# Hampton Inn & Suites – National Harbor, MD

Photo courtesy of OTO Development, http://www.otodevelopment.com/


John Pillar

Dr. Memari Thesis Advisor


## Technical Report #1

10/5/06

# Executive Summary


Photo courtesy of STV Incorporated, http://www.stvinc.com/

This report is intended to analyze and discuss the structural system used for the Hampton Inn & Suites in National Harbor, Maryland. It is an eleven story concrete structure designed by STV Incorporated, architect, and Hope Furrer Associates, structural engineer. The hotel is set to open on March 1st, 2008.

Concrete is traditionally used in buildings such as hotels and housing facilities due to the advantages it has regarding plenum space, simplicity of design, and redundancy of use.

In this report, the building's structural systems are overviewed, and a detailed calculation of each element is included. A single shear wall, column and slab are checked for adequacy in the report, and detailed calculations are included in the appendix at the end.

In order to analyze each element, the building was analyzed using ASCE 7-05 for wind and seismic provisions. For wind, the analytical method was used, and for seismic, the equivalent force method was used.

# Table of Contents

## Structural Summary

Foundations

Foundations consist of spread or strip footings, or a combination of the two, based on the recommendations of the geotechnical report prepared by ECS Mid-Atlantic, LLC, dated December 15, 2005. Soil bearing capacity is calculated at 4500 psf. Tops of footings are assumed to be 14'-2" below finished grade, unless noted otherwise. Typical spread footings are centered below columns and range between 7' square to 15' square. These footings are up to 4' thick under shear walls, while the strip footings are typically 24" deep and 6'-6" wide, and span between the spread footings. Sixteen number 5 reinforcing bars are used longitudinally in the strip footings with eight on top and eight on the bottom. Number 4 bars spaced at 12" on center are used transversely for the top and bottom.

Concrete strengths vary according to placement. Footings and walls receive 4000 psi concrete, while slab on grade uses 3500, both normal weight. Shear walls are to match strengths called out on the column schedule. The slab on grade is reinforced with 6x6-W2.9xW2.9 WWF.

Columns

All columns are 12"x24" with chamfered edges, where exposed. There are 32 columns which span from the foundation to the roof, over 115 feet, with number 4 ties spaced at 12 inches all the way up. Vertical reinforcing ranges from ten number 11 bars to six number 8 bars. In all cases, the vertical reinforcing is distributed along the 24" face of the column in two sheets, one on each side. In all cases, class B lap splices are required for vertical splicing. Concrete strength is normal weight 6000 psi from the foundation to the third floor, where it drops to 5000 psi until it reaches the roof. Typical floor to floor heights are close to 10'.

There is a double-height pool structure on the first floor that rests on grade. Because it intersects with two column lines, the two columns start at the second floor and proceed to the roof. They cannot continue down to the foundation, so their weight is picked up by a transfer beam that is 36" deep, 44" wide, and heavily reinforced with six number 8 bars on top, ten number 11 bars on the bottom with an additional row of six number 9 bars also on the bottom. The reinforcing is tied together with number 5 closed stirrups spaced at ten inches on center. This transfer beam also frames into to two similar girders, tied into columns, at either end.

The last two columns start at the roof and help hold up a mechanical screen wall. The roof of the screen wall consists of W14x22 curved steel members with 1-1/2" galvanized metal roof deck resting on top.

Floor Slabs

The floor slabs are usually 10-1/2" thick when not near columns. At each column there is a 2-1/2" drop panel to combine for a 13" slab thickness. A typical drop panel size is 5'-6"x6'-9" and accounts for 38 square feet. Steel reinforcing is laid out longitudinally and transversely on both the bottom and top. The slab reinforcing ranges from number 4 bars to number 6 bars spaced approximately 12 inches apart. Where not specified, number 5 bars spaced at 6" is the minimum required.

For slabs on level 3 and below, concrete strength is normal weight 6000 psi. Slabs resting on the fourth floor and up have a strength of 5000 psi. Minimum reinforcing protection for floor slabs is 3/4".

The slabs on this project are considered to act as two way slabs, meaning that they carry load in both lateral directions. The three largest bays have dimensions of 29'x26'-10". There are no beams spanning between columns in this case. In the largest bay, the drop panels cover roughly 6 feet of the span, or 20.7%.

Lateral System

The lateral components of this building are comprised of twelve shear walls of varying length. Five of the twelve are aligned with Plan North, while the other seven are aligned plan East-West. Each shear wall is one foot thick and is vertically reinforced with number 5 bars at 18" on center. They are each tied into the foundation by rebar that matches vertical reinforcing called out in the plans. All rebar is to have class B splices and extend one foot into the foundation with $90^{o}$ hooks. In most cases, two columns act as bookends for each shear wall. In these cases, the shear wall reinforcement of number 5 bars spaced at 18 inches is continued into the columns and hooked $90^{o}$.

The longest shear walls are 23' along grid lines B and C running North to South. In the East-West direction, the longest shear wall is located along grid line 6, and is 19'-6" long. Nine of the twelve shear walls wrap around the two stair cases and lone elevator shaft that are spaced evenly throughout the building's long dimension.

## Code List

<u>Building Code</u>
Maryland Building Performance Standards (MBPS) – based on IBC 2003 and IRC

<u>Structural Concrete Code</u>
The American Concrete Institute (ACI) – sections 301, 318 and 315
Aggregate shall comply with ACI 304, and slump with 211.1
Reinforcing shall comply with ASTM A615, Grade 60

<u>Masonry Code</u>
ACI – section 530.1
Reinforcing shall comply with ASTM A615, Grade 60

<u>Structural Steel Code</u>
Load and Resistance Factor Design Specification (LRFD) conforming with the American Institute of Steel Construction (AISC) specification for structural steel for buildings, and AWS D1.1, latest edition
Connection bolts shall conform to ASTM A325

| | |
|---|---|
| W shapes, columns | ASTM A992 or ASTM 572-50 |
| S, M, and HP shapes | ASTM A36 |
| column baseplates, web doubler plates | ASTM A992 or ASTM 572-50 |
| channels, tees, bars, angles and plates | ASTM A36 |
| HSS rectangular or square | ASTM A500 – GR. B (Fy=46ksi) |
| steel pipe | ASTM A500 – GR. B (Fy=42ksi) |
| anchor rods | ASTM A307, A449 where noted |

## Load Summary

| | Corridor | Storage | Guest | Roof | Canopy |
|---|---|---|---|---|---|
| Slab | 148 | 148 | 148 | 148 | -- |
| M/E/C/L | 8 | 8 | 8 | 8 | 8 |
| Roof | -- | -- | -- | 2 | 2 |
| Insulation | -- | -- | -- | 8 | 8 |
| Total Dead | 156 | 156 | 156 | 166 | 16 |
| Live | 100 | 125 | 40 | 30 | 30 |
| Partition | -- | -- | 20 | -- | -- |
| Total | 256 | 281 | 216 | 196 | 48 |

## Design Wind Pressure

| Design Pressure | | ⊥55' | | | ⊥178' | | |
|---|---|---|---|---|---|---|---|
| Level | Height | p w-w | p l-w | p roof | p w-w | p l-w | p roof |
| 1 | 0 | 8.371749 | -4.5166 | -18.965 | 8.147242 | -9.5140 | -19.789 |
| 2 | 12 | 8.371749 | -4.5166 | -15.642 | 8.147242 | -9.5140 | |
| 3 | 22.25 | 9.399858 | -4.5166 | | 9.147781 | -9.5140 | |
| 4 | 32.5 | 10.5014 | -4.5166 | | 10.21979 | -9.5140 | |
| 5 | 42.75 | 11.34592 | -4.5166 | | 11.04166 | -9.5140 | |
| 6 | 53 | 12.09351 | -4.5166 | | 11.76919 | -9.5140 | |
| 7 | 63.25 | 12.681 | -4.5166 | | 12.34093 | -9.5140 | |
| 8 | 74.25 | 13.33605 | -4.5166 | | 12.97841 | -9.5140 | |
| 9 | 84.5 | 13.8501 | -4.5166 | | 13.47868 | -9.5140 | |
| 10 | 94.75 | 14.3201 | -4.5166 | | 13.93607 | -9.5140 | |
| 11 | 105 | 14.724 | -4.5166 | | 14.32914 | -9.5140 | |
| Low Roof | 115.25 | 15.09118 | -4.5166 | | 14.68648 | -9.5140 | |
| High Roof | 130 | 15.64195 | -4.5166 | | 15.22248 | -9.5140 | |

## Story Shear and Overturning Moment - Wind

| Story Shear | ⊥55' | ⊥178' |
|---|---|---|
| Level | | |
| 1 | 8.506319 | 37.72452 |
| 2 | 7.265814 | 32.22303 |
| 3 | 7.845411 | 34.04851 |
| 4 | 8.466408 | 36.00438 |
| 5 | 8.942505 | 37.50389 |
| 6 | 9.363954 | 38.83127 |
| 7 | 10.40455 | 42.79204 |
| 8 | 10.06444 | 41.0375 |
| 9 | 10.35424 | 41.95024 |
| 10 | 10.6192 | 42.78475 |
| 11 | 10.84689 | 43.5019 |
| Low Roof | 15.90682 | 63.53848 |
| Total | 118.5866 | 491.9405 |

| Overturning Moment | | |
|---|---|---|
| Level | ⊥55' | ⊥178' |
| 1 | 51.03792 | 226.3471 |
| 2 | 124.4271 | 551.8193 |
| 3 | 214.7681 | 932.0779 |
| 4 | 318.5486 | 1354.665 |
| 5 | 428.1224 | 1795.499 |
| 6 | 544.2799 | 2257.068 |
| 7 | 715.3131 | 2941.953 |
| 8 | 798.8648 | 3257.351 |
| 9 | 927.9985 | 3759.79 |
| 10 | 1060.592 | 4273.127 |
| 11 | 1194.514 | 4790.647 |
| Low Roof | 1950.574 | 7791.406 |
| Total | 8329.041 | 33931.75 |

Wind load calculations were performed according to ASCE 7-05 using method 2 – analytical procedure. $K_{zt}$ was assumed to be equal to 1.0 and the building was considered enclosed when analyzing the main wind force resisting system (mwfrs) according to case 1. Through seismic calculations, the building was determined to be rigid. Linear interpolation was used where permitted.

## Seismic Criteria

| Total Weight by Floor | | |
|---|---|---|
| Floor | Total Weight | Elevation |
| 1 | 1464840 | 0 |
| 2 | 1472841.5 | 12 |
| 3 | 1803184 | 22.25 |
| 4 | 1803184 | 32.5 |
| 5 | 1803184 | 42.75 |
| 6 | 1803184 | 53 |
| 7 | 1803184 | 63.25 |
| 8 | 1803184 | 74.25 |
| 9 | 1803184 | 84.5 |
| 10 | 1803184 | 94.75 |
| 11 | 1327969 | 105 |
| Low Roof | 1055250 | 115.25 |
| High Roof | 44464 | 130 |

| Vertical Distribution of Forces | | |
|---|---|---|
| Floor | $C_{vx}$ | $F_x$ (k) |
| High Roof | 0.00555783 | 3.56674304 |
| Low Roof | 0.11505062 | 73.833832 |
| 11 | 0.13025935 | 83.5940525 |
| 10 | 0.15740865 | 101.017135 |
| 9 | 0.13822721 | 88.7074301 |
| 8 | 0.11935803 | 76.5981204 |
| 7 | 0.09949812 | 63.8530041 |
| 6 | 0.08140748 | 52.2433173 |
| 5 | 0.06378575 | 40.9345602 |
| 4 | 0.04673032 | 29.9892254 |
| 3 | 0.030397 | 19.5073017 |
| 2 | 0.01231964 | 7.90613663 |
| 1 | 0 | 0 |
| | 1 | 641.750858 |

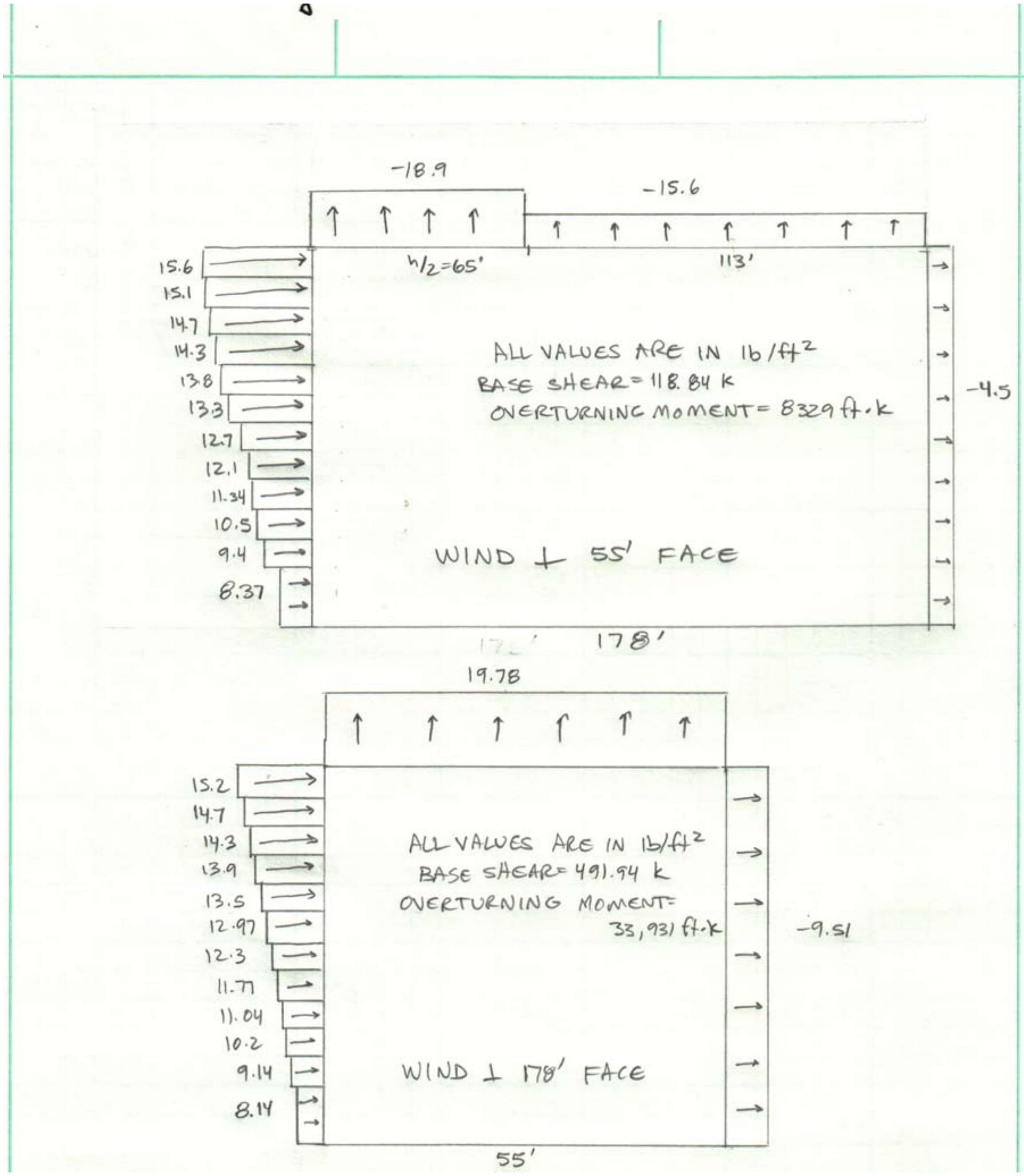
| Overturning Moment | |
|---|---|
| Level | |
| 1 | 52155.3057 |

As the vertical distribution of forces shows, seismic analysis was the controlling factor in both directions. That is, the seismic base shear, which is the same in both directions, was larger than either direction of wind base shear. This result is not surprising, as the seismic response is based on the building weight. Concrete buildings tend to carry more mass per story, and consequently are often controlled by seismic design criteria.

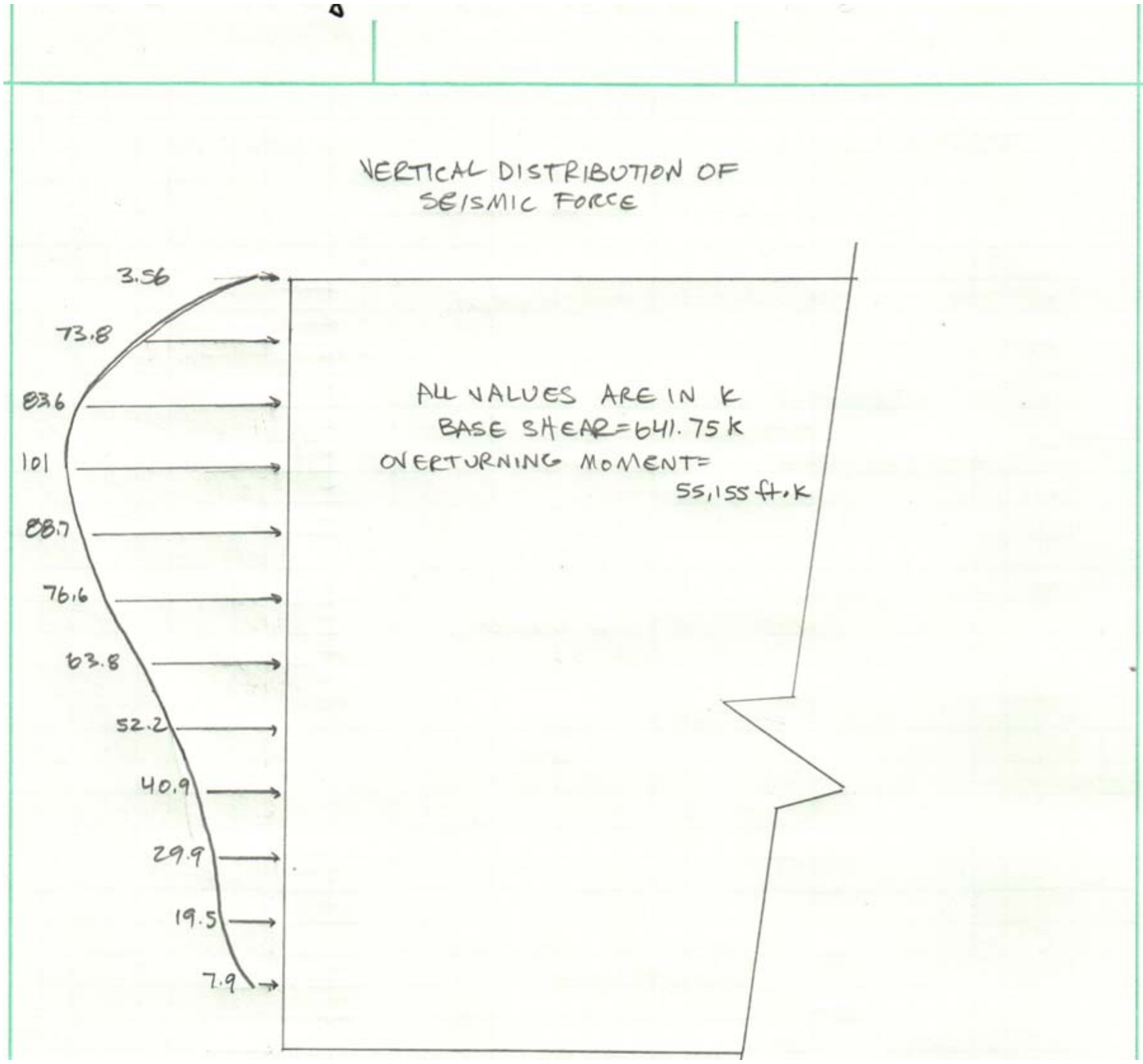
The overturning moment also turned out to be larger for seismic than wind. This can be attributed to larger forces being present at higher elevations for the seismic design. The vertical distribution of forces equation attempts to take a whiplash effect into account. As the base of the building moves one way, the top wants to catch up to it. As it does this, the base of the building switches directions and moves back, thus pulling the top of the building back to its original position with much greater force.

Once the seismic and wind forces are determined, the analysis of the lateral elements of the building can begin. Because the seismic load controls, the shear walls will be analyzed according to their relative stiffness within the group using seismic loads.

## Wind Analysis Diagram



## Seismic Analysis Diagram

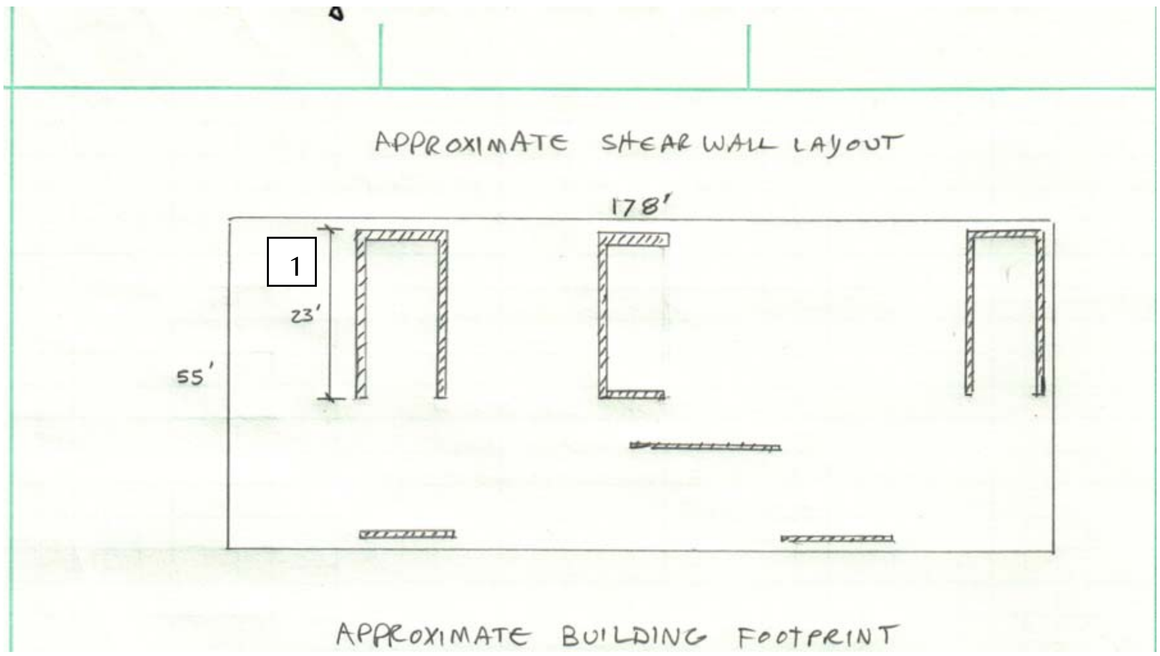


## Quick Design Spot Checks

Shear Wall Check

Estimates on how much load a certain shear wall absorbs can be made from the principle of relative stiffness, which involves direct shear, torsion and bending. After calculating the center of rigidity and the torsional constant for this building, it became clear that the overall effect of eccentric loading on the center of stiffness had a negligible impact on the outcome of the shear calculation. In fact, calculating each shear wall using the direct shear method brought me to within 99.2% of the actual shear.

I chose to analyze shear wall 1, as I have designated it:

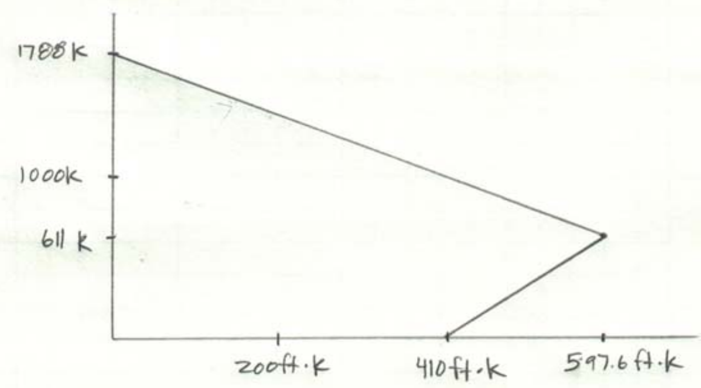



Multiplying the total seismic force by the ratio of the length of shear wall 1 to the total length of North-South oriented shear walls yielded a tributary shear of 132k. After factoring the reinforcing into shear wall 1, I found that its capacity for shear was 821k, which is greater than 132k, therefore shear wall 1 is ok. The large difference in the two numbers is to be expected, as the wall essentially acts as an extremely deep beam when subjected to a force along its axis.

Calculations for the shear capacity of shear wall 1 are included in the appendix immediately following this report.

Column Check

Column D-3 is a 12"x24" column with eight number 9 vertical reinforcing bars, 4 in each face. Assuming a cover of 1-1/2" all around, I found the pure axial capacity of the column to be 1788k. Similarly, the pure bending capacity of the column, about an axis perpendicular to the 24" side, was found to be 410 ft-k. The balanced strain condition is the last point needed to make a preliminary column interaction diagram. After calculating the balanced condition, which yielded 611k of compression and 597.6 ft-k of bending capacity, the diagram looked like this:



COLUMN INTERACTION DIAGRAM
FOR COLUMN D-3

If the actual point lies somewhere inside this conservative area, the column is deemed adequate.

Calculations for the column interaction are included in the appendix immediately following this report.

Punching Shear Check

Punching shear occurs when there is too much load on a slab where it ties in to a column. If the slab is overloaded sufficiently, the connection to the column will effectively punch through the slab from a shear failure. At column B-6 there is a 2-1/2" drop panel which helps reduce the risk of punching shear.

The total slab thickness at B-6 is 13", and the tributary area of column B-6 is roughly 237 $ft^2$. Through calculations, Vu=82k. After checking three different punching shear calculations, the least of which yielded Vc=420k, $\phi$Vc was still larger than Vu at 315k.

Slab Check

In order to qualify for using the direct design method of analyzing two-way slabs, the typical bays have to have regularity. That is, the dimensions of each bay in a three bay span cannot vary by more than a code specified distance. For this hotel, I could not find three bays that met the requirement to be analyzed using direct design. Because two way slabs are considered to be very stable in design, I conservatively analyzed a typical bay for one way slab behavior.

The bay under investigation lies between column lines F and G, and between lines 5 and 6. It is a 20' wide bay that is designed to support the design corridor loading of 100 psf. In addition to the live load, it must support its own dead load of 156 $lb/ft^3$.

The capacity of the slab with its reinforcing was calculated to be 20.15 ft-k, 18.135 ft-k after the safety factor of .9 was included. This capacity exceeded the design moment of 17.36 ft-k, so the slab is adequate.

# Appendix A
# Table of Contents

## Wind Calculations

| Wind Variables | |
|---|---|
| Variable | Value |
| h | 130 |
| V | 100 |
| Kd | 0.85 |
| I | 1 |
| Kzt | 1 |
| GCpi | 0.18 |
| qh | 21.8144 |
| Iz | 0.259931 |
| Q ⊥178' | 0.812881 |
| Q ⊥55' | 0.853623 |
| G ⊥178' | 0.821083 |
| G ⊥55' | 0.843709 |

| Velocity Pressures by Floor | | | | |
|---|---|---|---|---|
| Level | Height | Kz | qz | |
| 1 | 0 | 0.57 | 12.4032 | |
| 2 | 12 | 0.57 | 12.4032 | |
| 3 | 22.25 | 0.64 | 13.9264 | |
| 4 | 32.5 | 0.715 | 15.5584 | |
| 5 | 42.75 | 0.7725 | 16.8096 | |
| 6 | 53 | 0.8234 | 17.91718 | |
| 7 | 63.25 | 0.8634 | 18.78758 | |
| 8 | 74.25 | 0.908 | 19.75808 | |
| 9 | 84.5 | 0.943 | 20.51968 | |
| 10 | 94.75 | 0.975 | 21.216 | |
| 11 | 105 | 1.0025 | 21.8144 | |
| Low Roof | 115.25 | 1.0275 | 22.3584 | |
| High Roof | 130 | 1.065 | 23.1744 | qh |
| Parapet | 132 | 1.07 | 23.2832 | |

| Cp by Wind Direction | | |
|---|---|---|
| | Cp ⊥55' | Cp ⊥178' |
| Windward | 0.8 | 0.8 |
| Leeward | -0.231 | -0.5 |
| Side | -0.7 | -0.7 |
| Roof | 0-h/2 - 0.97<br>>h/2 - 0.8 | -1.04 |

## Seismic Calculations

### Weight Inputs, Slabs

| Floor | Thickness | Material | Area | Area Voids | Approx. Weight |
|---|---|---|---|---|---|
| 1 | 10.5 | 150 | 9790 | 2250 | 989625 |
| 2 | 10.5 | 150 | 9790 | 2700 | 930562.5 |
| 3 | 10.5 | 150 | 9790 | 750 | 1186500 |
| 4 | 10.5 | 150 | 9790 | 750 | 1186500 |
| 5 | 10.5 | 150 | 9790 | 750 | 1186500 |
| 6 | 10.5 | 150 | 9790 | 750 | 1186500 |
| 7 | 10.5 | 150 | 9790 | 750 | 1186500 |
| 8 | 10.5 | 150 | 9790 | 750 | 1186500 |
| 9 | 10.5 | 150 | 9790 | 750 | 1186500 |
| 10 | 10.5 | 150 | 9790 | 750 | 1186500 |
| 11 | 10.5 | 150 | 9790 | 750 | 1186500 |
| Low Roof | 10.5 | 150 | 9790 | 1750 | 1055250 |
| High Roof | n/a | 11.2 | 3970 | 0 | 44464 |
| High Roof | 6 | 150 | 290 | 0 | 21750 |
| | | | | | 13720151.5 |

### Weight Inputs, Columns & Shearwalls

| Mark | Area ($ft^2$) | Quantity | Height | Material | Approx. Weight |
|---|---|---|---|---|---|
| Numerous | 2 | 32 | 130 | 150 | 1248000 |
| I3, J3 | 2 | 2 | 130 | 150 | 78000 |
| H1, I1, J1, J4 | 2 | 4 | 12 | 150 | 14400 |
| G7, F7 | 2 | 2 | 15 | 150 | 9000 |
| SW1 | 23.33 | 3 | 130 | 150 | 1364805 |
| SW2 | 12 | 2 | 130 | 150 | 468000 |
| SW3 | 23.33 | 1 | 130 | 150 | 454935 |
| SW4 | 22.25 | 1 | 130 | 150 | 433875 |
| SW5 | 8.5 | 2 | 130 | 150 | 331500 |
| SW6 | 19.5 | 1 | 130 | 150 | 380250 |
| SW7 | 12 | 2 | 130 | 150 | 468000 |
| | | | | | 5250765 |

## Additional Weight

| Floor | Type | Amount | Type | Amount | Total |
|---|---|---|---|---|---|
| 1 | n/a | | n/a | | 0 |
| 2 | Partition | 64720 | Storage | 2344 | 67064 |
| 3 | Partition | 136000 | Storage | 5469 | 141469 |
| 4 | Partition | 136000 | Storage | 5469 | 141469 |
| 5 | Partition | 136000 | Storage | 5469 | 141469 |
| 6 | Partition | 136000 | Storage | 5469 | 141469 |
| 7 | Partition | 136000 | Storage | 5469 | 141469 |
| 8 | Partition | 136000 | Storage | 5469 | 141469 |
| 9 | Partition | 136000 | Storage | 5469 | 141469 |
| 10 | Partition | 136000 | Storage | 5469 | 141469 |
| 11 | Partition | 136000 | Storage | 5469 | 141469 |

## Seismic Inputs

| Variable | Value |
|---|---|
| $S_s$ | 0.152 |
| $S_1$ | 0.5 |
| $F_a$ | 1.6 |
| $F_v$ | 2.4 |
| I | 1 |
| $SM_s$ | 0.2432 |
| $SM_1$ | 1.2 |
| $SD_s$ | 0.16213333 |
| $SD_1$ | 0.8 |
| R | 5 |
| $C_s$ | 0.03242667 |
| $C_t$ | 0.02 |
| $h_n$ | 130 |
| x | 0.75 |
| $T_a$ | 0.7699943 |
| $T_o$ | 0.98684211 |
| $T_s$ | 4.93421053 |
| V (k) | 641.750858 |

## Portion of Columns & Shearwalls Shared by Floor

| Floor | Attributed Weight |
|---|---|
| 1 | 498615 |
| 2 | 475215 |
| 3 | 475215 |
| 4 | 475215 |
| 5 | 475215 |
| 6 | 475215 |
| 7 | 475215 |
| 8 | 475215 |
| 9 | 475215 |
| 10 | 475215 |
| 11 | 475215 |

## Weight Seen by Floor

| Floor | Weight | Story Shear |
|---|---|---|
| High Roof | 44464 | 1.44181931 |
| Low Roof | 1099714 | 35.6600593 |
| 11 | 2427683 | 78.7216674 |
| 10 | 4230867 | 137.192914 |
| 9 | 6034051 | 195.66416 |
| 8 | 7837235 | 254.135407 |
| 7 | 9640419 | 312.606653 |
| 6 | 11443603 | 371.0779 |
| 5 | 13246787 | 429.549146 |
| 4 | 15049971 | 488.020393 |
| 3 | 16853155 | 546.491639 |
| 2 | 18325996.5 | 594.25098 |
| 1 | 19790836.5 | 641.750858 |

## Shear Wall Calculations

Shear wall check

$l = 23'$

Total length of shear walls = 112'  $\frac{23}{112} = 0.205$

$(0.205)(641.75) = 131.6k$ to shear wall 1

Wall reinforcement

From ACI 21.7.2.2.

If $V_u \geq 2 A_{cv} \sqrt{f'_c}$, need 2 layers of reinf.

$2(12)(23)(12)\sqrt{6000} = 513.1k < V_u$

$\therefore$ 2 layers needed

$$\rho_l, \rho_t = \frac{A_{sl}}{A_{cv}} \geq 0.0025$$

$A_{cv} = (144 \text{ in}^2/\text{ft})(.0025) = 0.36 \text{ in}^2/\text{ft}$ req'd

Assume #5

$A_{sl} = 0.62 \text{in}^2 / S$ , $S$ = spacing

$\frac{0.36}{12} = \frac{0.62}{S} \Rightarrow S = 20.67''$ MAX

try #5 @ 18" o.c., both directions

$V_u = A_{cv}(\alpha_c \sqrt{f'_c} + \rho_t f_y)$  $\frac{h_w}{l_w} = \frac{130}{23} = 5.65 > 2$

$\alpha_c = 2.0$

$A_{cv} = 3312 \text{in}^2$  $\rho_t = 0.0043$

$V_u = 3312(2\sqrt{6000} + .0043 \cdot 60{,}000) / 1000 = 1367.6k$

$\phi V_n = 0.6(1367.6) = 820.55k > V_u$

$\therefore$ ok

Shear wall (cont'd)

$M_u = 10{,}366$ ft·k
$P_u = 1605$ k

$C_u = \frac{1604}{2} + \frac{10366}{23} = 1253k$ (BE)

$A_g = 23 \text{ ft}^2$
$I_g = \frac{(23)^3}{12} = 1014 \text{ ft}^4$
$f_c = \frac{P_u}{A_g} + \frac{M_u \cdot \frac{h_w}{2}}{I_g} = \frac{1604}{23} + \frac{10366\left(\frac{23}{2}\right)}{1014} = 187 \text{ k/ft}^2$
$= 1.3 \text{ k/in}^2$

$0.2(6) = 1.2 \text{ k/in}^2 < 1.3$ ∴ need boundary element

10-#9 vert in each boundary @ level 1

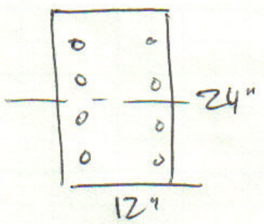
$A_{st} = 10 \text{ in}^2$ $\rho_{st} = \frac{10}{12(48)} = 0.0174$ $A_g = 48(12) = 576 \text{ in}^2$

$\rho_{min} = 0.01 < \rho_{st} < \rho_{max}$ ∴ ok

$\phi P_{n\,MAX} = 0.8\,\phi\,[(1.85 f'_c(A_g - A_{st}) + f_y A_{st}]$
$= 0.8(0.7)\,[(1.85)(6000)(576-10) + (60{,}000)(10)]$
$= 1952k > 1253k$
∴ ok

## Column Calculations

Column Interaction




$f'_c = 6\text{ ksi}$
$f_y = 60\text{ ksi}$
8-#9 bars

Pure axial $P_0 = (0.85)(6)(12 \times 24 - 8.0) + (8)(60)$
$= 1788\text{k}$

Balanced Condition

$\varepsilon_y = 60/29{,}000 = 0.00207$

$c = \dfrac{.003}{.003 + .00207}(22.5) = 11.84"$

$\varepsilon_{s1} = \dfrac{.003}{11.84}(11.84 - 1.5) = 0.00262 \quad f_{s1} = 60\text{ ksi}$

$\varepsilon_{s2} = \dfrac{.003}{11.84}(11.84 - 8.5) = 24.5\text{ ksi}$

$\varepsilon_{s3} = \dfrac{.003}{11.84}(11.84 - 15.5) = -26.9\text{ ksi}$

$\varepsilon_{s4} = -60\text{ ksi}$

$M_b = (.85)(6)(12)(.85)(11.84)\left(12 - \dfrac{(.85)(11.84)}{2}\right)$
$+ 2(60)(12 - 1.5) + 2(24.3)(12 - 8.5)$
$+ 2(-26.9)(12 - 15.5) + 2(-60)(12 - 22.5) = 597.6\text{ ft}\cdot\text{k}$

$P_b = (.85)(6)(12)(.85)(11.84) + 2(60) + 2(24.5) + 2(-26.9)$
$+ 2(-60) = 611\text{ k}$

pure bending  assume 2 don't yield, 2 do

$f_{s_1} = \frac{.003}{c}(c-1.5)(29k) =$

$f_{s_2} = \frac{.003}{c}(c-8.5)(29k)$

$f_{s_3} = f_{s_4} = -60$

$\Sigma F = 0 \quad (.85)(6)(12)(.85)c + 2f_{s_1} + 2f_{s_2} + 2f_{s_3} + 2f_{s_4}$

$0 = 52c + \frac{174}{c}(c-1.5) + \frac{174}{c}(c-1.5) - 4(60)$

$0 = 52c^2 + \frac{348}{\cancel{c}}(c-1.5) - 240c$

$-52c^2 + 240c = .348c - 522$

$-52c^2 - 108c + 522 = 0$

$\frac{108 \pm \sqrt{108^2 - 4(-52)(522)}}{2(-52)}$

$52c - 240 + \frac{.003}{c}(c-1.5)(29000)(4) = 0$

$52c^2 - 240c + \frac{348}{\cancel{c}}(c-1.5) = 0$

$52c^2 - 240c + 348c - 522 = 0$

$52c^2 + 108c - 522 = 0$

$\frac{-108 \pm \sqrt{108^2 - 4(52)(-522)}}{2(52)} = \frac{-108 \pm 347}{104}$

$c = 2.3''$

$$f_{s_1} = \frac{.003}{2.3}(2.3-1.5)(29000) = 30.3k$$

$$f_{s_2} = \frac{.003}{2.3}(2.3-8.5) > -60 \text{ ksi} \therefore \text{yielded}$$

$$.85(6)(12)(.85)c + \frac{.003}{c}(c-1.5)(29000)(2) = 360$$

$$52c - 360 + \frac{.003}{c}(c-1.5)(29000)(2)$$

$$52c - 360 + \frac{174}{c}(c-1.5) = 0$$

$$52c^2 - 360c + 174c - 261 = 0$$

$$\frac{186 \pm \sqrt{186^2 - 4(52)(-261)}}{104}$$

$$c = 4.655''$$

$$f_{s_1} = 58.9k$$

$$f_{s_2} = \frac{.003}{4.655}(4.655-8.5) = -60$$

$$f_{s_3} = f_{s_4} = -60$$

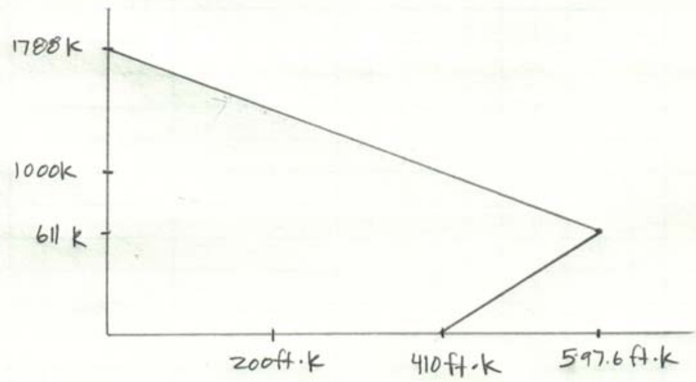
$$M_o = .85(6)(12)(.85)(4.655)\left(12 - \frac{.85(4.655)}{2}\right) \quad 2427$$

$$+ 2(58.9)(12-1.5) + 2(-60)(12-8.5)$$

$$+ 2(-60)(12-15.5)$$

$$+ 2(-60)(12-22.5) = 4924$$

$$= 410 \text{ ft}\cdot\text{k}$$

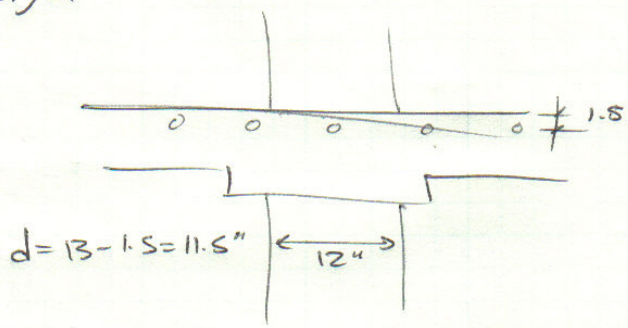



COLUMN INTERACTION DIAGRAM
FOR COLUMN D-3

## Punching Shear Calculations

Punching Shear Design
@ Col. B-6

Trib. Area = 237 ft$^2$




$b_o = 118"$

| | D | L | A |
|---|---|---|---|
| corridor | 156 | 100 | 142 ft$^2$ |
| guest | 156 | 66 | 40 ft$^2$ |
| storage | 156 | 125 | 55 ft$^2$ |
| | | | 237 ft$^2$ |

$1.2(156) + 1.6(100)(142) = 49{,}302$ lb
$1.2(156) + (1.6)(60)(40) = 11{,}328$ lb
$1.2(156) + 1.6(125)(55) = 21{,}296$ lb
$81{,}926$ lb $= 82$k

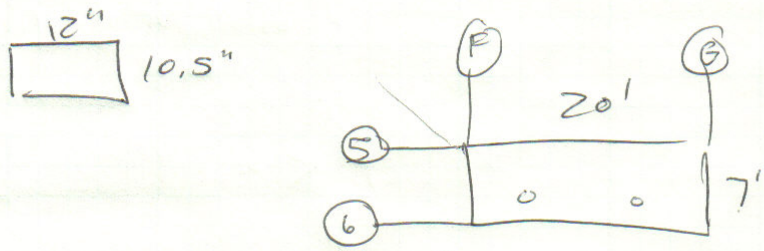
$V_c = 4\sqrt{6000}(118)(11.5) = 420$k

$V_c = (2 + \frac{4}{2})\sqrt{6000}(118)(11.5) = 420$k ← smallest

$V_c = (\frac{40(11.5)}{118} + 2)\sqrt{6000}(118)(11.5) = 619.9$k

$\phi V_c = 0.75(420) = 315$k $< V_u$
$\therefore$ ok

## Slab Calculations




$f'_c = 6000$

$f_y = 60\ ksi$

$A_s = 0.44\ in^2$      #6 @ 12" bottom

$d = 10.5 - 3/4 - 3/8 = 9.375$

$$a = \frac{.44(60)}{.85(6)(12)} = 0.43$$

$$M_n = .44(60)(9.375 - .215) = 20.15\ ft{\cdot}k$$

$$S_{max} = \frac{12(36)}{.6(60)} = 12"$$

$$\rho = \frac{.44}{12(9.375)} = 0.0039 > 0.0018 \therefore ok$$

Ductility: $c = \frac{a}{\beta_1} = 0.8 < .375(9.375)$

$0.8 < 3.5 \therefore ok$    $\phi = 0.9$

$$\phi M_n = (.9)(20.15) = 18.135\ ft{\cdot}k$$

$$M_u = \frac{wl^2}{8} = \frac{[(1.2)(156)+(1.6)(100)](20')^2}{8} = 17.36\ ft{\cdot}k$$

$\phi M_n > M_u \therefore ok$