# Hilton Hotel at BWI Airport

Linthicum Heights, MD

# Technical Report 1 Existing Conditions

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Structural
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## 1.1 EXECUTIVE SUMMARY

The Hilton Hotel at the BWI Airport is located in Linthicum Heights, MD. The size of the hotel is 203,300 s.f. and elevates from the ground eleven stories plus a mechanical penthouse. This 280 guestroom hotel houses a ballroom/ assembly room, pool with an indoor/ outdoor sundeck, restaurant, and an 80-car parking garage below grade. The cost of construction was roughly $35 million which began April 25, 2005 and was substantially completed on September 21, 2006.

Technical Report 1 is an existing conditions report. Structural systems of the building are described within this report. The main structural system of the building is cast-in-place reinforced concrete. Typical hotel room floors are two-way post tensioned concrete slabs. Loading conditions and calculations are given in tables throughout the report. Gravity loads, both Live and Dead were computed for a given area where spot checks were calculated. Spot checks on one column in two different levels were checked for nominal strengths. Columns were more than sufficient to carry pure axial loads. Punching shear checks were also evaluated in two locations, the 6$^{th}$ floor and the ground floor. The 6$^{th}$ floor location is a flat two-way post tension slab, while the ground floor is a two-way mild reinforced slab with drop panels. Both locations were sufficient to carry the shear forces due to the factored gravity loads.

Lateral load calculations were computed for the building. Seismic results produced a larger base shear than that calculated by the Engineer of record. Wind loading results were within 10% of the results of the Engineer of Record's calculations. A shear wall check for direct shear produced by lateral loads was calculated on two shear walls. The results yielded that the design strength surpassed the direct shear force on each wall.

## 2.1 STRUCTURAL SYSTEM DESCRIPTION

This section gives a summary of the structural system as well as a description of the system floor by floor, the foundation and the lateral load resistance system.

## 2.2 STRUCTURAL SUMMARY

The structural system of the Hilton Hotel at the BWI Airport varies throughout the building. The primary structural system of the building is cast-in-place reinforced concrete. The typical floor system (floors 4-11) is a two-way post tensioned concrete slab. Floors ground through three are a two-way mild reinforced concrete slab with drop panels. The penthouse roof deck is a two-way post-tensioned reinforced concrete slab with concrete beams. The columns transfer load to he foundation which are typical spread footings. The lateral resistance system is a series of shear walls that extend up from the foundation through the building. These shear walls transfer load to a mat foundations.

The double-heighted ballroom, adjacent assembly room, and main entrance spaces are all enclosed by a structural steel system. This system consists of various shape beams, girders, and joists with a corrugated metal deck roof.

## 2.3 FOUNDATION AND PARKING LEVEL

Columns transfer gravity load to the foundation. The foundation consists of reinforced concrete spread footings ranging in size of (3’-0” by 3’0” by 12”) to (10’-0” by 10’-0” by 40”). Lateral loads carried by the series of 12 shear walls transfer load to reinforced concrete mat foundations. Concrete used in footing is specified to reach a 28-day f’c = 3000 psi.

The floor system for the Parking Level is a 5” Slab-On-Grade (SOG) reinforced with 6x6 w2.0 x w2.0 WWF. A concrete strength of f’c = 3500 psi was specified for the SOG.

## 2.4 GROUND LEVEL FRAMING PLAN

The ground level floor system consists of a two-way mild reinforced 9" thick concrete slab with typical 9'x9'x4" drop panels around the columns. The bottom reinforcement in the concrete slab consists of #5 bars at 12" o.c. each way. The top of slab reinforcement varies in reinforcing bars.

Concrete columns are typically spaced 27'-0" o.c. Columns vary in size:

| | |
|---|---|
| 14x14 | 12x12 |
| 14x26 | 18x18 |
| 14x76 | 18x26 |
| 16x16 | 26x14 |
| 16x28 | |

Concrete columns are specified to reach a 28-day f'c = 5600 psi.

## 2.5 SECOND FLOOR FRAMING PLAN

The second floor is similar to the ground floor framing. The floor system is a two way mild reinforced concrete slab with typical 9'x9'x4" drop panels around the columns. Slab thickness is 9". Bottom reinforcement in the concrete slab consists of #5 bars at 12" o.c. each way. Top of slab reinforcement varies in reinforcing bars.

Column sizes are 14x26 and 16x28. Columns are also specified to reach a 28-day f'c = 5600 psi. There is a Transfer Girder near the elevator shear walls. This girder transfers load from columns on level two to columns on the ground level which are offset from the columns on level two.

The double-heighted ballroom, adjacent assembly room, and main entrance spaces are all enclosed by a structural steel system. This area also contains a pool but the pool structure is composed of epoxy-coated reinforced concrete transferring load to W-shape steel columns embedded in concrete. Adjacent to the pool area is a reinforced concrete roof deck framed with reinforced concrete beams. This roof deck runs flush with the metal roof deck over the assembly room. See Appendix B Drawings.

The adjacent assembly room roof is framed out with 20LH4 shape steel members with a 3” deep 18 gage galvanized metal deck. Load transferred from these shapes to typical W16x40 beams transfer load to W-shape columns.
The main entrance roof is framed out with 3” deep 18 gage galvanized metal deck which transfers load to LH-shape steel joists which then transfers the load to W-shape beams to W-shape steel columns.

**2.6 THIRD FLOOR FRAMING PLAN**

The third floor is a two-way mild reinforced concrete slab with with typical 9’x9’x4” drop panels. Slab thickness is 9”. Bottom reinforcement in the concrete slab consists of #5 bars at 12” o.c. each way. Top of slab reinforcement varies in reinforcing bars.

The ballroom area on this floor level has a system of 3-W8x31 Girders spanning East-West in two locations to partially support the roof and also support folding partitions that are hung using MC-shape steel from the 3-W8x31 system. Joists carry the remaining load of the roof over the ballroom. Roughly 75% of the metal deck area over the ballroom transfers loads to typical 52DLH13 joists 9’-0” o.c. The rest of the metal deck transfers loads to 52DLH16 joists 4’-6” o.c. Joists transfer loads to W16x50 to W-shape steel columns on the south-west side. On the opposite end joists transfer loads to reinforced concrete beams integrated into the adjacent floor slab.

The roof decks are 3” deep 18 gage galvanized metal deck. The load over the pool area roof transfers to typical W27x94 beams which transfer the load to W-shape steel columns.

**2.7 FOURTH THROUGH ELEVENTH FLOOR FRAMING PLAN**

Floors four through eleven are typical framing plans. These are the hotel room floors. The floor system is a two-way post-tensioned reinforced concrete slab. The floor thickness is 7-1/2” with a specified f’c = 4000 psi. The bottom reinforcing consists of #4 bars 30” o.c. in both directions. Tendons spanning North – South are tensioned at 24 k/ft. Interior tendons spanning East – West are tensioned at $295^k$. End tendons spanning

East – West are tensioned at $135^k$ on the North side while at $215^k$ on the South side. Two pour strips each 4'-0" wide were left open to tension tendons. Columns sizes are 14"x26" and 16"x28" with a specified f'c = 4000psi.
See Appendix B Drawings for typical framing plan.

**2.8 PENTHOUSE FLOOR AND ROOF PLAN**

The penthouse floor system is similar to the typical floors of four through eleven. The floor system is a two-way post-tensioned reinforced concrete slab. Floor slab thickness is 9" with a specified f'c = 4000 psi. Bottom reinforcing consists of #5 bars 24" o.c. in both directions. Two pour strips each 4'-0" wide were left open to tension tendons. Columns sizes are 14"x26", 14"x14"and 16"x28" with a specified f'c = 4000 psi.

**2.9 PENTHOUSE ROOF FRAMING PLAN**

The penthouse roof framing is two-way post-tensioned reinforced concrete slab with typical 18" drop beams. The roof slab thickness is 7-1/2" with a specified f'c = 4000 psi. The bottom reinforcing consists of #4 bars 30" o.c. in both directions.

**2.10 LATERAL LOAD RESISTANCE SYSTEM**

The lateral load resistance system is a series of twelve reinforced concrete shear walls, eleven of which are located in three locations: around two stairwells near either edge of the north and south sides, and an elevator core in the center of the building. The twelfth shear wall is located on the North side of the building and only spans vertically from foundation to the second floor. Three shear walls are located around each stairwell and five are located around the elevator core. Shear walls are 1'-0" thick specified f'c = 4000 psi. Shear walls transfer load to mat foundations previously mentioned.

## 3.1 LOADS

This section includes all required loading conditions including gravity, wind, and seismic as well as spot checks of the original design. Codes and references used in this section were ASCE7-05, *Design of Concrete Structures* 13th Edition. The concrete text book references the ACI code.

All the codes and references that the Engineer of Record used for the project may be found Appendix A.

## 3.2 GRAVITY LOADS

Dead loads used in calculations for spot checks were a combination of structure self weight and superimposed dead loads used by the Engineer of Record. See Table 3.1

| Area | PSF |
|---|---|
| Roofs | 30 |
| Penthouse Roof | 40 |
| Penthouse Floor | 20 |
| Guestroom Floors | 10 |
| Second Floor | 10 |
| First Floor | 10 |
| Pool Deck | 40 |

**Table 3.1 Engineer of Record Superimposed Dead Loads**

The Engineer of Record references the Live Loads from ASCE7-02. Table 3.3 shows the loads used for the given areas.

| Area | PSF | | Area | PSF |
|---|---|---|---|---|
| Roof Live Load | 30 | | Garage Level | 150 |
| Penthouse Floor | 150 | | Pool Deck | 100 |
| Guestroom Floors | 40 | | First Floor | 100 |
| Second Floor | 100 | | First and second Floor Storage Kitchen and Laundry | 125 |
| Second Floor Mechanical Rooms | 150 | | Meeting Rooms | 100 |
| Meeting Rooms | 100 | | Stairs | 100 |
| Stairs | 100 | | Garage Level | 150 |

**Table 3.2 Engineer of Record Live Loads**

Live loads used for spot check calculations come from ASCE7-05 can be found in Table 3.3. Live loads used in gravity load spot checks were chosen from ASCE7-05 to try to match the loads used by the engineer. An occupancy type for mechanical rooms could not be found, so therefore the load was taken from the Engineer of Record.

| Live Loads (psf) | ASCE7-05 |
|---|---|
| Dance halls and ballrooms | 100 |
| Dining rooms & restaurants | 100 |
| Private rooms & corridors serving them | 40 |
| Public rooms & corridors serving them | 100 |
| Storage warehouses – Light | 125 |

**Table 3.3 ASCE7-05 Live Loads**

Live load reductions factors were taken from ASCE7-05 and used in spot check calculations.

### 3.2.1 Snow Loads

Snow loads were calculated using Chapter 7 of ASCE-05. The Engineer of Record references ASCE7-02. Both years of the code yields the same roof snow load. Table 2.3 shows the factors used in the roof snow load calculation. The flat roof snow load was calculated only. Unbalance drifting and sliding snow where not taken into consideration for this technical report.

| Factors | Engineer of Record ASCE7-02 | Experimental Data ASCE7-05 |
|---|---|---|
| Ground Snow Load($P_g$) | 25 PSF | 25 PSF |
| Snow Exposure ($C_e$) | 0.9 | 0.9 |
| Importance Factor ($I_s$) | 1.0 | 1.0 |
| Roof Thermal ($C_t$) | 1.0 | 1.0 |
| Flat Roof Snow Load | 16 PSF | 16PSF |

**Table 3.4 Snow Load**

### 3.2.2 Gravity Load Spot Checks

Gravity load spot checks were evaluated to compare experimental data and results of the Engineer of Record's original design. Columns were checked in two locations for strength. Punching shear was checked in two locations: at a drop panel on the ground floor and on a flat slab on the 6$^{th}$ floor. Punching shear was checked instead of flexure of the floor systems due to shear most likely being the controlling factor. This assumption was given by a consultant.

#### 3.2.2.1 Column Spot Checks

Column spots checks were performed on one column (F-5.2), in two locations (6$^{th}$ floor and Ground floor). Calculated column factored loads per floor compared to the Engineer of Record's unfactored loads are shown in Table 3.5. The load combination shown controlled over all other applicable ASCE7-05 “Basic Combinations” found in section 2.3.2 of the code.

| Experimental Data Gravity Loads (k) | Engineer of Record Gravity Loads (k) |
|---|---|
| $1.2D + 1.6L + 0.5L_r$ | Unfactored |
| 58.30 | 60 |
| 374.09 | 208 |
| 462.88 | 279 |
| 551.64 | 350 |
| 640.39 | 420 |
| 729.14 | 490 |
| 817.90 | 560 |
| 906.65 | 631 |
| 995.40 | 701 |
| 1084.15 | 771 |
| 1185.31 | 841 |
| 1321.56 | 968 |
| 1457.82 | 1088 |

**Table 3.5 Load on Column F-5.2**

Columns were checked for nominal strength in pure axial and combined loading. The combined loading strength calculations were performed using the Design Aid A.6 in the *Design of Concrete Structures* textbook.

Two eccentricities were used in determining combined loading nominal strength. An eccentricity which would produce the largest moment and an eccentricity near the bending axis were used. The combined loading using the largest eccentricity, for columns in both locations, produced insufficient strength to carry the factored gravity loads calculated for those columns. For the eccentricity taken at 2” from the bending axis, the column on the $6^{th}$ floor had sufficient strength while the column on the ground floor was not adequate by about 50 kips. See calculations in the Appendix C “Column Strength Check”. Further investigation on the moments on the column induced by the floor slab will come in later technical reports. This check was just preliminary since moments were not calculated on columns but were assumed by eccentricities.

Nominal pure axial strengths of the columns were sufficient to carry the factored gravity loads on the column computed in the experimental data. See calculations in the Appendix C “Column Strength Check”.

#### 3.2.2.2 Punching Shear Spot Checks

Punching shear spot checks were taken from two locations around the column F-5.2. The first location on the $6^{th}$ floor, is a 7-1/2” two-way post tension slab. The second location was on the ground floor where the floor is a 9” thick two-way mild reinforced slab with a 9’x9’x4” drop panel.

Nominal shear capacities of the concrete in both locations were adequate enough to carry the punching shear force produced by the gravity loads. See calculations in Appendix C “Punch Shear Slab Check”.

## 3.3 LATERAL LOADS

Lateral Loads were computed using excel spread sheets. Electronic copies of the spread sheets can be obtained upon request. Procedures and equations for wind and seismic loads are referenced to ASCE7-05 Chapters 6, 11, 12 and 19.

### 3.3.1 Wind Loading

Assumptions:

Exposure B Case 2 values were used for finding the $K_h$ values from Table 6-3 in Chapter 6 of ASCE7-05. $K_h$ values were conservatively used for simplification of wind loads at varying floor heights, e.g. the floor to floor height of the $1^{st}$ floor is 18 ft. A $K_h$ value of 0.62 ($K_h$ value at 20 ft) was used in the computation instead of breaking the loading up into two $K_h$ values per floor ( 0-15 ft and 20 ft loading). The width and lengths used in the calculations were taken from the base length and width of the building.

Conditions:

| Factors | Engineer of Record ASCE7-02 | | Experimental Data ASCE7-05 | |
|---|---|---|---|---|
| Basic Wind Speed | 90 mph | | 90 mph | |
| Building Category | II | | II | |
| Site Exposure | B | | B | |
| Importance Factor | 1.0 | | 1.0 | |
| External Pressure Coefficient $GC_P$ | Windward | Leeward | Windward | Leeward |
| | + 0.68 | - 0.43 | + 0.73 | - 0.64 |
| Internal Pressure Coefficient $GC_{Pi}$ | +/- 0.18 | | +/- 0.18 | |

**Table 3.6 Wind Factors**

Results:

The wind load calculations yielded a pressure of 19.22 psf in the N-S direction and a pressure 22.69 psf in the E-W direction at the top of the building. A wind loading diagram of the building can be found in the E-W direction in Appendix B. The windward $GC_P$ of +0.73 was calculated compared to the original design value of +0.64. This value is within 10% of the original design. Discrepancies may be found in the calculation gust factor calculations where certain values might have been assumed differently by either party. Table 3.7 breaks down the pressures, shears, and overturning moment at each level.

| Level | hx | Pressures | | | | Shears (k) | | Overturning Moment | |
|---|---|---|---|---|---|---|---|---|---|
| | | NS windward | NS leeward | EW windward | EW leeward | N/S | E/W | N/S | E/W |
| Penthouse | 129.67 | 13.98 | -5.24 | 13.96 | -8.73 | 0.00 | 0.00 | 9515.57 | 13466.57 |
| 11 | 114 | 13.34 | -5.24 | 13.32 | -8.73 | 49.81 | 70.86 | 5677.81 | 8077.977 |
| 10 | 103 | 13.34 | -5.24 | 13.32 | -8.73 | 90.56 | 128.84 | 4197.23 | 5971.519 |
| 9 | 94 | 12.70 | -5.24 | 12.68 | -8.73 | 129.90 | 185.13 | 3698.28 | 5291.428 |
| 8 | 85 | 12.31 | -5.24 | 12.30 | -8.73 | 168.40 | 240.41 | 3272.47 | 4698.912 |
| 7 | 76 | 11.93 | -5.24 | 11.91 | -8.73 | 206.05 | 294.68 | 2861.84 | 4124.584 |
| 6 | 67 | 11.42 | -5.24 | 11.40 | -8.73 | 242.58 | 347.60 | 2447.56 | 3545.878 |
| 5 | 58 | 10.90 | -5.24 | 10.89 | -8.73 | 277.99 | 399.18 | 2053.52 | 2991.422 |
| 4 | 49 | 10.39 | -5.24 | 10.38 | -8.73 | 312.27 | 449.41 | 1679.74 | 2461.219 |
| 3 | 40 | 9.75 | -5.24 | 9.74 | -8.73 | 345.14 | 497.95 | 1314.96 | 1941.793 |
| 2 | 31 | 9.75 | -5.24 | 9.74 | -8.73 | 392.63 | 568.07 | 1472.03 | 2173.729 |
| 1 | 18 | 7.95 | -5.24 | 7.94 | -8.73 | 450.50 | 655.73 | 1041.70 | 1577.854 |

**Table 3.7 Wind Loading Calculations**

### 3.3.2 Seismic Loading

Assumptions:

The seismic Site Classification was taken directly from the Geotechnical Report.

Self-weight was calculated by performing quantity takeoffs of the structure, façade, and roofing. Structure weight quantities were calculated by the square footage of the concrete slabs, multiplying by the thickness and then multiplying by the weight of concrete per cubic foot (150 lb/ft$^3$). Columns were also counted and quantified as well as beams and drop panels to obtain an accurate weight. The weight of the façade was taken as the weight of the concrete panels over the square feet of the elevations. A glass to concrete panel ratio was taken and then multiplied to the area for an approximation of concrete panels per elevation. Roof areas were calculated and then a weight per square foot was used to calculate the entire roof weight. The adjacent steel structure weight, for simplification, was assumed to be 10 psf and then multiplied by the area to obtain the weight.

The factors used in the seismic calculations are broken down and compared to the Engineer of Record’s in Table 3.8.

Conditions:

| Factors | Engineer of Record ASCE7-02 | | Experimental Data ASCE7-05 | |
|---|---|---|---|---|
| Seismic Use Group | I | | II | |
| Importance Factor | 1.0 | | 1.0 | |
| Seismic Design Cat. | B | | B | |
| Mapped Spectral Response Accel. | $S_S$ | $S_1$ | $S_S$ | $S_1$ |
| | 0.187 | 0.063 | 0.15 | 0.053 |
| Design Spectral Response Factors | $S_{DS}$ | $S_{D1}$ | $S_{DS}$ | $S_{D1}$ |
| | 0.20 | 0.10 | 0.16 | 0.085 |
| Site Classification | D | | D | |
| Seismic Response Coefficient ($C_s$) | 0.017 | | 0.022 | |
| Response Modification Factor (R) | 5 | | 5 | |
| Design Base Shear (V) | $695^K$ | | $779^K$ | |

**Table 3.8 Seismic Factors**

Results:

The results of the seismic loading calculations yielded a design base shear (V) of $779^K$. This is slightly larger than the Engineer of Records design base shear of $695^K$. Reasons for the discrepancy may be in the difference of the calculated self weight of the building. The assumptions and/ or take off values of the building's self-weight by the Engineer of Record were unknown. The self weight was calculated to the best of knowledge outlined in the previous assumptions. The base shear calculated is within 10% of the original design. Seismic loads per floor are broken down in Table 3.9

| LEVEL | $W_x$ (K) | $H_x$ (ft) | $W_xH_x^{1.135}$ | $C_{VX}$ | $F_X$ (k) | $V_X$ (k) | Overturning Moment (ft*K) |
|---|---|---|---|---|---|---|---|
| Penthouse | 1965.88 | 131 | 1965.88 | 0.15 | 117.05 | 117.05 | 15332.92 |
| 11 | 2457.34 | 114 | 2457.34 | 0.16 | 124.95 | 242.00 | 14244.58 |
| 10 | 1504.51 | 103 | 1504.51 | 0.09 | 68.18 | 310.18 | 7022.57 |
| 9 | 1504.51 | 94 | 1504.51 | 0.08 | 61.46 | 371.64 | 5777.19 |
| 8 | 1504.51 | 85 | 1504.51 | 0.07 | 54.83 | 426.46 | 4660.13 |
| 7 | 1504.51 | 76 | 1504.51 | 0.06 | 48.28 | 474.75 | 3669.66 |
| 6 | 1504.51 | 67 | 1504.51 | 0.05 | 41.85 | 516.60 | 2803.87 |
| 5 | 1504.51 | 58 | 1504.51 | 0.05 | 35.53 | 552.12 | 2060.66 |
| 4 | 1504.51 | 49 | 1504.51 | 0.04 | 29.34 | 581.46 | 1437.66 |
| 3 | 3788.57 | 40 | 3788.57 | 0.08 | 58.68 | 640.15 | 2347.29 |
| 2 | 6238.01 | 31 | 6238.01 | 0.09 | 72.35 | 712.50 | 2242.83 |
| 1 | 10574.48 | 18 | 10574.48 | 0.08 | 66.17 | 778.67 | 1191.13 |

**Table 3.9 Seismic Calculations**

### 3.3.3 Shear Wall Check

To check the base shear capacity of the shear walls a check was done for direct shear in the E-W direction. Six walls that resist shear in that direction where taken and a relative stiffness was calculated for each wall based on their respective lengths. All walls are 1'-0" thick. The base shear, due to seismic loading, of $779^k$ was used for the direct shear load. Shear walls #5 and #2 were checked. See calculations in the Appendix C "Shear Wall Check".

Both shear walls are more than adequately designed for the direct shear. The design of the wall was most likely controlled by another factor other than direct shear. Torsion was not considered for this Technical report and could be the reason for the extensive strength design of the wall. Further investigation of the shear wall design will come about in later technical reports.

## 4.1 CONCLUSIONS

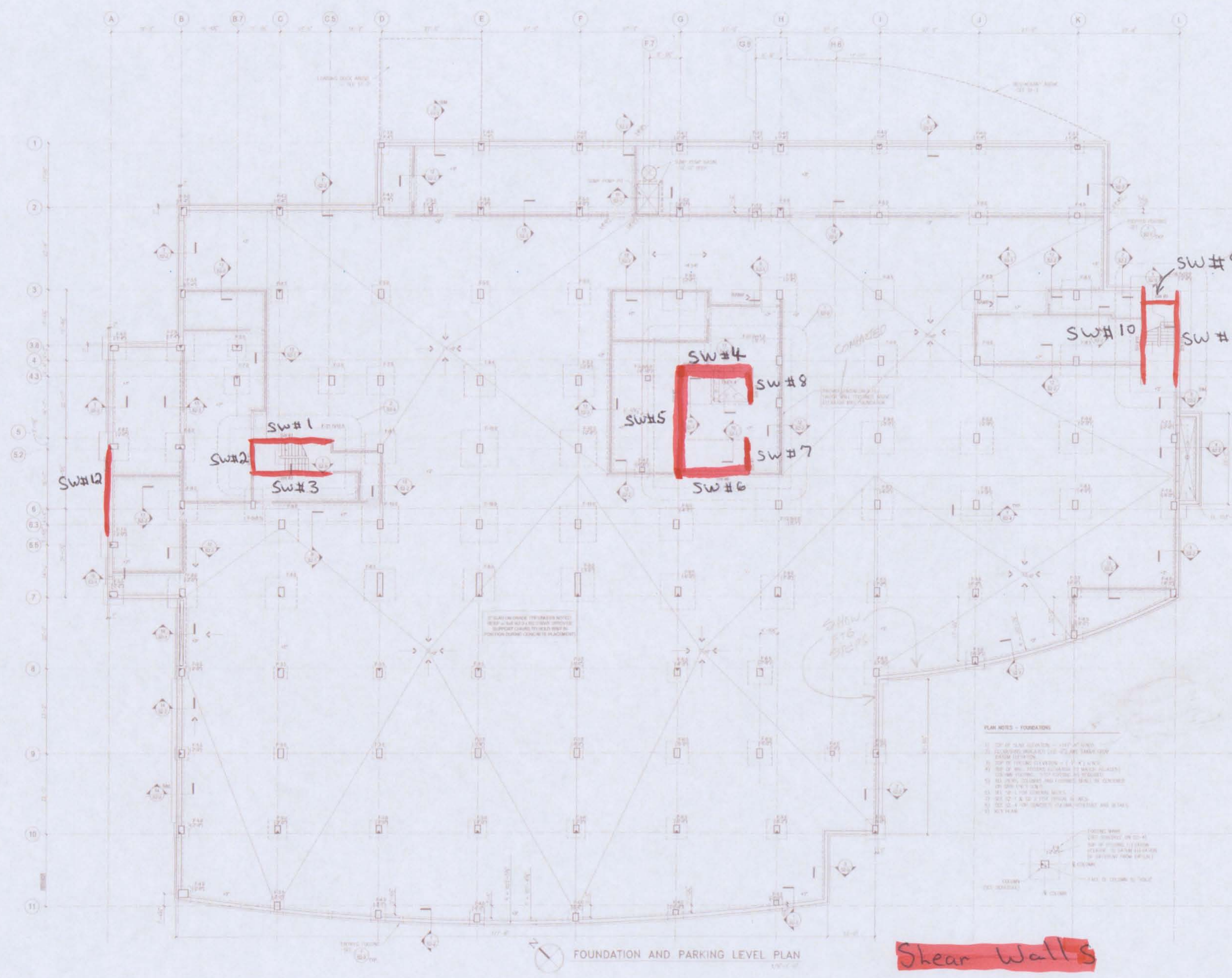
Technical Assignment 1 was worked on extensively to reverse engineer the structure of the Student's Thesis building. Spot checks were performed to give the student a better understanding of the structural design. Assumptions for loading and structural member strength calculations were made to the best of the Student's knowledge. The results for loading and structural member strengths compared closely to the Engineer of Record's results. Major discrepancies may have come in the difference of assumptions between the Student and Engineer. All calculations not available in the Appendix were completed using Excel Spreadsheets. Electronic copies of spread sheets are available upon request.

# APPENDIX A

**CODES AND STANDARDS USED BY ENGINEER OF RECORD**

1. "International Building Code", 2003 International Code Council, Inc.
2. "Minimum Design Loads for Buildings and Other Structures" (ASCE7-02), American Society of Civil Engineers
3. "Building Code Requirements for Structural Concrete ACI318-02," American Concrete Institute
4. "ACI Manual of Concrete Practice – Parts 1 through 5- 2001"
5. "Manual of Standard Practice", Concrete Reinforcing Steel Institute
6. "Post Tensioning Manual", Fifth Edition, Post Tensioning Institute
7. "Manual of Steel Construction – Allowable Stress Design", Ninth Edition, 1989 American Institute of Steel Construction
8. "Manual of Steel Construction, Volume II Connections", ASD 9th Edition/ LRFD 1st Edition, American Institute of Steel Construction
9. "Detailing for Steel Construction", American Institute of Steel Construction
10. "Structural Welding Code ANSI/AWS D1.1-96", American Welding Society
11. "Standard Specifications for Open Web Steel Joists, K-Series", Steel Joist Institute. (August 1994)
12. "Standard Specifications for Long Span Steel Joists, LH-Series and Deep Long-span Steel Joists, DLH-Series", Steel Joist Institute. (August 1994)
13. "Design Manual for Floor Decks and Roof Decks," Steel Deck Institute, 2001
14. "Load Resistance Factored Design Specification for Design of Cold-Formed Steel Structural Members", American Iron and Steel Institute 1997
15. National Design Specification for Wood Construction", 1997 (with Supplement). National Forest Products Association
16. "Performance Standard and Polices for Structural-Use Panels", PRP-108, American Plywood Association (APA)

# APPENDIX B




Shear Walls

BBGM

PROJECT NAME

Hilton Hotel at BWI Airport
Linthicum, Maryland

OWNER

CONSULTANTS

SEAL

ISSUES

FILE INFORMATION

SHEET NAME

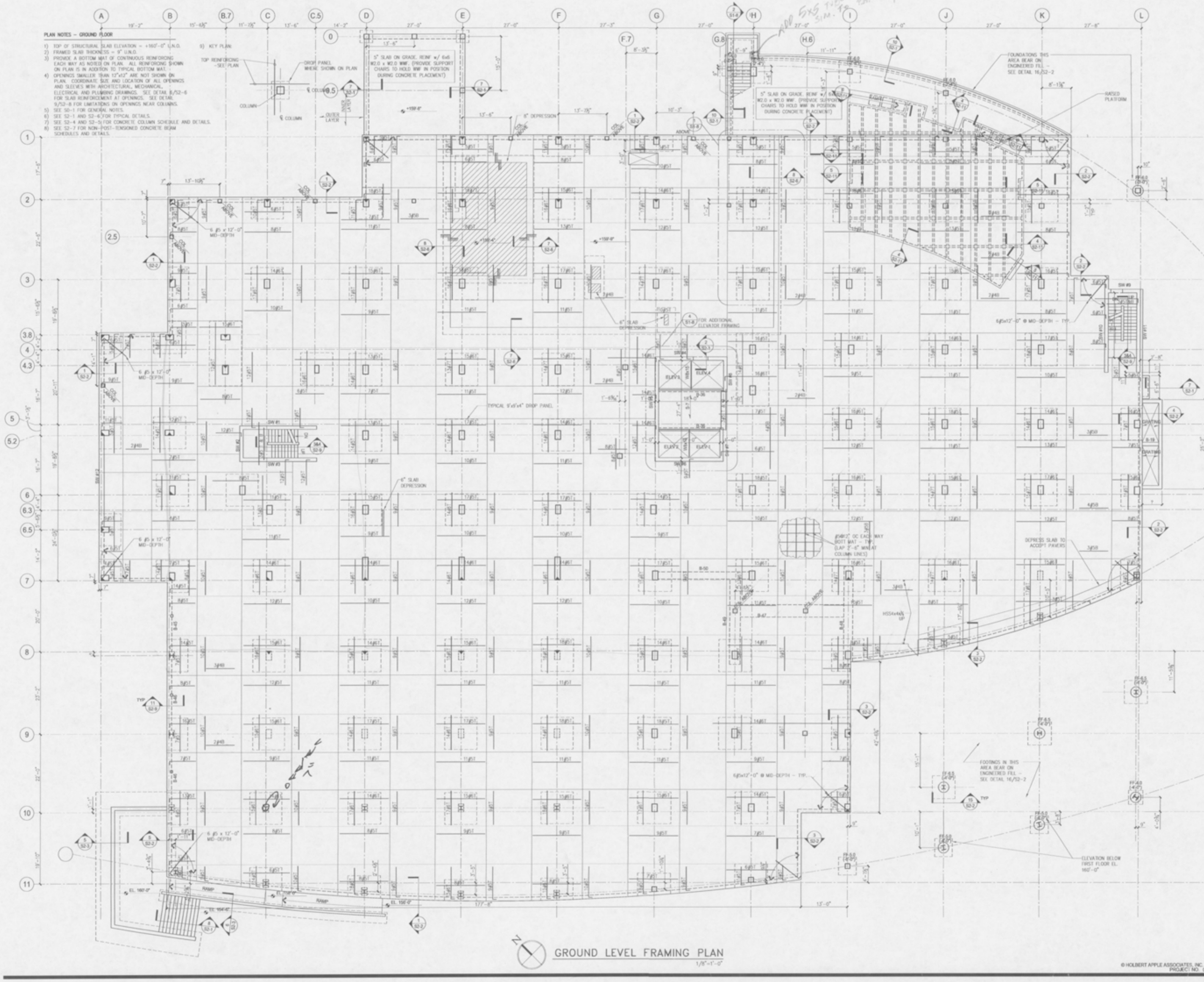
SHEET NUMBER

S1-1

BRENNAN BEER GORMAN MONK / ARCHITECTS & INTERIORS

PLAN NOTES – GROUND FLOOR

1) TOP OF STRUCTURAL SLAB ELEVATION = +160'-0" U.N.O.
2) FRAMED SLAB THICKNESS = 9" U.N.O.
3) PROVIDE A BOTTOM MAT OF CONTINUOUS REINFORCING EACH WAY AS NOTED ON PLAN. ALL REINFORCING SHOWN ON PLAN IS IN ADDITION TO TYPICAL BOTTOM MAT.
4) OPENINGS SMALLER THAN 12"x12" ARE NOT SHOWN ON PLAN. COORDINATE SIZE AND LOCATION OF ALL OPENINGS AND SLEEVES WITH ARCHITECTURAL, MECHANICAL, ELECTRICAL AND PLUMBING DRAWINGS. SEE DETAIL 6/S2-6 FOR SLAB REINFORCEMENT AT OPENINGS. SEE DETAIL 9/S2-8 FOR LIMITATIONS ON OPENINGS NEAR COLUMNS.
5) SEE S0-1 FOR GENERAL NOTES.
6) SEE S2-1 AND S2-6 FOR TYPICAL DETAILS.
7) SEE S2-4 AND S2-5 FOR CONCRETE COLUMN SCHEDULE AND DETAILS.
8) SEE S2-7 FOR NON-POST-TENSIONED CONCRETE BEAM SCHEDULES AND DETAILS.
9) KEY PLAN:

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Fax: 240.314.0667

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**SEAL**

**ISSUES**

| No. | Date | Description |
|---|---|---|
| - | - | - |
| - | - | - |
| - | - | - |
| - | - | - |
| - | - | - |
| - | - | - |
| 4 | 04-30-04 | BID/PERMIT SET |
| 3 | 03-05-04 | SPG SUBMISSION |
| 2 | 12-11-03 | HILTON SUBMITTAL |
| 1 | 08-15-03 | DRAFT SOC/BOD |

**FILE INFORMATION**

Filename 0304S1-2
Drawn -
Checked DHH
Date 03/05/04
Job # 9306.00

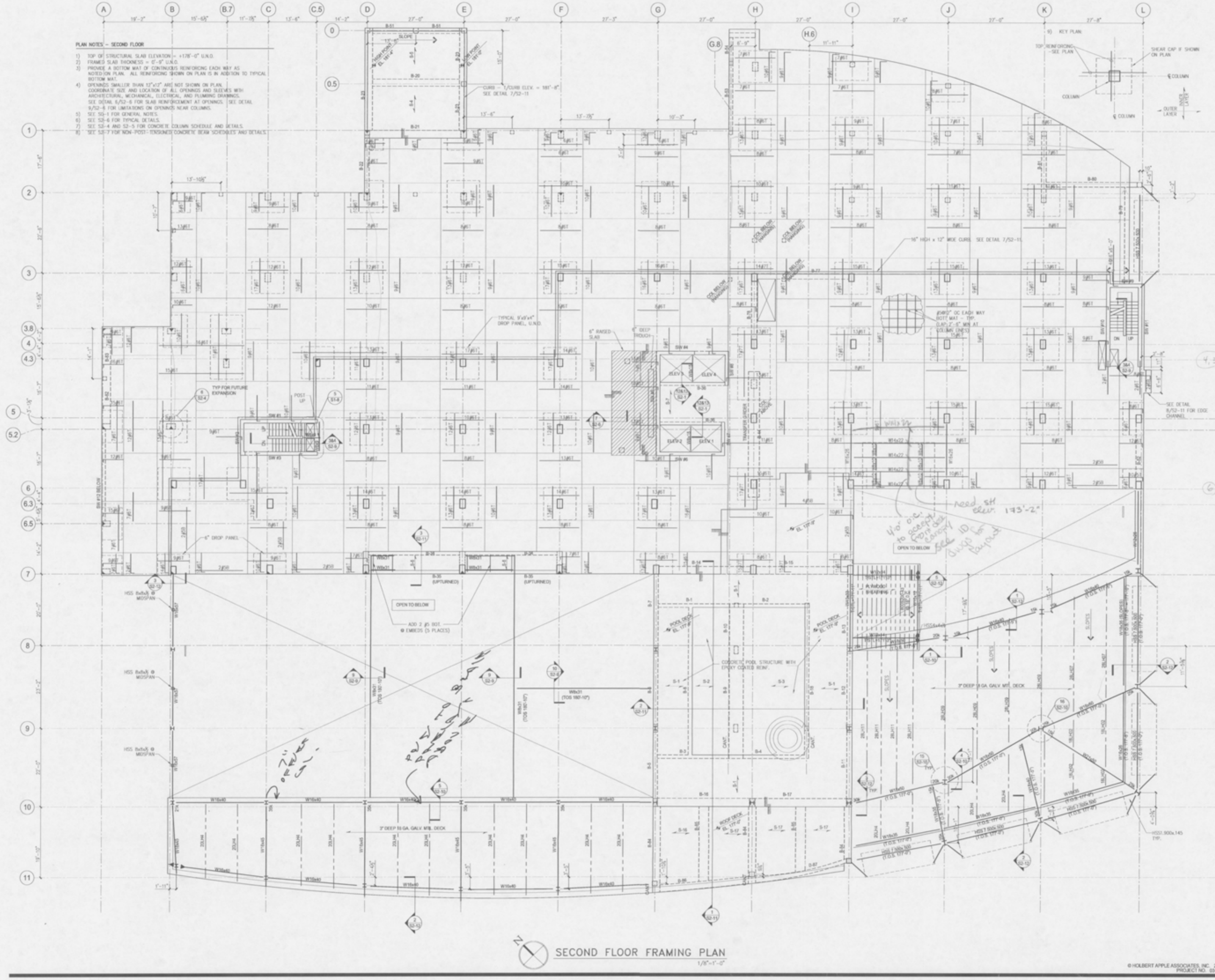
**SHEET NAME**

GROUND LEVEL
FRAMING PLAN

**SHEET NUMBER**

S1-2

BRENNAN BEER GORMAN MONK / ARCHITECTS & INTERIORS

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## PROJECT NAME

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**Linthicum, Maryland**

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SEAL

## ISSUES

| No. | Date | Description |
|---|---|---|
| 4 | 04-30-04 | BID/PERMIT SET |
| 3 | 03-05-04 | 50% SUBMISSION |
| 2 | 12-12-03 | HILTON SUBMITTAL |
| 1 | 08-11-03 | DRAFT SOC/SOD |

## FILE INFORMATION

Filename 0304151-3
Drawn -
Checked DHH
Date 03/05/04
Job # 9396.00

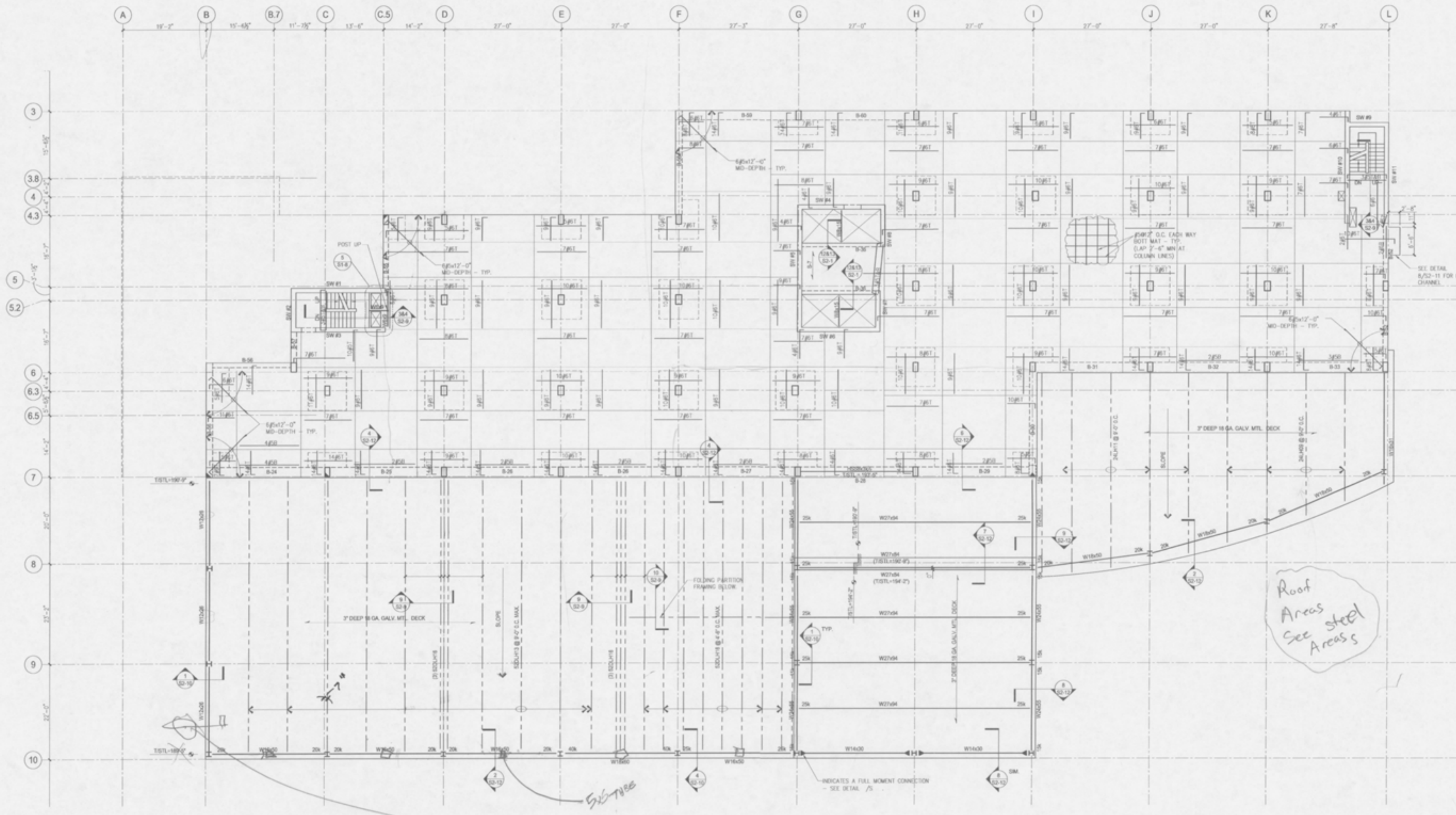
## SHEET NAME

SECOND FLOOR FRAMING PLAN

## SHEET NUMBER

S1-3

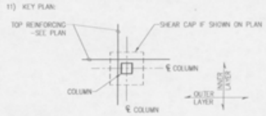
BRENNAN BEER GORMAN MONK / ARCHITECTS & INTERIORS

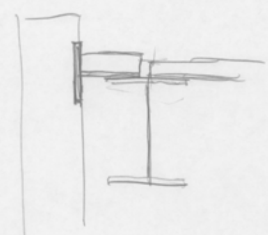



THIRD FLOOR FRAMING PLAN
1/8"=1'-0"

PLAN NOTES – THIRD FLOOR (CONCRETE)

1) TOP OF STRUCTURAL SLAB ELEVATION = +191'-0" U.N.O.
2) FRAMED SLAB THICKNESS = 0'-9" U.N.O.
3) PROVIDE A BOTTOM MAT OF CONTINUOUS REINFORCING EACH WAY AS NOTED ON PLAN. ALL REINFORCING SHOWN ON PLAN IS IN ADDITION TO TYPICAL BOTTOM MAT.
4) TOP OF STEEL ELEVATION OF BEAMS PARALLEL TO JOISTS IS TO MATCH TOP OF JOIST UNLESS NOTED OTHERWISE.
5) TOP OF STEEL IS 8" BELOW TOP OF FLOOR SLAB UNLESS NOTED OTHERWISE.
6) OPENINGS SMALLER THAN 12"x12" ARE NOT SHOWN ON PLAN. COORDINATE SIZE AND LOCATION OF ALL OPENINGS AND SLEEVES WITH ARCHITECTURAL, MECHANICAL, ELECTRICAL, AND PLUMBING DRAWINGS. SEE DETAILS 9/S2-6 FOR SLAB REINFORCEMENT AT OPENINGS. SEE DETAIL 9/S2-8 FOR LIMITATIONS ON OPENINGS NEAR COLUMNS.
7) SEE S0-1 FOR GENERAL NOTES.
8) SEE S2-6 FOR TYPICAL DETAILS.
9) SEE S2-4 AND S2-5 FOR CONCRETE COLUMN SCHEDULE AND DETAILS.
10) SEE S2-7 FOR NON-POST-TENSIONED CONCRETE BEAM SCHEDULES AND DETAILS.
11) KEY PLAN:



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SEAL

ISSUES

| No. | Date | Description |
| --- | --- | --- |
| 4 | 04-30-04 | BID/PERMIT SET |
| 3 | 03-01-04 | SPN SUBMISSION |
| 2 | 12-12-03 | HILTON SUBMITTAL |
| 1 | 08-11-03 | DRAFT SOC/BOD |

FILE INFORMATION
Filename 0304121-4
Drawn -
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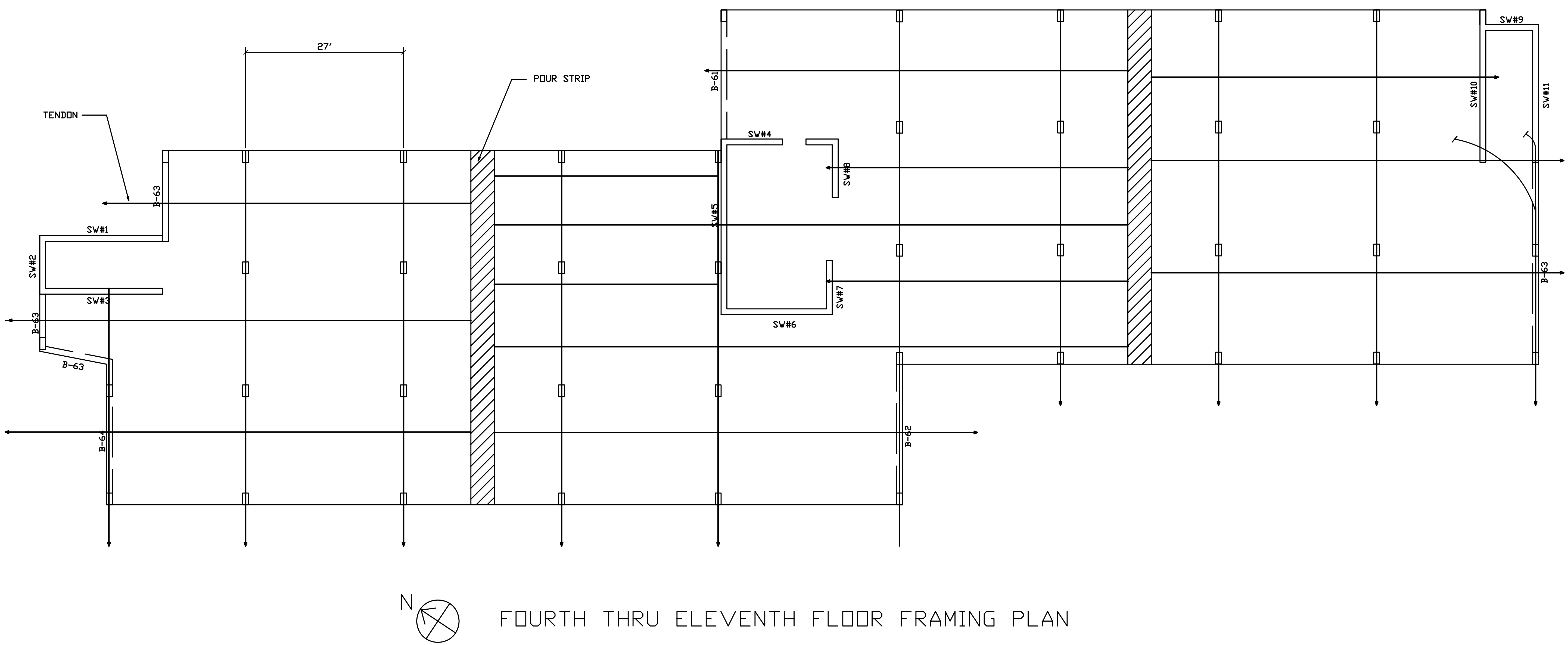
Job # 9396.00

SHEET NAME
THIRD FLOOR FRAMING PLAN

SHEET NUMBER
S1-4

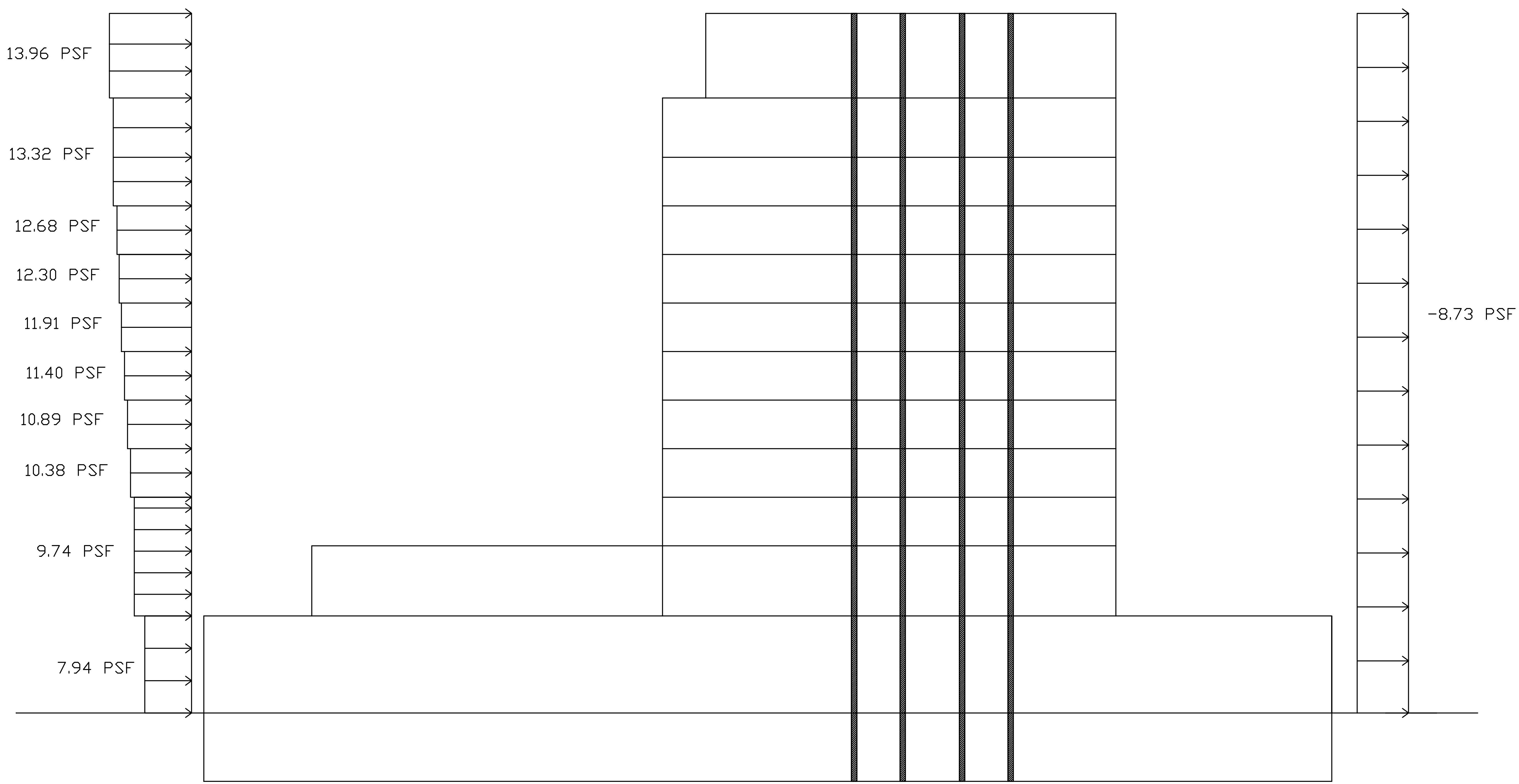


BRENNAN BEER GORMAN MONK / ARCHITECTS & INTERIORS




FOURTH THRU ELEVENTH FLOOR FRAMING PLAN

# E-W WIND PRESSURES ON BUILDING



# APPENDIX C

# Shear Wall Check

Seismic Base Shear $779^k$

Direct Shear in E-W Direction

| SW | $l$ (ft) |
|---|---|
| # 2 | 10 |
| # 5 | 29.21 |
| # 7 | 9.75 |
| # 8 | 10 |
| # 10 | 26 |
| # 11 | 23.29 |

All SW's are 1'-0" Thick

Relative K

$$K_2 = \frac{10}{10} = 1.0$$

$$K_5 = \frac{29.21}{10} = 2.92$$

$$K_7 = \frac{9.75}{10} = 0.975$$

$$K_8 = \frac{10}{10} = 1.0$$

$$K_{10} = \frac{26}{10} = 2.60$$

$$K_{11} = \frac{23.29}{10} = 2.33$$

Check SW # 5

Direct Shear

$$F_{EW} = \frac{K_i}{\Sigma K} P_{EW}$$

$$= \frac{2.92}{10.825}(779) = 210.1^k$$

Check SW # 2

$$F_{EW} = \frac{1.0}{(10.825)}(779) = 72^k$$

## SW # 5 Base Shear Capacity

From Structural Drawings:
- $f'_c = 4{,}000$ psi
- reinforcement : ASTM A615 , Grade 60 => $f_y = 60$ ksi
  #4 @ 12" each way , each face
- wall thickness = 1'-0"
- $l = 29.21$ ft
- $h = 298'-8"$ => $298.67'$

Nominal Shear capacity

$$V_u \leq A_{cv} \left( \alpha_c \sqrt{f'_c} + \rho_t f_y \right)$$

$$\frac{h_w}{l_w} = \frac{298.67}{29.21} = 10.22 > 2.0 \quad \therefore \alpha_c = 2.0$$

$$A_{cv} = (12)(29.21 \times 12) = 4206.24 \text{ in}^2$$

There are 2 curtains of Reinforcement (#4's)

$$A_{s\ell} = 2(0.20) = 0.40 \text{ in}^2$$

$$\rho = \frac{A_{s\ell}}{12 \times t} = \frac{0.40}{(12)(12)} = 0.0028$$

$$V_n = 4206.24 \left[ 2.0\sqrt{4} + (0.0028)(60) \right] = 17526^K$$

$$\phi V_n = (0.6)(17526) = \underline{10518^K > 210.1^K}$$

$\therefore$ ok

# SW # 2 Base Shear Capacity

From Stuctural Drawings

$f'_c = 4000psi$
reinforcement: 60 ksi #4 @ 12" each way, each face

wall thickness = 1'-0"
$l = 10'$
$h = 298.67'$

Nominal Shear Capacity

$$\frac{h_w}{l_w} = \frac{298.67}{10} = 29.87 > 2.0 \therefore \alpha_c = 2.0$$

$$A_{cv} = (12)(10 \times 12) = 1440 \text{ in}^2$$

2 curtains of Reinforcement

$$A_{sl} = 2(0.20) = 0.40 \text{ in}^2$$

$$\rho = \frac{A_{sl}}{12 \times t} = \frac{0.40}{(12 \times 12)} = 0.0028$$

$$V_n = 1440\left[2.0\sqrt{4} + 0.0028(60)\right]$$
$$= 6002^k$$

$$\phi V_n = 0.6(6002) = \underline{3601.2^k > 72^k}$$

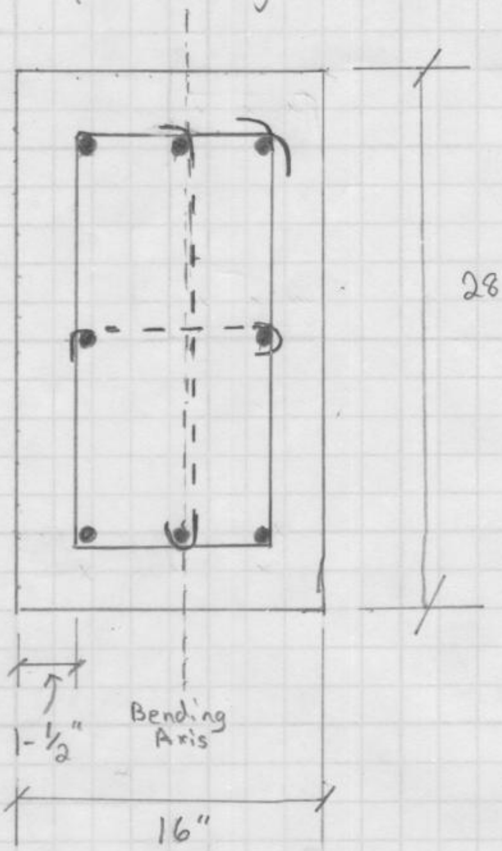
$\therefore$ ok

# Column Strength Check

## 6th Floor Column F - 5.2

$f'_c = 4000psi$
$f_y = 60\,000psi$

Size: 16" × 28"
length: 9'-0"
vertical Reinforcement: 8 - #9 bars, #3 stirrup
splice length: 5'-10"




## Axial & Bending:

Assume Bending Along weak Axis

$$\gamma = \frac{16 - 2(1.5") - 2(0.375) - 1.128}{16}$$

$$\gamma = 0.70$$

$$\rho = \frac{A_s}{bh} = \frac{8(1.0)}{(28)(16)} = 0.018$$

Assume ecentricity (e) of Load to be worse case

use $e = 8''$

$e/h = 8/16 = 0.5$

From Design Aid Graph A.6 (Design of Concrete Structures)

↳ $K_n = 0.33$

$$K_n = \frac{P_u}{\phi f'_c A_g}$$

$$P_n = K_n f'_c A_g$$

$$= (0.33)(4)(28 \times 16)$$

$$\underline{P_n = 591.4^K}$$

Assume $e = 2''$

$e/h = 2/16 = 0.13$

Design Aid A.6

↳ $K_n = 0.83$

$$P_n = (0.83)(4)(28 \times 16)$$

$$\underline{P_n = 1487.4^K}$$

Pure Axial:

$$P_o = 0.85 f'_c A_c + f_y A_{sr}$$

$$= 0.85(4)[(16 \times 28) - 8(1.0)] + 60(8)(1.0)$$

$$\underline{P_o = 1976^K}$$

6th Floor

$P_u = 906.7^k$ (Factored)
(From Table)

@ $e = 8''$

$\phi P_n = (0.65)(591.4) = 384^k$

@ $e = 2''$

$\phi P_n = (0.65)(1487.4) = 966.8^k$

@ Pure Axial

$\phi P_n = (0.65)(1976) = 1284.4^k$

## Ground Floor  Column F-5.2

$f'_c = 5600\,psi$
$f_y = 60\,000\,psi$

size: $16'' \times 28''$
length: 18'
vertical reinforcement: 8 - #10
#3 stirrup

splice length = 5'-6"

Same bar Layout as 6th floor column
(same column line)

Axial & Bending:

$$\gamma = 0.70$$

$$\rho = \frac{A_s}{bh} = \frac{(8)(1.27)}{(28)(16)} = 0.022$$

Worse Case eccentricity $e = 8''$

$$e/h = 0.5$$

Design Aid A.6

$$\hookrightarrow K_n = 0.36$$

$$P_n = (0.36)(5.6)(28 \times 16)$$

$$\underline{P_n = 903.2^k}$$

Assume $e = 2''$

$$e/h = 0.13$$

Design Aid A.6

$$\hookrightarrow K_n = 0.86\,(5.6)(28 \times 16)$$

$$\underline{P_n = 2157.6^k}$$

Pure Axial

$$P_o = 0.85\,(5.6)\left[(28 \times 16) - (8)(1.27)\right] + 60\,(8)(1.27)$$

$$\underline{P_o = 2693.7^K}$$

Ground Floor

$P_u = 1457.8^K$ (Factored)
(From Table )

@ $e = 8''$

$\phi P_n = (0.65)(903.2) = 587.1^K$

@ $e = 2''$

$\phi P_n = (0.65)(2157.6) = 1402.4^K$

@ Pure Axial

$\phi P_n = (0.65)(2693.7) = 1750.9^K$

# Punching Shear Slab Check

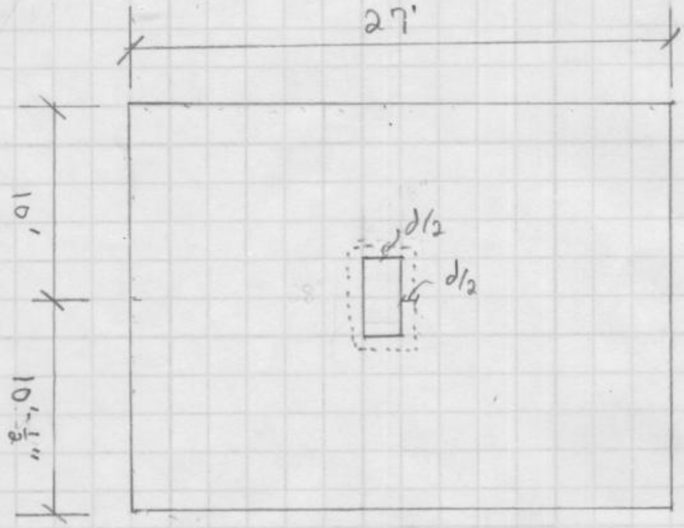
6th floor around column F-5.2 ⇒ size: 16" × 28"

2-way Post-Tension Flat Slab, Reinforced with bars top & bottom in each direction

Thickness: 7-½"

$f'_c = 4000\,psi$

Plan



$\alpha_s = 40$ (Int. Col)

$d = 7\frac{1}{2}" - 3/4"$
$= 6.75"$

$b_o = 2(16 + \frac{6.75}{2}) + 2(28 + \frac{6.75}{2})$
$= 101.5"$

$\beta_c = \frac{28}{16} = 1.75 \le 2$

Self weight of slab

$$\frac{(7.5)}{12}(150) = 43.75\ psf$$

Superimposed DL (From Engineer of Record)
10psf

Live Load 40psf
Reduction factor 0.57
↳ Reduced LL = 0.57(40) = 22.8psf

$$W_u = 1.2(43.75 + 10) + 1.6(22.8)$$
$$= 161\ psf$$

$$V_u = W_u \cdot Area$$
$$= (161)(27 \times 20.5)$$
$$= 89102^{lb}$$
$$= 89^K$$

Shear strength:

$$V_c = 4\sqrt{f'_c}\, b_o d = 4\sqrt{4000}(101.5)(6.75)\frac{1}{1000} = 173^K \checkmark$$

$$V_c = \left(2 + \frac{4}{\beta_c}\right)\sqrt{f'_c}\, b_o d = \left(2+\frac{4}{1.75}\right)\sqrt{4000}\,(101.5)(6.75)\left(\frac{1}{1000}\right)$$

$$= 186^K$$

$$V_c = \left(\frac{\alpha_s}{b_o/d} + 2\right)\sqrt{f'_c}\, b_o d = \left[\left(\frac{40}{101.5/6.75}\right)+2\right]\sqrt{4000}\,(101.5)(6.75)\left(\frac{1}{1000}\right)$$

$$= 202^K$$

$$\phi V_c = (0.75)(173)$$

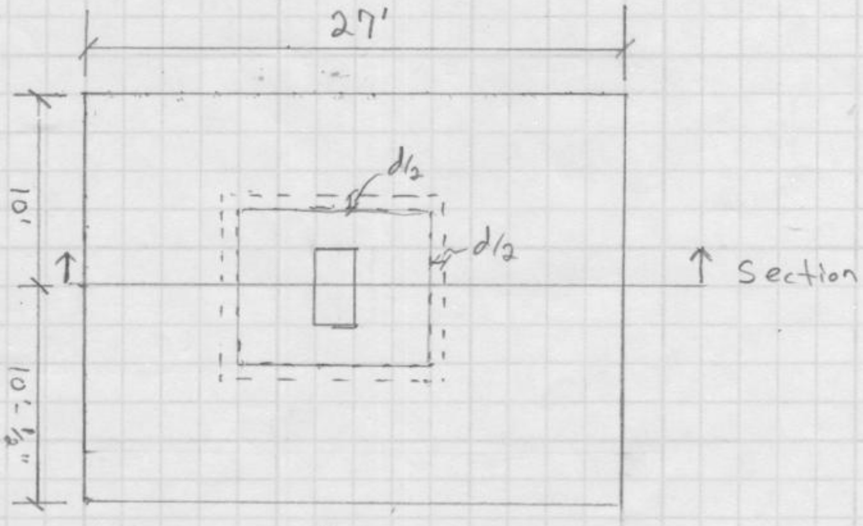
$$\boxed{\phi V_c = 129.8^K \; > \; V_u = 89^K}$$

$\therefore$ ok

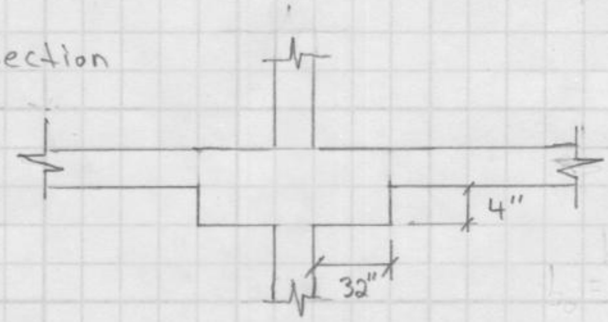
<u>Ground Floor</u> around column F-5.2 => size 16" × 28"

2-way mild reinforced Slab - 9" thick
- with 9' × 9' × 4" Drop Panel
$f'_c = 4000\,psi$

Plan




Section




$\alpha = 0.40$

$d = 9" - 3/4"$
$= 8.25$

$d = \frac{8.25}{2} = 4.125$

$b_o = 4\left[(9\times12) + 4.125\right] = 448.5"$

Self weight

$$\left[\left(\frac{9}{12}\right)\left[(27'\times 20.5') - (9'\times 9')\right](150\,\tfrac{lb}{ft^3})\right] / (27\times20.5) = 96\,psf$$

Superimposed DL (From Engineer of Record)
10 psf

Live Load 100 psf
Reduction Factor 0.57
↳ Reduced LL = 0.57(100) = 57 psf

$W_u = 1.2(96+10) + 1.6(57)$
$= 218.4\,psf$

$$V_u = (218.4)(27 \times 20.5)\left(\frac{1}{1000}\right)$$
$$= 121^K$$

Shear strength:

$$\frac{b_o}{d} = \frac{448.5}{8.25} = 54.4 > 20 \quad \therefore \text{ 2nd Eq. Governs}$$

$$V_c = \left[\left(\frac{\alpha_s}{b_o/d}\right) + 2\right]\sqrt{f'_c}\, b_o d$$

$$= \left[\left(\frac{0.40}{54.4}\right) + 2\right]\sqrt{4000}\,(448.5)(8.25)\left(\frac{1}{1000}\right)$$

$$= 469.8^K$$

$$\phi V_c = (0.75)(469.8)$$

$$\boxed{\phi V_c = 352^K > V_u = 121^K}$$