of Steel Construction

Welding:

Structural Welding Code – Steel, AWS D1.1-2002

Structural Welding Code – Reinforcing Steel, AWS D1.4-1998

Metal Decking:

Design Manual for Floor Decks and Roof Decks
Steel Deck Institute

III. Material Strengths

The following tables list the design strengths and properties of various building materials, as specified by the structural engineering consultant.

Concrete Compressive Strengths

Location	f'c @ 28 Days (PSI)	Unit Wt. (PCF)
Mat Foundation	5000	145
Topping Slabs	3000	145
Normal Wt. Slabs on Metal Deck	3500	145
Slabs on Grade	4000	145
Walls (Other Than Shear Walls)	4000	145
Framed Slabs and Beams	5000	145
Columns: Below Level 12	9000*	145
Levels 12 to 23	7000*	145
Above Level 23	5000	145
Shear Walls: Below Level 12	9000*	145
Levels 12 to 23	7000*	145
Above Level 23	5000	145

* Indicates 56 – Day Strength

Reinforcing Steel

Deformed Reinforcing Bars #10 and Smaller #11 and Larger	ASTM A615, Grade 60 ASTM A615, Grade 75
Weldable Deformed Reinf Bars	ASTM A706
Welded Wire Fabric (WWF)	ASTM A185
Seven-Wire Stress Relieved Prestressing Strands	ASTM A416, Grade 270
Epoxy Coated Reinf Bars	ASTM A775
Reinforcing Steel Mechanical Splice Couplers	Lenton Splice Couplers or Approved Equal
Doweling Adhesive for Anchoring Reinf Bars into Existing Concrete	Hilti System or Powers Acrylic 100 System

Structural Steel

W Shapes	ASTM A992	Fy=50ksi
Channels, Angles, Plates and Bars	ASTM A36	Fy=36ksi
Round Pipe	ASTM A53 Grade E or S	Fy=35ksi
Square and Rectangular HSS	ASTM A500, Grade B	Fy=46ksi
High Strength Bolts	ASTM A325 Typ. UNO and ASTM A490 Where Indicated	
Anchor Rods	ASTM F1554 Grade 55 w/ Section SI Weldability Supplement or Grade 105 (Where Indicated)	
Round Rods and Threaded Rods	ASTM A36	
Headed Shear Studs	ASTM A108	
Expansion Bolts	Powers "Power-Stud" or Hilti "Kwik Bolt"	
Adhesive Anchors	Powers "Acrylic-100" System or Hilti "Hit Hy 150" System	
Wedge-Bolts	Powers "Wedge-Bolt"	
Sleeve Anchors	Powers "Lok/Bolt" or Hilt "Sleeve Anchor"	
Galvanized Metal Deck	ASTM A653, Grade 40 (Fy=40ksi)	

IV. Structural Systems

The proceeding section contains detailed descriptions of the various structural systems that have been incorporated into the design of the Trump Taj Mahal Hotel. Descriptions of the foundation system, columns, floor systems, miscellaneous systems, and lateral system are provided and follow in that respective order. Figure 1 provides an illustration of the framing plan of a typical level of the tower.

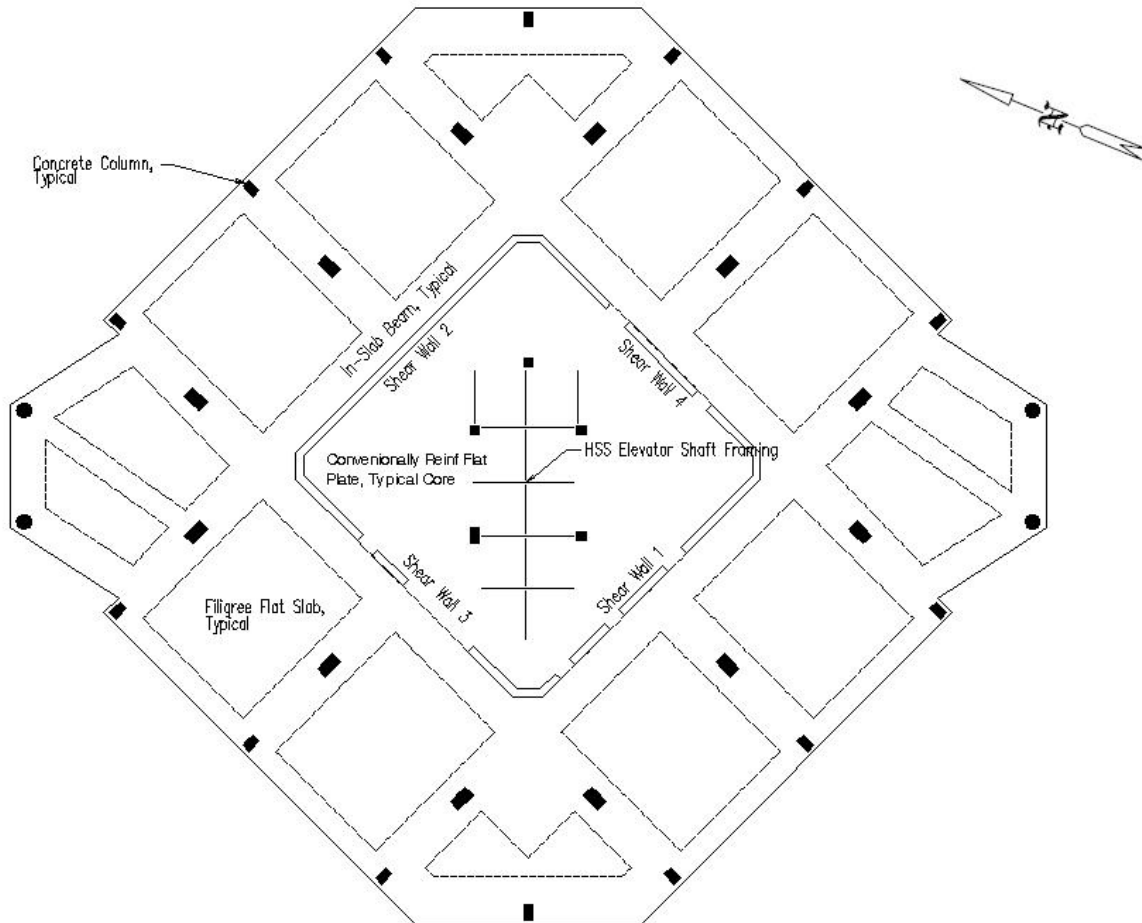


Figure 1: Typical Framing Plan

Foundation System

The foundation system of the Trump Taj Mahal Hotel is comprised of a mat foundation, as recommended by the geotechnical report. The perimeter of the mat foundation is 6'-0" thick, the center 9'-0" thick. #11 bars at 10" each way, top and bottom are provided for the 9'-0" section and #11 at 15" each way, top and bottom are provided for the 6'-0" section. Additional reinforcing is provided around openings and columns. The mat foundation acts as the floor system of level one, no topping slab provided.

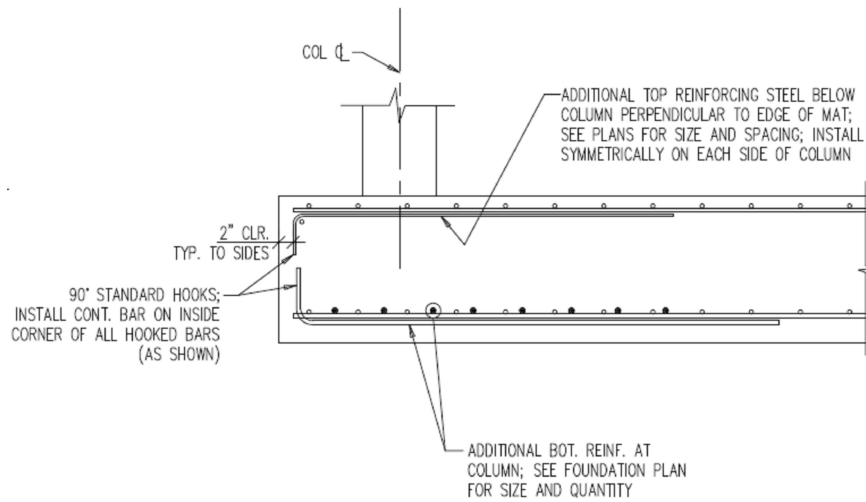


Figure 2: Typical Section at Mat Foundation

Columns

Square, rectangular, and round reinforced concrete columns are used throughout the hotel tower, with a wide range of sizes and reinforcing arrangements. Figure 3 provides a typical detail that illustrates the tie arrangements, vertical reinforcing steel arrangements, and dimensions of the columns that are found throughout the tower. Specified compressive strength of concrete used for the columns varies by level, generally higher at lower levels. See Section III (Material Strengths) for details.

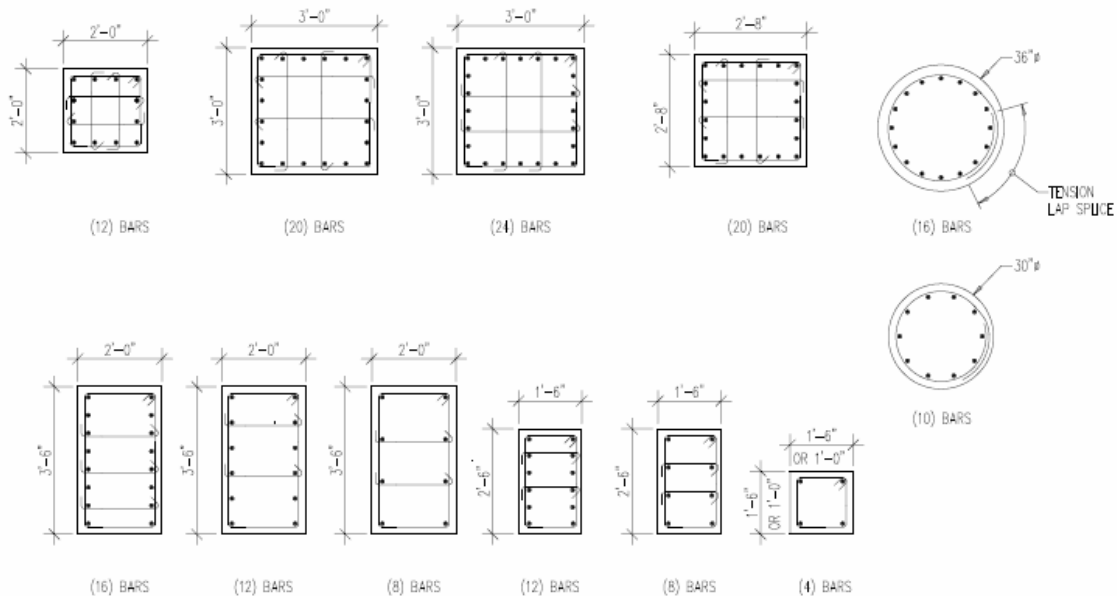


Figure 3: Detail of Typical Column Types

Floor Systems

Two types of floor systems are used on a typical level of the hotel tower. A one-way pre-stressed filigree flat plate system is utilized in the areas outside of the central elevator core. Inside of the core, a conventionally reinforced flat plate system is utilized. 5000psi is the specified compressive strength of both systems.

A filigree flat plate floor slab acts as a composite system, utilizing both pre-cast and cast-in-place components. 8'-0" wide 2 ¼" thick pre-stressed planks form the base of the system. Foam voids are cast on top of the planks, lowering the dead weight of the system. However, some floors of the tower with higher loads may have solid slabs instead of voided slabs. A layer of concrete is poured on top of the planks and 2 ¼" on top of the voids, if present. 10x10 W4xW4 Welded Wire Fabric is used as temperature reinforcing for the cast -in-place concrete.

The loads of the filigree flat slab are transferred to the columns via 8'-0" wide conventionally reinforced in-slab beams that run 32'-0" x 16'-0" bays, typically. The filigree flat slabs are connected to the in-slab beams by reinforcing dowels, typically #7 bars on the top layer. The base of the beams are formed using the filigree planks, however the prestressed tendons are not utilized in the design strength of the beam.

Please note, because this particular type of filigree system is proprietary to Mid-State Filigree, construction documents issued by the structural engineering consultant only indicate design moments. Reinforcing of the filigree flat slab system can be found on shop drawings issued by the filigree contractor (See Appendix 1).

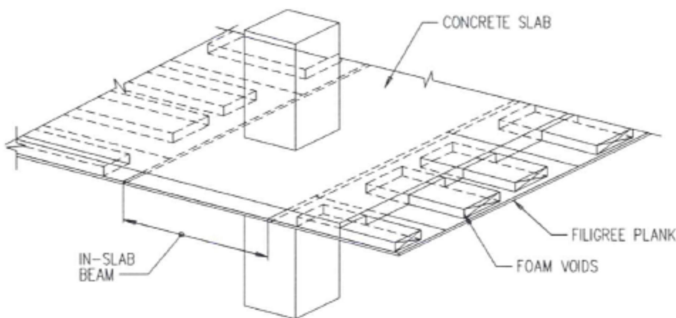


Figure 4: Filigree Flat Plate System



Figure 5: Filigree Construction Photo

Filigree Flat Slab System (Non-Core)

The proceeding diagram describes the various filigree flat slabs, by level number.

Level Number	Solid or Voided	Total Depth (inches)
2, 3	Voided	12
4	Solid	10
5 thru 39	Voided	10
40	Solid	12
41	Solid	10

Conventionally Reinforced Flat Plate System (Core)

The proceeding diagram describes the various conventionally reinforced flat plate slabs, by level number.

Level	Reinforcing	Thickness (inches)
2, 3	#6 @ 12" Bottom, Each Way	12
4	#7 @ 12" Bottom, Each Way	10
5 thru 39	#6 @ 12" Bottom, Each Way	10
40	#6 @ 12" Bottom, Each Way	12
41	#7 @ 12" Bottom, Each Way	10

Miscellaneous Framing

Level 3 – Catwalk

A catwalk that houses mostly MEP equipment above level 3 that encompasses the elevator core of the tower is framed using W shape beams. This steel framing is supported by both the concrete shear walls and concrete columns. The steel beams are connected to the concrete using embed plates with shear studs. 2" of bar grating serves as a floor for the catwalk.

Sign Support Framing (Level 41 to Top of Sign)

The Trump sign at the top of the hotel tower is supported by HSS girts, supporting the maximum sign weight of 550plf. Two lines of columns, typically W14x61, post up from the concrete floor system of the 41st level, forming the perimeter lines of the system. Another line of columns, typically W24x68, posts up at the center of the original two lines from transfer girders, making three column lines. W16x67 and W24x68 are the typical girder sizes. There are a total of 7 bays, varying in span length.

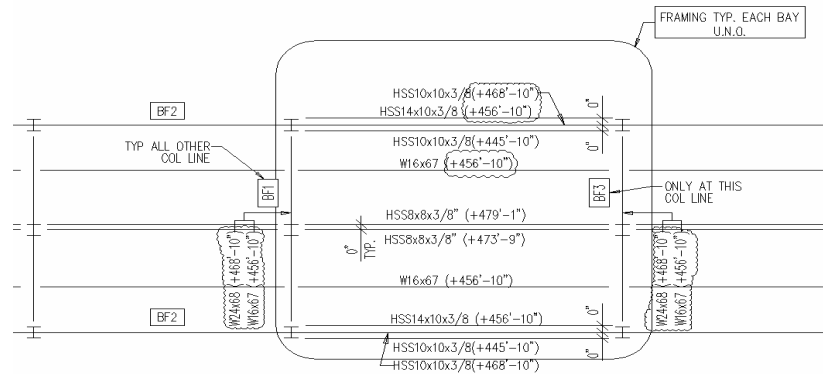


Figure 6: Typical Framing Plan at Sign Support

Elevator Separator/Support Framing

Elevator shafts are separated using a rectangular grid of HSS beams. The HSS beams are also used to resist the thrust force produced by the elevator systems. These beams tie to both the two-way slab floor system and the concrete columns by connecting to embed plates. See Appendix 2 for typical elevator separator beam framing plan.

Connection Bridge

The bridge that connects the existing hotel to the new hotel is framed using a composite steel system with slab on metal deck. The system frames into the vertical elements of the existing hotel tower and two W shape columns outside the perimeter of the new hotel. An expansion joint between the floor slab of the bridge and the concrete slab of the new hotel separates the two systems.

Lateral Systems

The primary lateral force resisting system of the hotel tower is comprised of four shear walls, encompassing the elevator core at the geometric center of the tower’s plan. A series of braced frames are used to stiffen the sign support structure at the top of the tower.

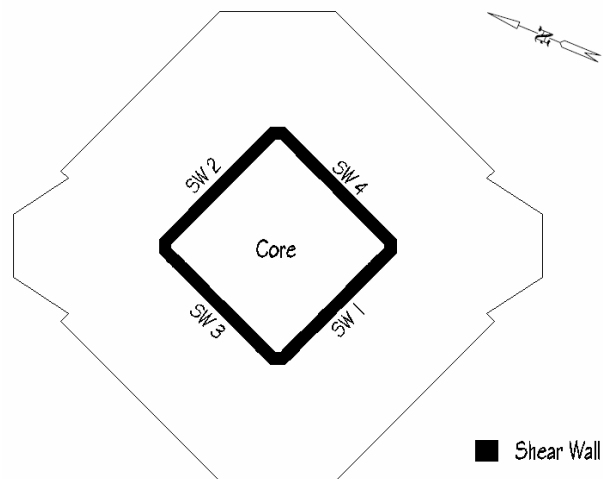


Figure 7: Location of Shear Walls

Reinforced Concrete Shear Walls

Four shear walls, spanning to level 41, are the primary lateral force resisting system of the Trump Taj Mahal Hotel. Two 60’ long walls resist the forces in the east/west direction, as well as the north/south direction. These four walls

form the elevator core that lies in the geometric center of the tower. Because of the symmetry of both the plan of the building and the shear wall core, it is highly unlikely that torsion will control the design of the shear walls.

The shear walls decrease in thickness, 24" from levels 1 through 4 and 16" from levels 4 through 41. Because numerous openings exist, link (coupling) beams provide load transfer across the openings. Specified compressive strength of the concrete used for the shear walls varies by level (See Section III, Material Strengths). A detailed elevation of each shear wall is provided in Appendix 3.

Braced Frames

Because the framing system supporting the large sign at the top of the tower is long and narrow, lateral bracing is needed to stiffen the system against strong wind forces. In the short (north/south) direction, seven X braced frames with single angle diagonals and one single strut braced frame with double angle diagonals.

The long (east/west) direction does not require much lateral stiffening because of its depth. Only two X braced frames with single angle diagonals are provided.

The loads of these braced frames are transferred to the concrete floor system on the 41st level below. The concrete floor system acts as a rigid diaphragm, transferring the loads to the concrete shear walls.

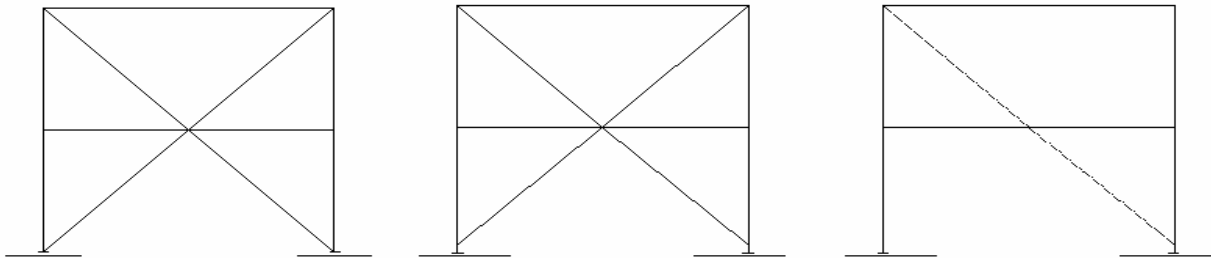


Figure 8: Braced Frame 1

Figure 9: Braced Frame 2

Figure 10: Braced Frame 3

Note: For location of braced frames, see Figure 6 in the sign support section of this report

V. Load Analysis

Gravity Loads

Self weights of building structural elements were tabulated (See Appendix 4) and are relatively close to the weights listed on The Harman Group's load maps. 145pcf, as specified by the structural engineer, was used for the unit weight of concrete. A 25% reduction (rule of thumb per Mid-State Filigree) was used on the self weight calculation of voided filigree slabs. 24" x 24" dimensions were assumed for every column for simplicity. Results of these calculations are provided in the following table.

Self Weight Loads

Level 2	12" Voided Slab:	110psf
	12" Two-Way Slab:	145psf
	Shear Walls:	905kips
	Columns:	240kips
Level 3	12" Voided Slab:	110psf
	14" Two-Way Slab:	170psf
	Shear Walls:	1,600kips
	Columns:	427kips
Level 4	10" Solid Filigree Slab:	120psf
	10" Two-Way Slab:	120psf
	Shear Walls:	1500kips
	Columns:	423kips
Levels 5 – 39	10" Voided Slab:	90psf
	10" Two-Way Slab:	120psf
	Shear Walls:	125kips
	Columns:	178kips
Level 40	12" Solid Slab:	145psf
	12" Two-Way Slab:	145psf
	Shear Walls:	980kips
	Columns:	260kips
Level 41	10" Solid Slab:	120psf
	10" Two-Way Slab:	120psf
	Shear Walls:	730kips
	Columns:	195kips

Superimposed dead loads for the tower are taken directly from the load maps provided by the structural engineer’s drawings. Snow loads were calculated using ASCE 7-05 (See Appendix 5). Live loads are taken directly from Table 4-1 of ASCE 7-05. A summary is provided in the following table.

Superimposed Dead Loads and Live Loads (Including Reduction if Applicable)

Level	Superimposed Dead Load	Live Load	Live Load Reduction Comments (ASCE 7-05)
1	Partitions: 15psf	100psf	Not Applicable
2	Non-Core Suspended Ceiling: 10psf Suspended MEP: 10psf Floor Finishes: 10psf Core Suspended Ceiling: 10psf Suspended MEP: 10psf Floor Finishes: 10psf	Non – Core: 150psf Core: 100psf	4.8.5 Limitations on One-Way Slabs
3	Non-Core Suspended Ceiling: 5psf Suspended MEP: 10psf Floor Finishes: 5psf Topping Slab: 10psf Core Suspended Ceiling: 5psf Suspended MEP: 10psf Floor Finishes: 5psf Topping Slab: 10psf	Non-Core: 150psf Core: 100psf	4.8.5 Limitations on One-Way Slabs
4	Non-Core & Core Partitions: 15psf Suspended MEP: 15psf	40psf	4.8.5 Limitations on One-Way Slabs
5 Thru 38	Non-Core & Core Partitions: 15psf	40psf	4.8.5 Limitations on One-Way Slabs

39	Non-Core Partitions: 15psf Floor Finishes: 10psf Core Partitions: 15psf	40psf	4.8.5 Limitations on One-Way Slabs
40	Non-Roof Suspended MEP 30psf Roof Snow Load 11.2psf	MEP: 150psf Roof: 20psf	4.8.5 Limitations on One-Way Slabs 4.9.1 Flat, Pitched and Curved Roofs
41	Non-Roof Suspended MEP 30psf Roof Snow Load 11.2psf	20psf	4.9.1 Flat, Pitched and Curved Roofs

Wind Loads

Wind pressures were calculated using Analytical Method II per ASCE 7-05, Main Wind Force Resisting Systems (MWFRS). A spreadsheet with calculations and parameters can be referenced in Appendix 6. In the actual analysis of the tower, the structural engineer had a wind tunnel test performed by DFA. The calculated base shear using Analytical Method II was approximately 3300kips, compared to 2000kips as determined by the wind tunnel test. A spreadsheet of the wind tunnel test results can also be found in Appendix 6.

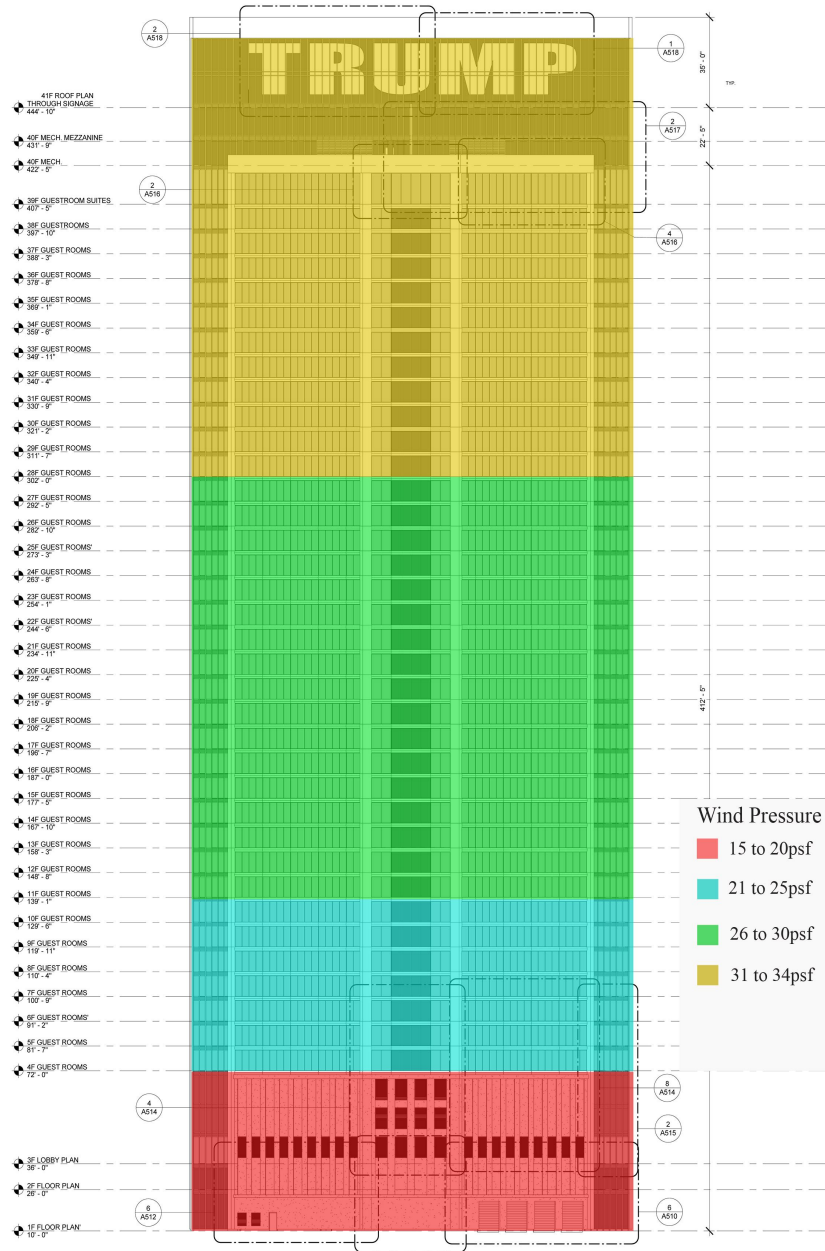


Figure 11: Windward Pressure Distribution
 Note: 21psf Leeward Pressure Not Shown

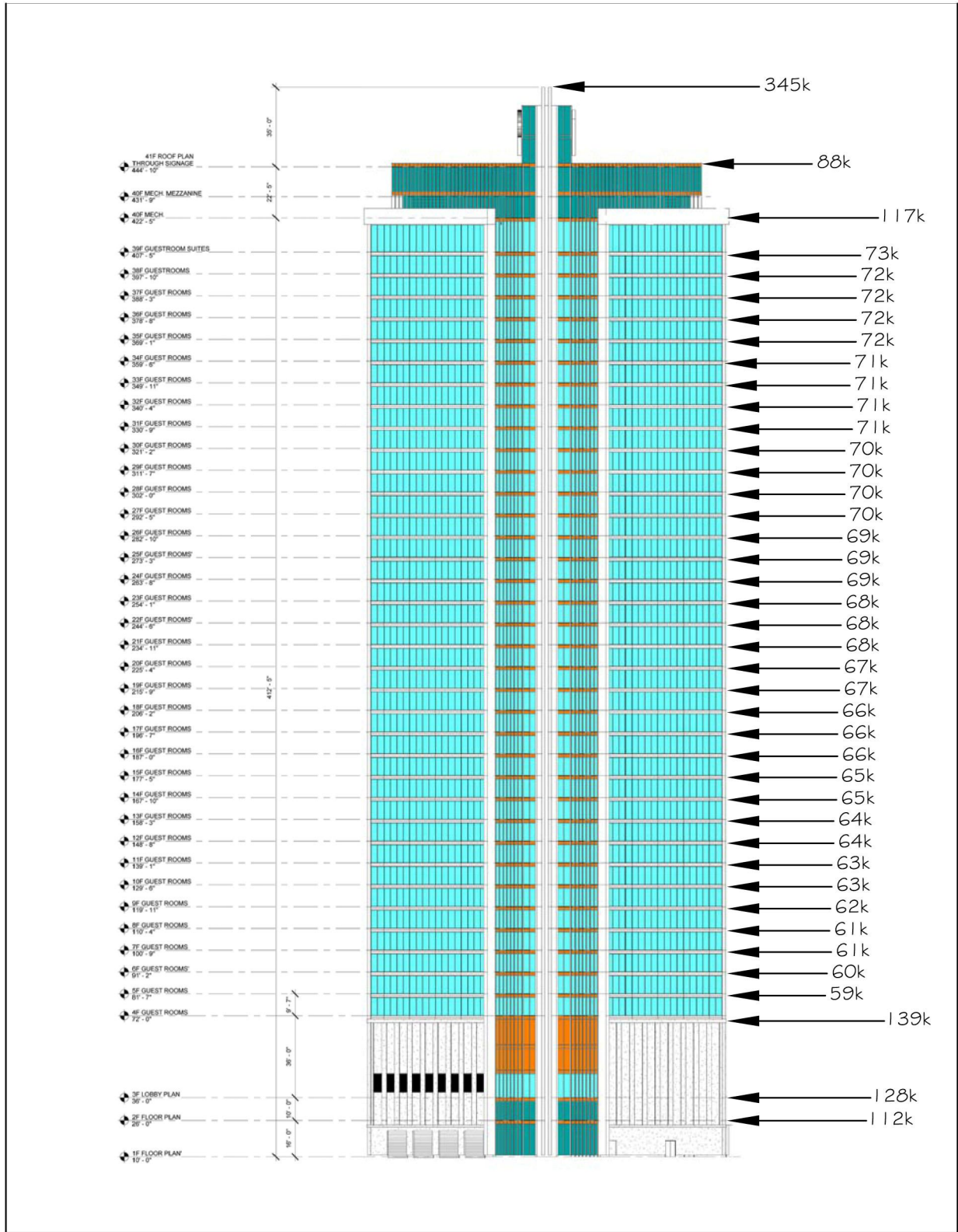


Figure 12: Wind Force Distribution North/South, ASCE 7-05

Seismic Loads

Seismic loads for the Trump Taj Mahal were calculated using ASCE 7-05, Equivalent Lateral Force Procedure. The calculations and parameters can be found in a spreadsheet referenced in Appendix 7 of this report. The base shear for both directions was calculated to be approximately 1086kips.

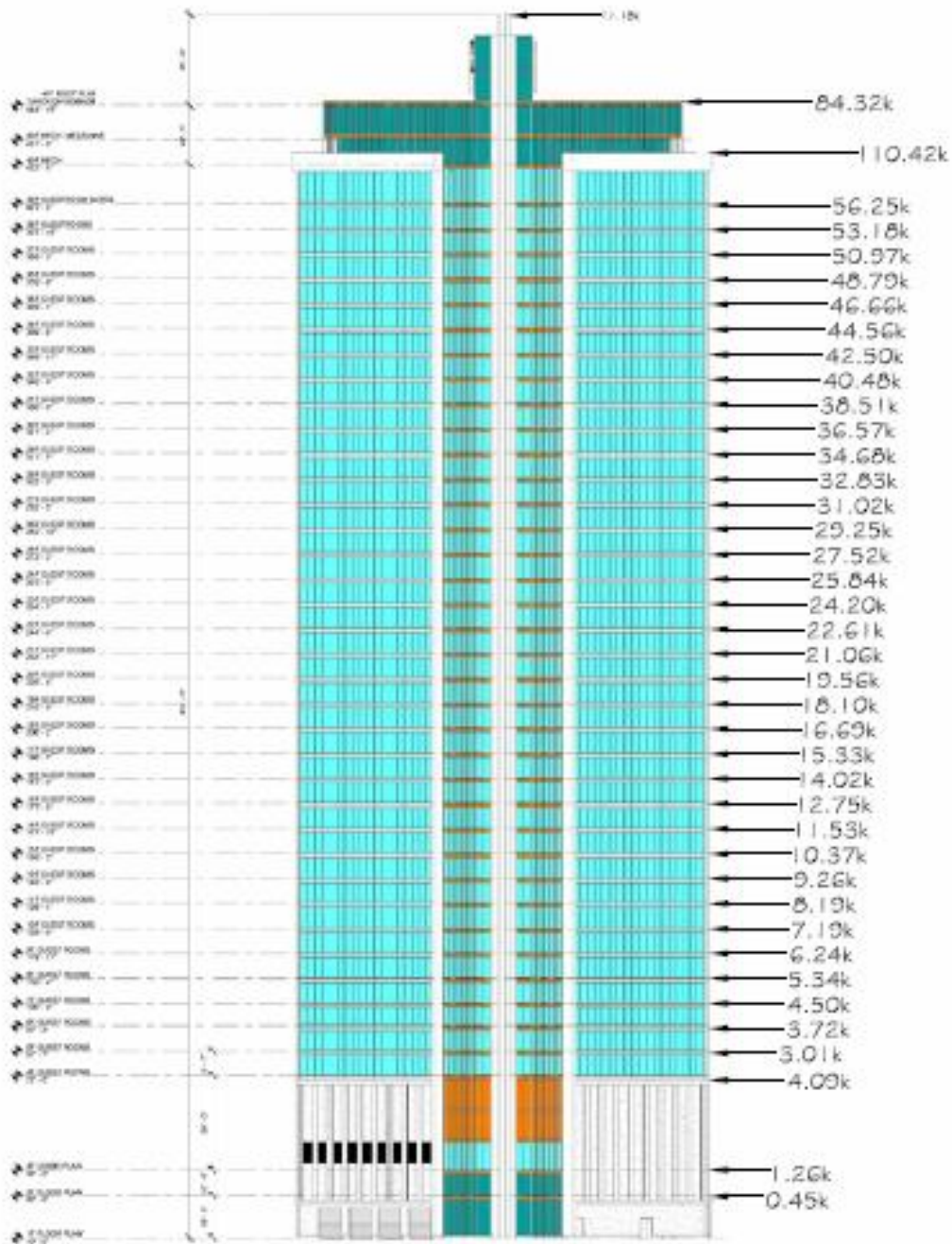


Figure 13: Seismic Force Distribution, Either Direction

VI. Structural Analyses

Several simplified structural analysis spot checks were performed on various structural elements of the tower. Conclusions of each analysis follow in the proceeding section. Calculations of each spot check can be found in the appendix of this report.

Shear Wall

Shear wall 2 (See Shear Wall Elevations in Appendix 3), oriented in the north/south direction, was chosen for a quick spot check analysis. Because its calculated base shear was higher than that of seismic, wind governed the design of the shear wall. The wind tunnel test results were used in the design of the shear wall.

Several simplified assumptions were made for the analysis. The distribution of lateral forces to the shear walls was done using the areas of the shear wall, accounting for reductions because of openings. For the calculation of a boundary element, it will be assumed that the adjacent shear wall can be utilized.

It was found that the vertical and horizontal steel required could be placed in only one curtain. This is different than the Harman Group's design, which provides two curtains of steel. The extra curtain could be used for deflection control.

When the necessity of a boundary element was checked, it was found that a boundary element was required. The calculated required amount of steel reinforcement was almost the same as the actual design. When tension was checked, it was found that additional steel was required.

Filigree Flat Slab

A typical 32'x16' filigree flat slab bay designed for levels 5 through 38 was analyzed for strength resistance of gravity loads. A filigree piece drawing illustrating the amount of provided reinforcement can be found in Appendix 1 for further reference.

The filigree slab was designed as a typical one-way slab system per ACI 318-05. It was assumed to be continuous over the middle support. However both ends were assumed to be pinned, meaning the in-slab beams are assumed to provide no torsional restraint against the moment from the slab.

Results yielded similar positive moment reinforcing requirements of that provided by Mid-State Filigree. It was found that fourteen 270ksi pre-stressing strands were adequate to resist the positive moment of the slab, compared to 17 that Mid-State Filigree is providing. This does however make sense because the planks are universally designed for each level. Since some of the other bays on each level have longer spans, the slabs will be subjected to higher moments.

Negative reinforcing requirements were very similar to The Harman Group's design, with just a small amount of additional reinforcement. Since it was assumed that no torsional restraint was provided by the in-slab beams at the two end spans, this would contribute to the increase in mid-span negative moments.

In-Slab Beam

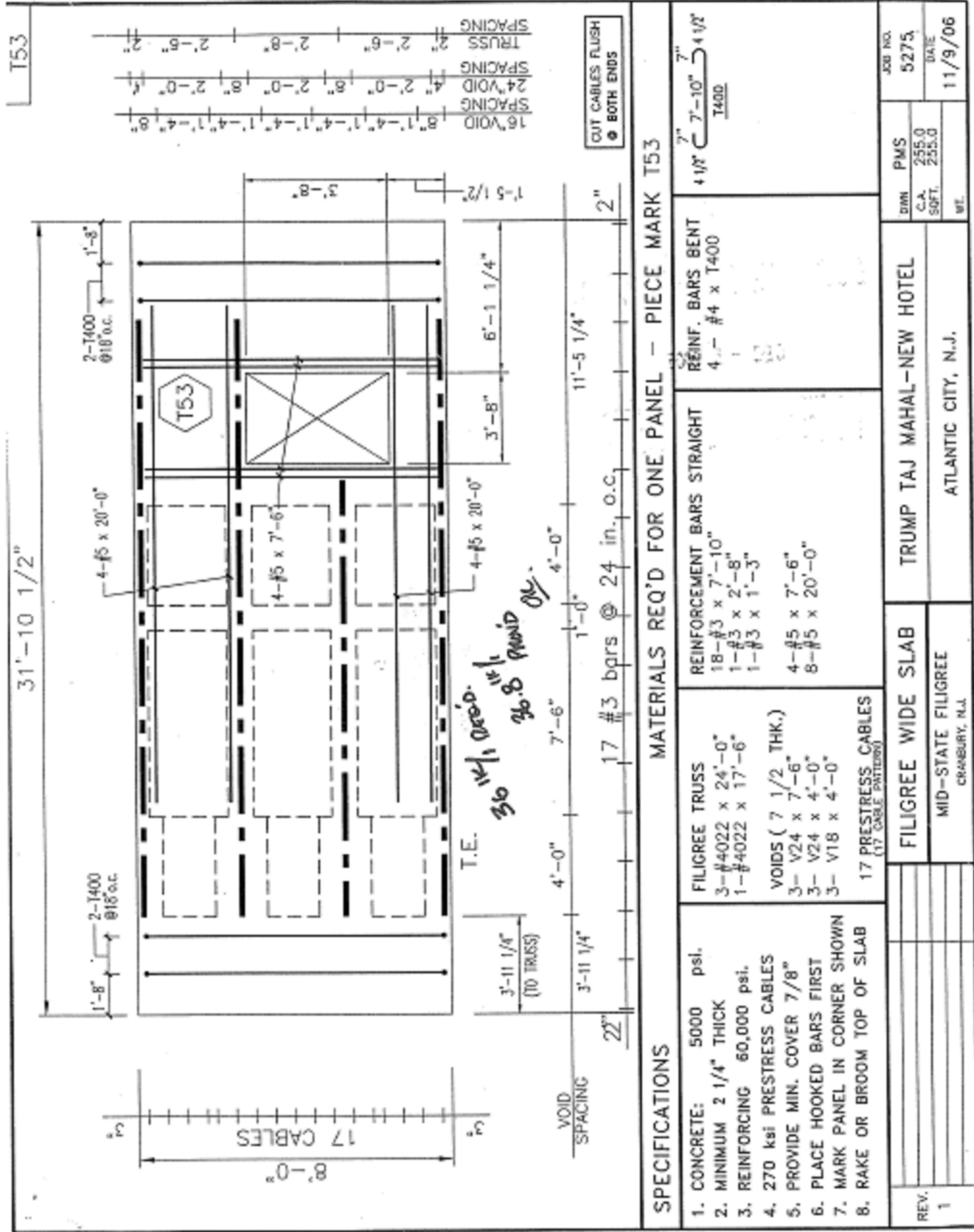
A typical 7 ¾" x 8'-0" in-slab conventionally reinforced beam with two-spans, 16' and 18'-9", was analyzed and designed for the resistance of gravity loads. ACI 318-05 coefficients of moment distribution were used to quickly calculate the negative and positive design moments. Results were very similar to The Harman Group's design. If a complete model of the building frame were to be analyzed, results may have been closer.

Gravity Column

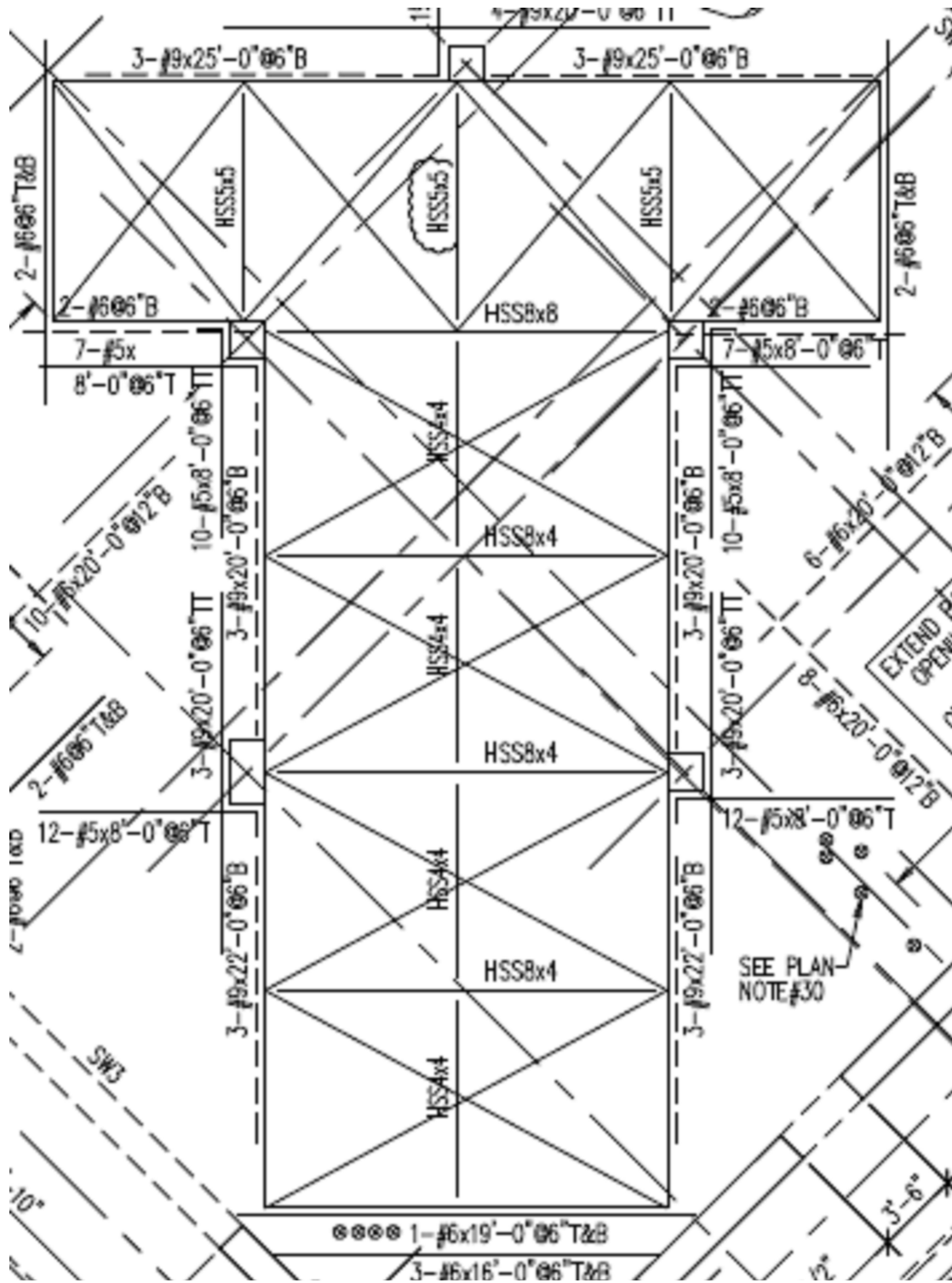
An exterior 18"x32" rectangular column at the intersection of gridlines G and 4 on level 38 was analyzed for resistance to gravity loads. Axial loads were calculated using the tributary area of the column and multiplying it by the factored dead, live, and self weights. The design moment was taken from the previous in-slab beam calculation utilizing ACI coefficients.

Once the loads were determined, a quick interaction diagram was drawn using PCA column. The loading of the column falls well within the safe region of the interaction diagram. The column appears to be over designed. This makes some sense because this exact column is used on many floors of the tower for redundancy.

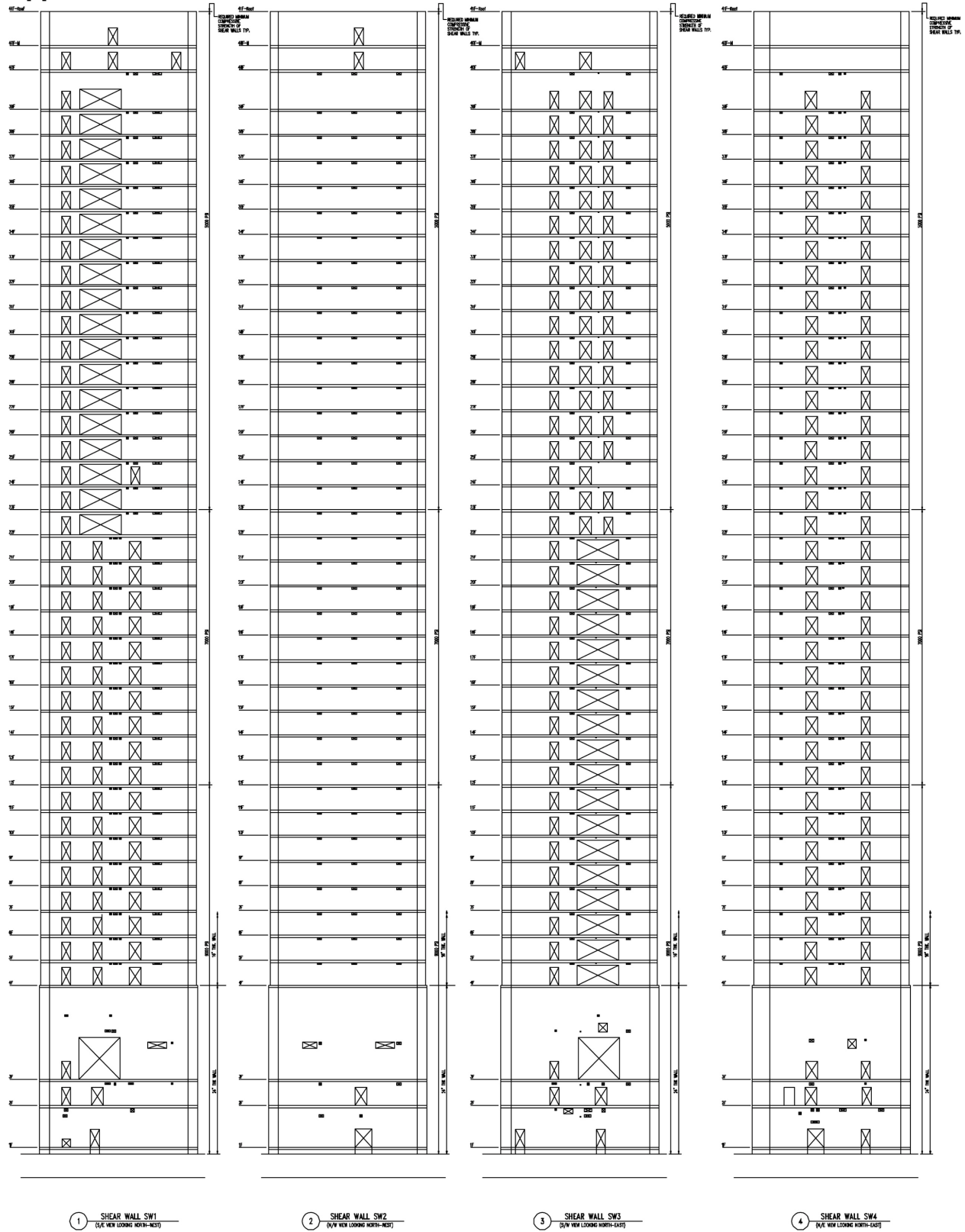
Appendix 1: Typical Filigree Floor Slab Piece Drawing



Appendix 2: Typical Elevator Shaft Framing Plan



Appendix 3: Shear Wall Elevations



Appendix 4: Self Weight Calculations

Architect: _____ Sheet: 1 of 4
 Project: TECH No 1 Job No.: _____
GMV LOADS Date: 9/26/07
 Engineer: SMR

SELF WEIGHT CALCS

LEVEL 2

- 12" VOIDED SLAB ↙ 25% REDUCTION FOR VOIDS
 $\frac{12''}{12''} (145 \text{ pcf}) (0.75) = 109 \text{ psf}$ USE 110 psf
- 12" TWO WAY SLAB
 $\frac{12''}{12''} (145 \text{ pcf}) = \underline{145 \text{ psf}}$
- SHEAR WALLS
 $\frac{24''}{12''} (145 \text{ pcf}) (13') (60') (4) = \underline{905 \text{ k}}$
- COLUMNS → ASSUME 2'x2' COLUMNS, ALL FIES
 $\frac{24''(24'')}{144} (13') (32) (145) = 240 \text{ k}$

LEVEL 3

- 12" VOIDED SLAB = 110 psf
- 14" TWO WAY SLAB
 $\frac{14''}{12''} (145 \text{ pcf}) = \underline{170 \text{ psf}}$
- SHEAR WALLS
 $\frac{24''}{12''} (145) (60') (4) (23') = \underline{1,600 \text{ k}}$
- COLUMNS
 $\frac{(24)(24)}{144} (23) (32) (145) = 427 \text{ k}$

Architect: _____ Sheet: 2 of 4
 Project: _____ Job No.: _____
GRANU LOADS Date: 9/26/07
 Engineer: SME

LEVEL 4

- 10" SOLID FILIGREE SLAB

$$\frac{10'' (145 \text{ pcf})}{12''} = \underline{120 \text{ psf}}$$
- 10" TWO-WAY SLAB = 120 psf
- SHEAR WALLS

$$\frac{24'' (18') (60') (145 \text{ pcf}) (4)}{12''} + \frac{18'' (60') (145 \text{ pcf}) (4) (4.8)}{12''}$$

1500 K
- COLUMNUS

$$\frac{(24)(24)}{144} (32) (18 + 4.8) (45) = 423 \text{ K}$$

LEVEL 5 - 39

- 10" VOIDED SLAB

$$\frac{10'' (145 \text{ pcf}) (0.75)}{12''} = 90 \text{ psf}$$
- 10" TWO-WAY SLAB

$$\frac{10'' (145 \text{ pcf})}{12''} = 120 \text{ psf}$$
- SHEAR WALLS

$$\frac{18'' (60') (9.6') (145 \text{ pcf})}{12''} = 125 \text{ K}$$
- COLUMNUS

$$\frac{(24)(24)}{144} (9.6) (32) (45) = 178 \text{ K}$$

Architect: _____ Sheet: 3 of 4
 Project: TECH 1 Job No.: _____
GRAV LOADS Date: 9/26/07
 Engineer: SMR

FLOOR 40

• 12" SOLID SLAB & TWO-WAY SLAB

$$\frac{12''}{12''/ft} (145 \text{ pcf}) = \underline{145 \text{ psf}}$$

• SHEAR WALLS

$$\frac{18''}{12''/ft} (145 \text{ pcf}) (60') (9.6/2 + 14.0) (4) = \underline{980K}$$

• COLUMNS

$$\frac{(24)(24) \text{ in}^2}{144 \text{ in}^2/\text{ft}^2} (145 \text{ pcf}) (9.6/2 + 14.0) (24) = \underline{260K}$$

FLOOR 41

• 10" SOLID SLAB & TWO-WAY SLAB

$$\frac{10''}{12''/ft} (145 \text{ pcf}) = \underline{120 \text{ psf}}$$

• SHEAR WALLS

$$\frac{18''}{12''/ft} (145 \text{ pcf}) (60') (14') (4) = \underline{730K}$$

• COLUMNS

$$\frac{(24)(24)}{144} (145) (14) (24) = \underline{195K}$$

Architect: _____ Sheet: 4 of 4
 Project: _____ Job No.: _____
GRAV LOADS Date: _____
 Engineer: _____

FACADE

- 5" PRECAST PANELS (UP TO LEVEL 4)

ASSUME 150 pcf DENSITY

$$150 \text{ pcf} \left(\frac{5"}{12"} \right) = 62.5 \text{ psf USE } \underline{63 \text{ psf}}$$

- GLASS CURTAIN WALL (4-41)

FROM STRUCTURAL DWGS = 15 psf APPROX

SIGN & FRAMING (TOP LEVEL)

- ASSUMED SIGN WT. = 20 psf

- FRAMING - STEEL

7 BAYS (4 PANELS PER BAY), 170 ft LONG TOTAL

TYPICALLY W16x67

$$67 \text{ plf} (170') (4) (1.50) \quad \leftarrow 50\% \text{ INCREASE FOR COLS \& BAYS}$$

$$\rightarrow \underline{70 \text{ k}}$$

Appendix 5: Roof Snow Load Calculations

ROOF SNOW LOAD:

GROUND SNOW LOAD, P_g : 20 PSF
TERRAIN CATEGORY : C
EXPOSURE OF ROOF : FULLY EXPOSED
SNOW EXPOSURE FACTOR, C_e : 1.0
THERMAL FACTOR, C_t : 1.0
SNOW IMPORTANCE FACTOR, I : 1.1
FLAT ROOF SNOW LOAD, P_f : $0.7 \times C_e \times C_t \times I \times P_g = 11.2$ PSF
 $I \times P_g = 22$ PSF
 $I \times 20$ PSF = 22 PSF

Appendix 6: Wind Load Calculations

Project Trump Taj Mahal - AE 481W
 Engineer Stephen Reichwein
 Date 10/2/2007

Wind Pressure Per ASCE 7-05 MWFRS Procedure 2

Basic Wind Speed	114.00	mph
Importance Factor	1.00	
Occupancy Category	II	
Exposure Category	C	
Directionality Factor (K_d)	0.85	
Gust Factor (G)	0.85	
$C_{p,windward}$	0.80	
$C_{p,leeward}$	0.50	
K_{zt}	1.00	
Z_g	900	ft
α	9.5	

$$K_z = 2.01(z/Z_g)^{2/\alpha}$$

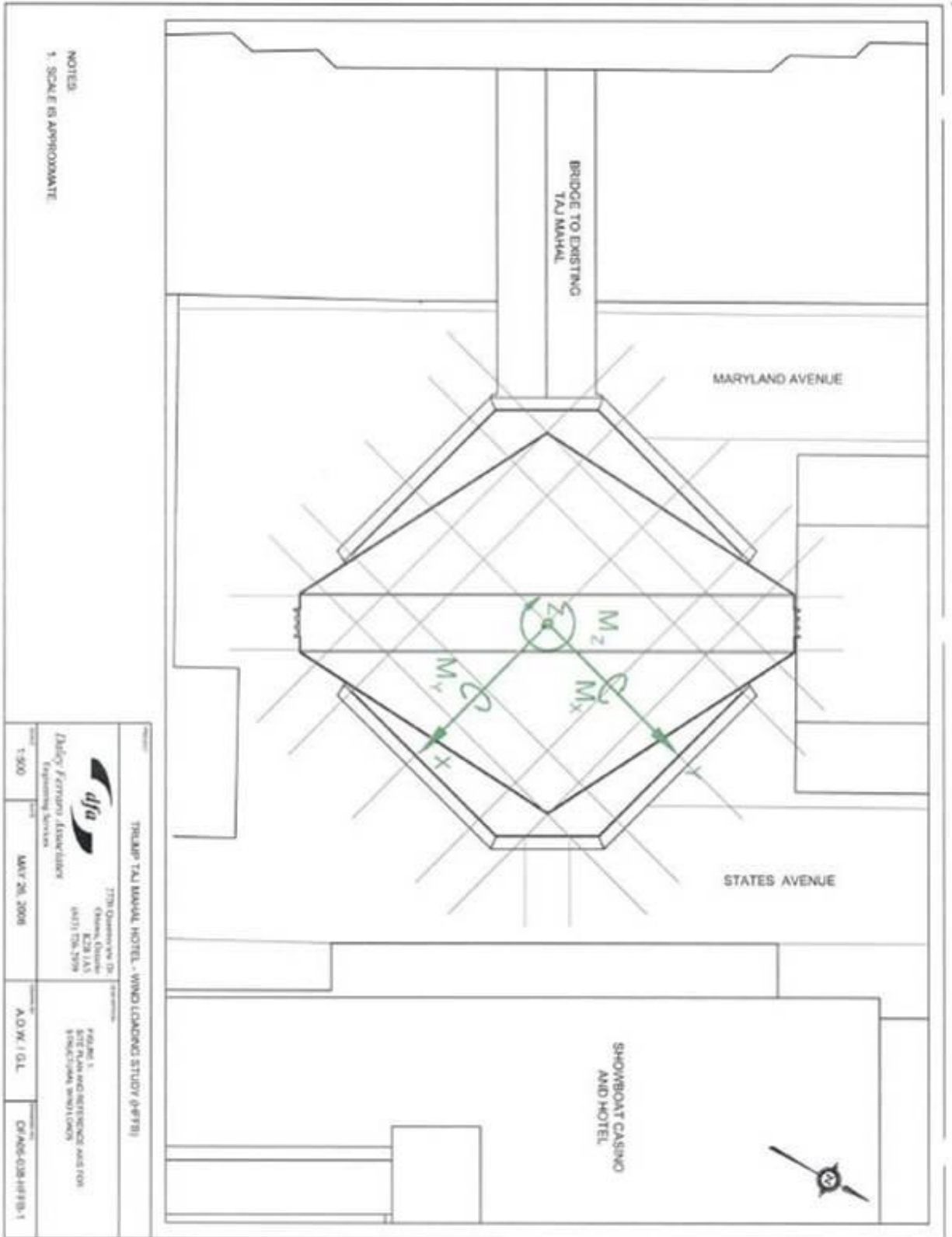
$$P = 0.00256 \times K_d \times G \times V^2 \times I \times (K_z C_{p,w} + K_n C_{p,l})$$

Level	Height (ft)	K_z	K_n	Windward Pressure	Leeward Pressure	Tributary Height (ft)	Perimeter N/S (ft)	Perimeter E/W (ft)	Floor Load N/S (kips)	Floor Load E/W (kips)
1	0.00	0.00	1.75	0	21	0.00	141.25	141.25		
2	16.00	0.86	1.75	17	21	21.00	141.25	141.25	112	112
3	26.00	0.95	1.75	18	21	23.00	141.25	141.25	128	128
4	62.00	1.14	1.75	22	21	22.79	141.25	141.25	139	139
5	71.58	1.18	1.75	23	21	9.58	141.25	141.25	59	59
6	81.17	1.21	1.75	23	21	9.58	141.25	141.25	60	60
7	90.75	1.24	1.75	24	21	9.58	141.25	141.25	61	61
8	100.33	1.27	1.75	24	21	9.58	141.25	141.25	61	61
9	109.92	1.29	1.75	25	21	9.58	141.25	141.25	62	62
10	119.50	1.31	1.75	25	21	9.58	141.25	141.25	63	63
11	129.08	1.34	1.75	26	21	9.58	141.25	141.25	63	63
12	138.67	1.36	1.75	26	21	9.58	141.25	141.25	64	64
13	148.25	1.37	1.75	26	21	9.58	141.25	141.25	64	64
14	157.83	1.39	1.75	27	21	9.58	141.25	141.25	65	65
15	167.42	1.41	1.75	27	21	9.58	141.25	141.25	65	65
16	177.00	1.43	1.75	27	21	9.58	141.25	141.25	66	66
17	186.58	1.44	1.75	28	21	9.58	141.25	141.25	66	66
18	196.17	1.46	1.75	28	21	9.58	141.25	141.25	66	66
19	205.75	1.47	1.75	28	21	9.58	141.25	141.25	67	67
20	215.33	1.49	1.75	29	21	9.58	141.25	141.25	67	67
21	224.92	1.50	1.75	29	21	9.58	141.25	141.25	68	68
22	234.50	1.51	1.75	29	21	9.58	141.25	141.25	68	68
23	244.08	1.53	1.75	29	21	9.58	141.25	141.25	68	68
24	253.67	1.54	1.75	30	21	9.58	141.25	141.25	69	69
25	263.25	1.55	1.75	30	21	9.58	141.25	141.25	69	69
26	272.83	1.56	1.75	30	21	9.58	141.25	141.25	69	69
27	282.42	1.57	1.75	30	21	9.58	141.25	141.25	70	70
28	292.00	1.59	1.75	30	21	9.58	141.25	141.25	70	70
29	301.58	1.60	1.75	31	21	9.58	141.25	141.25	70	70
30	311.17	1.61	1.75	31	21	9.58	141.25	141.25	70	70
31	320.75	1.62	1.75	31	21	9.58	141.25	141.25	71	71
32	330.33	1.63	1.75	31	21	9.58	141.25	141.25	71	71
33	339.92	1.64	1.75	31	21	9.58	141.25	141.25	71	71
34	349.50	1.65	1.75	32	21	9.58	141.25	141.25	71	71
35	359.08	1.66	1.75	32	21	9.58	141.25	141.25	72	72
36	368.67	1.67	1.75	32	21	9.58	141.25	141.25	72	72
37	378.25	1.67	1.75	32	21	9.58	141.25	141.25	72	72
38	387.83	1.68	1.75	32	21	9.58	141.25	141.25	72	72
39	397.42	1.69	1.75	33	21	9.58	141.25	141.25	73	73
40	407.00	1.70	1.75	33	21	18.71	116.25	116.25	117	117
Roof	434.83	1.72	1.75	33	21	13.92	116.25	116.25	88	88
Sign	470.83	1.75	1.75	34	21	36.00	175	25	345	49
								Σ	3283	2987

Project Trump Taj Mahal - AE 481W
 Engineer Stephen Reichwein
 Date 11/5/2007

Wind Loads per Wind Tunnel Test Performed by DFA

Level	Height (ft)	Force N/S, Y Direction (kips)	Force E/W, X Direction (kips)	Distribution to SW 2
1	0.00			
2	16.00	5.40	5.10	2.97
3	26.00	7.70	7.30	4.24
4	62.00	25.00	23.80	13.75
5	71.58	15.60	14.80	8.58
6	81.17	17.60	16.80	9.68
7	90.75	19.70	18.80	10.84
8	100.33	21.70	20.70	11.94
9	109.92	23.80	22.70	13.09
10	119.50	25.90	24.70	14.25
11	129.08	28.00	26.60	15.40
12	138.67	30.10	28.60	16.56
13	148.25	32.20	30.60	17.71
14	157.83	34.30	32.60	18.87
15	167.42	36.30	34.60	19.97
16	177.00	38.40	36.60	21.12
17	186.58	40.50	38.60	22.28
18	196.17	42.60	40.60	23.43
19	205.75	44.60	42.50	24.53
20	215.33	46.70	44.40	25.69
21	224.92	48.80	46.40	26.84
22	234.50	50.90	48.40	28.00
23	244.08	52.90	50.40	29.10
24	253.67	55.00	52.40	30.25
25	263.25	58.30	55.50	32.07
26	272.83	60.50	57.60	33.28
27	282.42	62.60	59.60	34.43
28	292.00	64.70	61.60	35.59
29	301.58	66.90	63.70	36.80
30	311.17	68.90	65.60	37.90
31	320.75	71.00	67.60	39.05
32	330.33	73.20	69.70	40.26
33	339.92	75.30	71.70	41.42
34	349.50	77.40	73.70	42.57
35	359.08	79.60	75.80	43.78
36	368.67	81.70	77.80	44.94
37	378.25	83.80	79.80	46.09
38	387.83	80.60	76.70	44.33
39	397.42	86.30	82.20	47.47
40	407.00	141.70	134.90	77.94
Roof	434.83	116.30	110.70	63.97
Sign	470.83	0.00	0.00	0.00
		2092.50	1992.20	1150.88



Appendix 7: Seismic Load Calculations

Project Trump Taj Mahal - AE 481W
 Engineer Stephen Reichwein
 Date 10/2/2007

Seismic Loads Per ASCE 7-05 Standard

Input

Occupancy Category	I
Importance Factor	1.00
Soil Site Class	D
Seismic Design Category	B
F _a	1.600
F _v	2.400
S _s	0.191
S ₁	0.061
S _{DS}	0.204
S _{D1}	0.0976
R	5.0
Q	2.5
C _d	4.5
T _s	0.319
h _n	434.830
X	0.750
C _t	0.020
T _a	1.904
T _L	6.0
C _s	0.0102
k	1.7
Base Shear (V _b)	1085.8

$$T_a = C_t \times h_n^x$$

$$T \leq T_L \quad \min$$

$$C_s = S_{D1} / (T (R / I)) \quad 0.0102$$

$$C_s = S_{DS} / (R / I) \quad 0.0408$$

$$T > T_L \quad \min$$

$$C_s = S_{D1} \times T_L / (T^2 (R / I))$$

$$C_s = S_{DS} / (R / I)$$

$$C_{smin} = .01$$

kips

Level	Area Non Core (sf)	Area Core (sf)	Tributary Height of Level (ft)	Perimeter (ft)	Façade Wt. (psf)	Self Weight Core (psf)	Self Weight Non Core (psf)	Shear Wall and Column Self Weight (kips)	Super-Imposed DL Core (psf)	Super-Imposed DL Non Core (psf)	Weight of Level (kips)	Elevation Height (feet)	w _x h _x ^k	(w _x h _x ^k)Σ(w _x h _x ^k)xV _b Shear Per Floor (kips)
Sign	N/A	N/A	36.00	400	20	N/A	0	70	0	0	358	470.83	12524409.81	11.18
Roof	13800	N/A	13.92	465	15	N/A	120	925	N/A	30	3092	434.83	94492632.96	84.32
40	14400	3500	18.71	565	15	145	145	1240	30	30	4531	407.00	123739868.4	110.42
39	14400	3500	9.58	565	15	120	90	303	25	15	2404	397.42	63037935.43	56.25
38	14400	3500	9.58	565	15	120	90	303	15	15	2369	387.83	59595054.84	53.18
37	14400	3500	9.58	565	15	120	90	303	15	15	2369	378.25	57113357.31	50.97
36	14400	3500	9.58	565	15	120	90	303	15	15	2369	368.67	54675288.16	48.79
35	14400	3500	9.58	565	15	120	90	303	15	15	2369	359.08	52281184.61	46.66
34	14400	3500	9.58	565	15	120	90	303	15	15	2369	349.50	49931395.49	44.56
33	14400	3500	9.58	565	15	120	90	303	15	15	2369	339.92	47626281.94	42.50
32	14400	3500	9.58	565	15	120	90	303	15	15	2369	330.33	45366218.21	40.48
31	14400	3500	9.58	565	15	120	90	303	15	15	2369	320.75	43151592.55	38.51
30	14400	3500	9.58	565	15	120	90	303	15	15	2369	311.17	40982808.12	36.57
29	14400	3500	9.58	565	15	120	90	303	15	15	2369	301.58	38860284.05	34.68
28	14400	3500	9.58	565	15	120	90	303	15	15	2369	292.00	36784456.64	32.83
27	14400	3500	9.58	565	15	120	90	303	15	15	2369	282.42	34755780.56	31.02
26	14400	3500	9.58	565	15	120	90	303	15	15	2369	272.83	32774730.34	29.25
25	14400	3500	9.58	565	15	120	90	303	15	15	2369	263.25	30841801.92	27.52
24	14400	3500	9.58	565	15	120	90	303	15	15	2369	253.67	28957514.42	25.84
23	14400	3500	9.58	565	15	120	90	303	15	15	2369	244.08	27122412.16	24.20
22	14400	3500	9.58	565	15	120	90	303	15	15	2369	234.50	25337066.88	22.61
21	14400	3500	9.58	565	15	120	90	303	15	15	2369	224.92	23602080.28	21.06
20	14400	3500	9.58	565	15	120	90	303	15	15	2369	215.33	21918086.94	19.56
19	14400	3500	9.58	565	15	120	90	303	15	15	2369	205.75	20285757.63	18.10
18	14400	3500	9.58	565	15	120	90	303	15	15	2369	196.17	18705803.12	16.69
17	14400	3500	9.58	565	15	120	90	303	15	15	2369	186.58	17178978.6	15.33
16	14400	3500	9.58	565	15	120	90	303	15	15	2369	177.00	15706088.87	14.02
15	14400	3500	9.58	565	15	120	90	303	15	15	2369	167.42	14287994.42	12.75
14	14400	3500	9.58	565	15	120	90	303	15	15	2369	157.83	12925618.59	11.53
13	14400	3500	9.58	565	15	120	90	303	15	15	2369	148.25	11619956.21	10.37
12	14400	3500	9.58	565	15	120	90	303	15	15	2369	138.67	10372084.04	9.26
11	14400	3500	9.58	565	15	120	90	303	15	15	2369	129.08	9183173.502	8.19
10	14400	3500	9.58	565	15	120	90	303	15	15	2369	119.50	8054506.508	7.19
9	14400	3500	9.58	565	15	120	90	303	15	15	2369	109.92	6987495.351	6.24
8	14400	3500	9.58	565	15	120	90	303	15	15	2369	100.33	5983708.208	5.34
7	14400	3500	9.58	565	15	120	90	303	15	15	2369	90.75	5044902.475	4.50
6	14400	3500	9.58	565	15	120	90	303	15	15	2369	81.17	4173069.39	3.72
5	14400	3500	9.58	565	15	120	90	303	15	15	2369	71.58	3370495.55	3.01
4	14400	3500	22.79	565	25.11	120	120	1923	30	30	4931	62.00	5495872.279	4.90
3	14400	3500	23.00	565	63	170	110	2027	30	30	5562	26.00	1414689.335	1.26
2	14400	3500	21.00	565	63	145	110	1145	30	30	4521	16.00	503776.6072	0.45
Σ												105935	1216734482	1085.83

Appendix 8: Shear Wall Spot Check Calculations

1

SHEAR WALL CHECK

ASSUMPTIONS:

- FORCES WILL BE DISTRIBUTED BASED ON WALL AREA ACCOUNTING FOR LARGE OPENINGS ONLY.
- LIVE & DEAD LOADS WILL BE NEGLECTED
- LOADS FROM THIS WIND TUNNEL TEST WILL BE USED IN PLACE OF LOADS CALCULATED PER ASCE - 705
- CONTROLLING LOADS (NORTH / SOUTH)

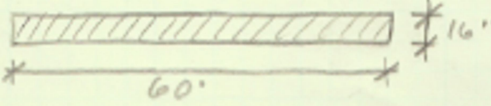
SEISMIC BASE SHEAR = 1086 K

WIND BASE SHEAR = 2092 K ← GOVERNS
↳ WIND TUNNEL TEST

SHEAR WALL 2 WILL BE CHECKED AT LEVEL 24

PROPERTIES:

h = 16"
L = 60'
f_c = 5000 psi
f_y = 60 ksi



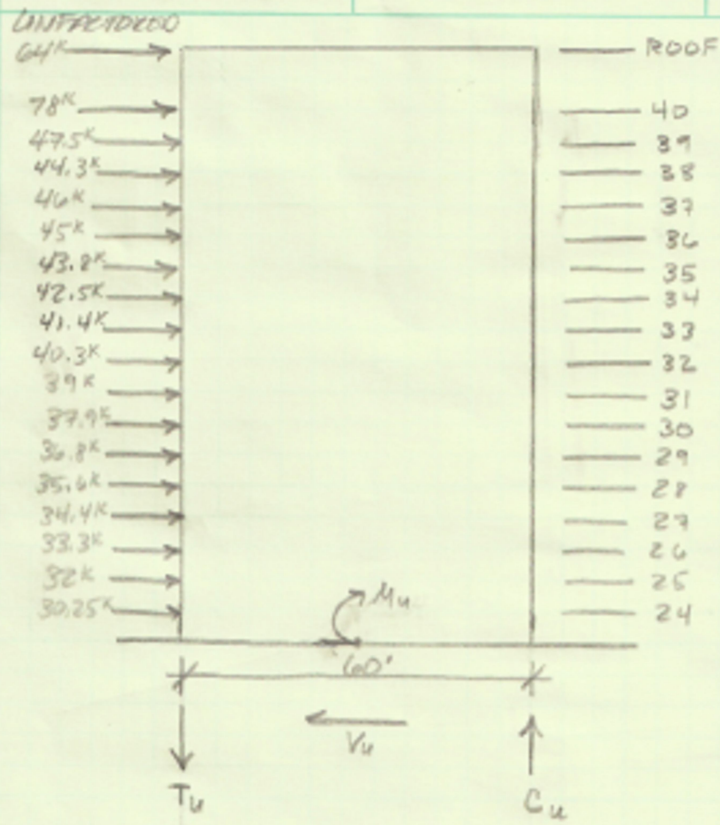
• GROSS AREA SHEAR WALL 1 - AREA OF OPENINGS:
 $A_g - A_o = A_1 = 26231 - 360 - 80(18) - 18(150) \cdot 100$
 $A_1 = 21631 \text{ ft}^2$

• GROSS AREA SHEAR WALL 2 - AREA OF OPENINGS:
 $A_g - A_o = A_2 = 26231 - 140 = 26091$

• DISTRIBUTION TO SHEAR WALL 2:

$$\frac{A_2}{A_2 + A_1} = \frac{26091}{21631 + 26091} = 0.55 = \underline{55\% \text{ SW 2}}$$

= 45% SW 1



$$V_u = 1.6(772k) = 1235k$$

$$M_u = 78475(1.6) = 125560k'$$

$$T_u = C_u = \frac{M_u}{e} = \frac{125560k'}{60'} = 2093k$$

FACTORED LOAD COMBINATION USED :

$$W_u = 1.6W \rightarrow \text{COLLECTED LIVE \& DEAD LOADS}$$

Shear Wall Overturning Moment

Level	Moment Arm (ft)	Force (k)	Ovt Moment (k-ft)
41	187.91	217	40777.19
40	160.08	59	9444.92
39	150.67	37	5574.67
38	141.25	36	5085.00
37	131.83	36	4746.00
36	122.42	36	4407.00
35	113.00	36	4068.00
34	103.58	36	3729.00
33	94.17	36	3390.00
32	84.75	36	3051.00
31	75.33	36	2712.00
30	65.92	35	2307.08
29	56.50	35	1977.50
28	47.08	35	1647.92
27	37.67	35	1318.33
26	28.25	35	988.75
25	18.83	35	659.17
24	9.42	35	329.58
			96213.11

ft-kips

A) DETERMINE LONGITUDINAL & TRANSVERSE REINF

IF $V_u \geq 2 A_{cv} \sqrt{f_c} \rightarrow 2$ CURTAINS REQ'D

$$\frac{2(60)(12)(16) \sqrt{5000 \text{ psi}}}{1000} = 1629 \text{ K} > 1235 \text{ K}$$

\therefore ONLY ONE CURTAIN REQ'D

B) FIND p_s & p_t REQUIRED

$$p_s = p_t = 0.0025 \text{ PER ACI 318-05-14.3}$$

$$A_{cv} = (16") (12") = 192 \text{ in}^2/\text{ft}$$

$$A_{s, \text{long}} = 0.0025 (192 \text{ in}^2/\text{ft}) = 0.48 \text{ in}^2/\text{ft}$$

TRY #7 BARS $A = 0.60 \text{ in}^2$

$$\frac{0.48 \text{ in}^2}{12 \text{ in}} = \frac{0.60 \text{ in}^2}{s} \quad s = 15" < s_{\text{max}} = 18"$$

• USE #7 BARS AT 15" O.C. VERTICAL & HORIZONTAL

ACTUAL DESIGN CALLS FOR #6 BARS AT 18" O.C. VERTICAL & HORIZONTAL EACH FACE.

• THIS COULD BE A RESULT OF DEFLECTION CONTROL.

C) NOMINAL SHEAR CAPACITY

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

$$\frac{h_w}{l_w} = \frac{16(16')}{60'} = 3.02 \geq 2 \quad \therefore \alpha_c = 2$$

$$A_{cv} = 16''(60')(12''/ft) = 11520 \text{ in}^2$$

$$\rho_t = \frac{0.60(12/15)}{16(12)} = 0.0025$$

$$V_n = \frac{11520 \text{ in}^2 (2(\sqrt{5000}) + 0.0025(60000))}{1000 \text{ lb/kip}}$$

$$V_n = 3357 \text{ K}$$

$$\phi V_n = 0.75(3357 \text{ K}) = 2517.75 \text{ K} > 1235 \text{ K}$$

OK

D) COMPRESSION BOUNDARY ELEMENT

$$\sigma \geq 0.2 f'_c \quad \text{B.E. REQ'D}$$

$$\sigma = \frac{M_u h/2}{I_g} = \frac{125560 \text{ K}(30')(144 \text{ in}^2/\text{ft}^2)}{497 \times 10^6 \text{ in}^4}$$

$$I_g = \frac{(60(12) \text{ in})^3(16) \text{ in}}{12} = 497 \times 10^6 \text{ in}^4$$

$$\sigma = 1.09 \text{ ksi}$$

$$0.2 f'_c = 0.2(5 \text{ ksi}) = 1.0 \leq 1.09 \text{ ksi}$$

\therefore B.E. REQ'D

ADDITIONAL STEEL WILL BE PLACED WITHIN WALL AT THE ENDS

D (CONTINUED)

GRAVITY LOADS WILL BE NEGLECTED

$$C_u = 2093K$$

$$\phi P_n = 0.65 (0.80) [0.85 f_c (A_g - A_{st}) + f_y A_{st}]$$

DEVOTE 60" OF ENDS TO B.E.

$$A_g = 48" (16") = 768 \text{ in}^2$$

$$2093K = 0.52 [0.85 (5 \text{ ksi}) (768 - A_{st}) + 60 \text{ ksi } A_{st}]$$

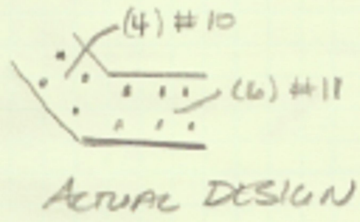
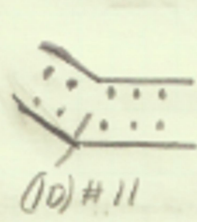
$$2093K = 2.21 (768 - A_{st}) + 31.2 A_{st}$$

$$2093K = 1697 - 2.21 A_{st} + 31.2 A_{st}$$

$$396 = 29 A_{st} \quad \therefore A_{st} = 13.7 \text{ in}^2$$

USE #11s $A = 1.56 \text{ in}^2$

$$n = \frac{13.7 \text{ in}^2}{1.56 \text{ in}^2} = 8.8 \approx 10 \text{ BARS (5) @ EACH FACE}$$



CONCLUSION: VERY SIMILAR

E) TENSILE STRENGTH

$$A_{ST} = (0.60 \text{ in}^2 / 15'') (30') (12'') + 10 (1.56)$$

$$A_{ST} = 30 \text{ in}^2$$

$$\phi T_n \geq T_u = 2093 \text{ K} \leq 0.90 (30 \text{ in}^2) (60) = 1620 \text{ K}$$

∴ ADD'L STEEL REQ'D

$$2093 \text{ K} \leq 0.90 A_{ST} 60$$

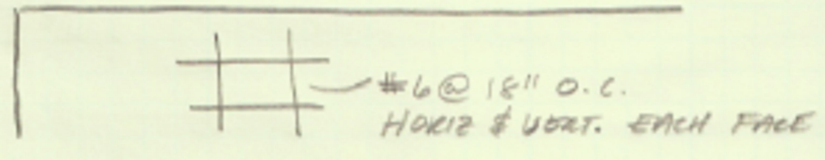
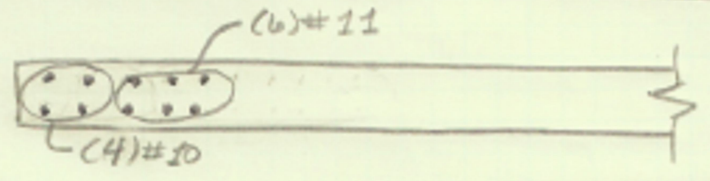
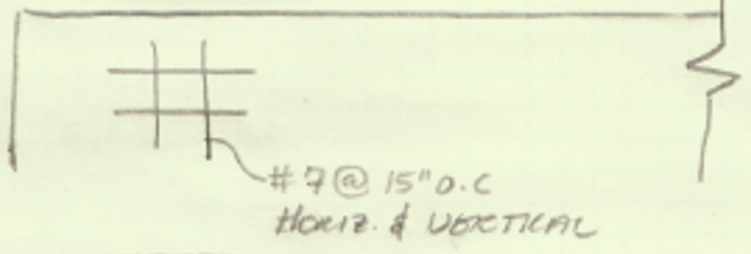
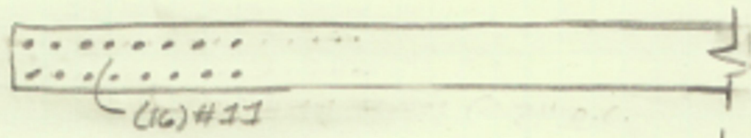
$$A_{ST} = 38.8 \text{ in}^2$$

$$A_{ADDL} = 38.8 - 30 = 8.8 \text{ in}^2$$

$$\frac{8.8}{1.56} = 5.6 \approx 6 \text{ BARS}$$

ADD AN ADD'L (3) #11 @ EACH FACE

7



ACTUAL DESIGN

CONCLUSIONS:

DESIGNS ARE QUITE DIFFERENT. I FOUND THAT 1 CURTAIN OF REINF WOULD SUFFICE, BUT THE ACTUAL DESIGN USES 2. COMPRESSION STEEL WAS ALMOST IDENTICAL TO THAT FOUND IN THE ACTUAL DESIGN, HOWEVER TENSION CONTROLLED & REQUIRED ADD'L REINF. THE ACTUAL DESIGN DIDN'T REQUIRE ANY ADD'L TENSION REINF BECAUSE OF THE 2 CURTAINS.

Appendix 9: Filigree Slab Spot Check Calculations

1

FILIGREE SLAB DESIGN (10" VOIDED SLAB TYPICAL FLOOR)

3/8" ϕ 7 STRAND 270 ksi TENDONS

$A_s = 0.085 \text{ in}^2$
 $f_y = 180,000 \text{ psi}$

ASSUME IN-SLAB BEAM PROVISIONS NO TORSIONAL RESTRAINT

DL = 15 psf
 LL = 40 psf
 SW = 90 psf

$W_u = 1.2(90 + 15) + 1.6(40) = 0.20 \text{ k/ft} / \text{ft}$

SECTION PROPERTIES

10"

2 1/4"

2 1/4"

MOMENTS (RAM ADVANCE)

DETERMINE PRESTRESSING REQUIREMENTS
 ↳ POSITIVE MOMENT RESISTANCE

$$M_u = 14.41 \text{ k-ft/ft}$$

$$f_y = 180,000 \text{ psi}$$

$$d = 10" - \frac{2.25"}{2} = 8.9"$$

$$\phi M_n = M_u = 0.9 A_s f_y \left(d - \frac{A_s f_y}{0.85 f'_c b} \right)$$

← 8'-0"

PLANKS ARE 8'-0" WIDE

$$M_u = 14.41(8) \text{ k-ft} = \frac{0.9(180)}{12} A_s (8.9) = \frac{180(A_s)}{0.85(5)(8)(12)}$$

$$8.54 = 8.125 A_s - 0.44 A_s^2$$

$$0.44 A_s^2 - 8.125 A_s + 8.54 = 0$$

$$A_s = \frac{8.125 \pm \sqrt{8.125^2 - 4(0.44)(8.54)}}{2(0.44)} = 1.12 \text{ in}^2$$

$$\# \text{ TENDONS} : \frac{1.12 \text{ in}^2}{0.085 \text{ in}^2/\text{TENDON}} = 13.18 \text{ USE } \underline{14}$$

COMPARE 14 CABLES TO 14 PROVIDED

OK

DETERMINE NEGATIVE REINF. REQ'D

$$M_u = 25.5 \text{ ft-kips/ft}$$

$$d = 8.9''$$

$$\frac{M_u}{4d} = \frac{25.5}{4(8.9)} = 0.716 \text{ in}^2/\text{ft}$$

TRY #6 BARS $A_s = 0.44 \text{ in}^2$

$$\frac{0.44}{5} = \frac{0.716}{12} \quad s = 7'' \text{ O.C.}$$

$$a = \frac{0.44(12/7)(60)}{0.85(5)(12)} = 0.89''$$

$$\phi M_n = \frac{0.90(0.44)(12/7)(60)(8.9 - 0.89/2)}{12} = 28.7 \text{ k-ft}$$

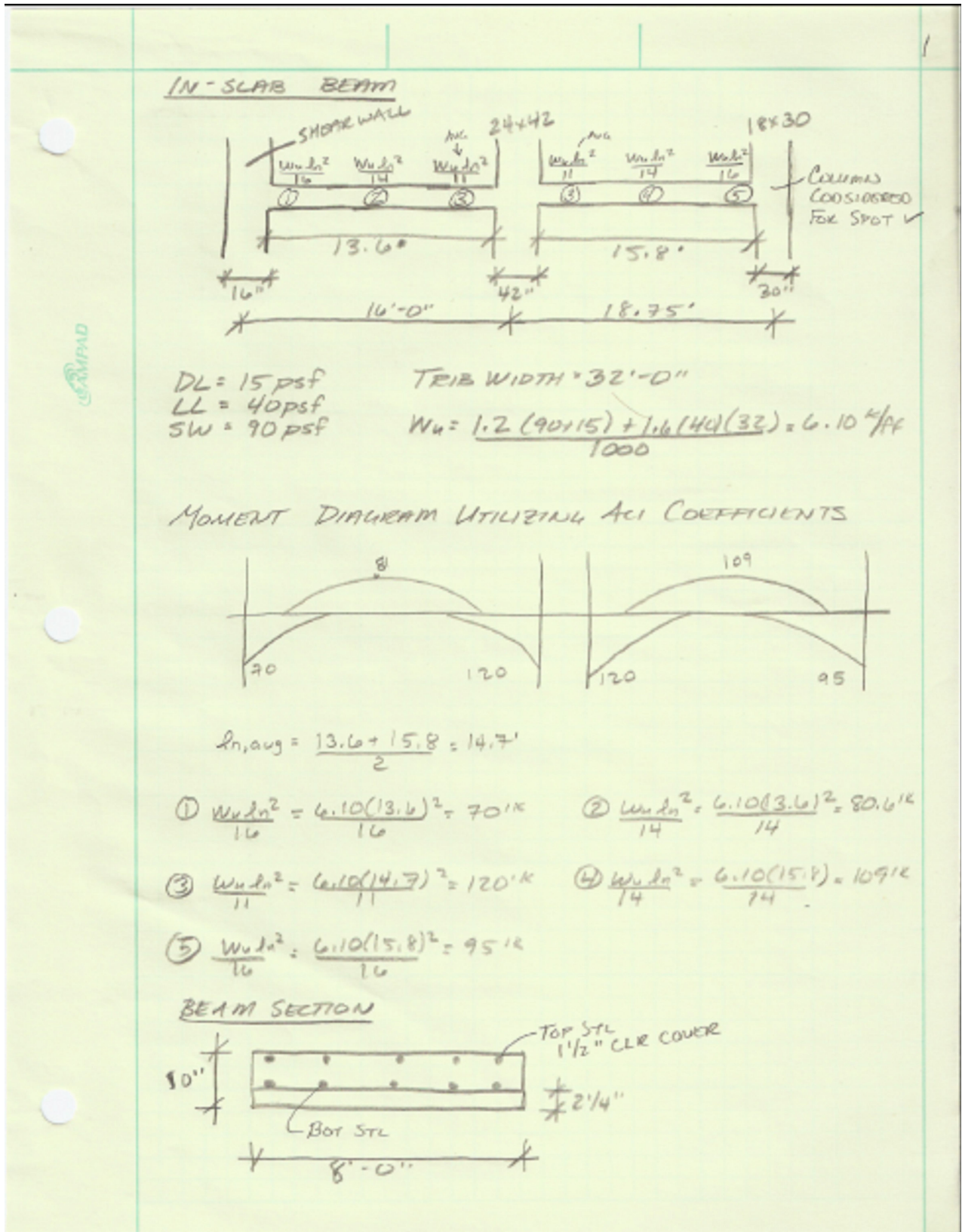
$$\phi M_n \geq M_u \quad \text{OK}$$

USE #6 @ 7" OC

#6 @ 9" PROVIDED FOR STRUCTURAL DUALS

SUPPORTS AT BOTH ENDS WERE ASSUMED TO PROVIDE NO TORSIONAL RESTRAINT. HOWEVER, IN REALITY THEY WILL PROVIDE SOME, DECREASING THE DESIGN MOMENTS IN THE SPAN.

Appendix 10: In-Slab Beam Spot Check Calculations



ASSUME #6 BARS $A = 0.44 \text{ in}^2/\text{BAR}$

$$d_{TOP} = 10'' - 1\frac{1}{2}'' - \frac{3}{8}'' = 8.125''$$

$$d_{BOT} = 10'' - 2.25'' = 7.75''$$

NEGATIVE STEEL:

$$M_u = 120 \text{ k}$$

$$\frac{M_u}{4d} = \frac{120}{4(8.125)} = 3.69 \text{ in}^2 \quad \frac{3.69}{0.44} = 8.37 \approx 9 \text{ BARS}$$

$$a = \frac{9(0.44)(60)}{0.85(5)(9)(12)} = 0.58''$$

$$\phi M_n = \frac{0.9(9)(0.44)(60)(8.125 - 0.58/2)}{12} = 139.6 \text{ k}$$

$$c = \frac{0.58}{0.80} = 0.72'' \quad \epsilon_t = \frac{0.003(8.125 - 0.72)}{0.72} \gg 0.005$$

$\therefore \phi = 0.90$

USE (9) #6 BARS \rightarrow MATCHES STRUCTURAL DWGS

POSITIVE STEEL:

$$M_u = 109 \text{ k} \quad M_u/4d = 109/4(7.75) = 3.52 \text{ in}^2$$

$$\frac{3.52 \text{ in}^2}{0.44 \text{ in}^2/\text{BAR}} = 8 \text{ BARS}$$

$$a = \frac{8(0.44)(60)}{0.85(5)(12)(8)} = 0.518'' \quad c = \frac{0.518}{0.80} = 0.647''$$

$$\phi M_n = \frac{0.90(8)(0.44)(60)(7.75 - 0.518/2)}{12} = 118.7 \text{ k}$$

$$\epsilon_t = \frac{0.003(7.75 - 0.647)}{0.647} \gg 0.005 \quad \therefore \phi = 0.90$$

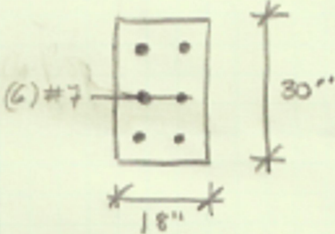
USE (8) #6

(7) #6 PROVIDED BY STRUCTURAL

\therefore ACCEPTABLE STAT \checkmark

Appendix 11: Column Spot Check Calculations

COLUMN CHECK
 COLUMN AT LEVEL 38 ; G-4 INTERSECTION



$f'_c = 5000 \text{ psi}$
 $f_y = 60000 \text{ psi}$

$A_{TRIB} = \frac{32' (18.75)'}{2} \approx 300 \text{ ft}^2$

LOADING

<u>LEVEL</u>	<u>DL (psf)</u>	<u>LL (psf)</u>
41	120 <u>22</u> 142	20
40	145 <u>22</u> 167	20
39	90 <u>25</u> 115	40
38	90 <u>15</u> 105	40

SELF WEIGHT = $\frac{18' (30")}{144 \text{ in}^2/\text{ft}^2} \frac{(47') (145 \text{ pcf})}{1000 \text{ lb/kip}} = 25.6 \text{ K}$

$P_u = [(142 + 167 + 115 + 105)(300) + 25.6](1.2)$
 $+ [20(20) + 40(2)](300)(1.6)$
 $P_u = 248 \text{ K}$

$M_u = 95 \text{ K}$ (FROM ACI COEFFICIENTS)
 SEE IN-SCAB BM CHECK

