# Structural Depth Study Summary

**Proposal Summary:**

*Proposal:*

Given its current location the JW Marriott is not likely to experience high seismic loads during its lifetime. If the owner wished to use the same design in Monterey, California, a seismically active location, the design will need to change. I propose to redesign the structure to withstand the forces conforming of the new seismically active site.

*Solution:*

The primary focus will be to keep as many characteristics the same for the new design. I will redesign the JW Marriott in accordance with ASCE7-05 and IBC2006 for Monterey, California. The main structural focus will be on the lateral force resisting system.

*Breadth Topics:*

There will be many affects on the JWM if significant changes must be made in the redesign for Monterey. Specifically, architecture and construction management topics will be compared and contrasted to the original design. Alternative floor plans for public and typical floors will be provided to exhibit the effects of the new system on architecture. A cost and schedule duration comparison will be studied in order to make a thorough conclusion concerning the new design.

**Structural Codes:**

- *Building Code*
  IBC 2006. The 2003 Michigan Building Code, an adoption of the IBC 2003 was used for the original design. This will be updated to IBC 2006 for the redesign.
- *Seismic Forces*
  ASCE7-05. Seismic forces will be determined in accordance with the newest version of ASCE7 for the Monterey redesign.
- *Structural Concrete*
  ACI 318-2002. Building Code Requirements for Structural Concrete.
- *Concrete Masonry*
  ACI 530-1999. Building Code Requirements for Masonry Structures.
- *Structural Steel*
  LRFD Specification for Structural Steel Buildings, 2$^{nd}$ Edition. AISC.

# Depth: Lateral Force Resisting System

**Introduction:**

In order to resist the larger seismic forces it was necessary to create a new design for the entire resisting system. Previously, one large "I-shaped" section was used to resist lateral movement. A much larger section, or collection of sections, was needed to withstand the new forces. It was found that four smaller "I-shaped" sections connected using coupling beams was the best design option for the new location.

The JWM redesign was penalized in numerous ways in order to guarantee an industry standard acceptable design that would easily pass peer review. The methods, assumptions, and results of the lateral system redesign are presented herein.

Nonlinear response will be designed to have plastic hinging occur in only the coupling beams, the shear walls will be designed to incur only flexural yielding. If the shear walls are governed by flexural yielding they will maintain their lateral-force resistance through large displacements. The building will deform in a manner that distributes deformations over the height of the structure. This manner of nonlinear action will guarantee seismic force dissipation while maintaining the integrity of the structure.

**Peer Review:**

According to Section 12.2.5.4 of ASCE7 the building height limitation for special reinforced concrete shear walls is 240 ft. for structures in seismic design category D or E. The JWM exceeds the height limitation at 256 ft and will require peer review per Section 16.2.5. The JWM will be subjected to the following considerations during peer review:

1. Review of acceptance criteria used to demonstrate the adequacy of structural elements and systems to withstand the calculated force and deformation demands, together with that laboratory and other data used to substantiate these criteria.
2. Review of the preliminary design including the selection of structural system and the configuration of structural elements.
3. Review of the final design of the entire structural system and all supporting analyses.

In order to insure an expeditious peer review measures will be taken to design the lateral system to exceed code requirements and limitations. The Maximum Considered Earthquake (MCE) will be for a 1000 year event, much greater than the code required MCE of a 50 year event. In addition, flexure and shear capacities will be designed much greater than code requirements.

**Spectral Response Parameters:**

The site specific spectral response parameters, presented in Table 1, have been determined in accordance with ASCE7-05. Detailed calculations are available upon request.

| Table 1. Spectral Response Parameters | | |
|---|---|---|
| Occupancy Category | | II |
| Importance Factor | I= | 1.0 |
| Seismic Design Class | SDC= | D |
| Site Class | SC= | C |
| Site Coefficients | $F_a$= | 1.0 |
| | $F_v$= | 1.3 |
| Spectral Response Accelerations | $S_s$= | 1.24 |
| | $S_1$= | 0.61 |
| Design Spectral Response Accelerations | $S_{DS}$= | 0.97 |
| | $S_{D1}$= | 0.53 |
| Period | T= | 1.792 |
| Dynamic Period | $T_d$= | 2.94 |
| Redundancy Factor | ρ= | 1.3 |
| Response Modification Coefficient | R= | 5 |
| Over Strength Factor | Ω= | 2.5 |
| Deflection Amplification Factor | $C_d$= | 4.5 |
| Seismic Response Coefficient | $C_s$= | 0.0590 |
| Torsion Amplification Factor | $A_x$= | 1.343 |

**Analysis Methods and Assumptions:**

Equivalent lateral force method (ELF) and ETABS computer analysis, Figure 2, was manipulated to determine those lateral forces for which the JWM must be designed.

The masses calculated for ELF assume a reduction in perimeter vertical walls. The removal of wall-columns in favor of circular columns is discussed in detail in the gravity system section. Normal weight concrete and identical Grand Rapids floor loads were used throughout these calculations.

The ETABS analysis method assumed several parameters in order to create an accurate model. The basement levels are not to be analyzed in the model. Ground level was assumed as the seismic base and with fixed support conditions. Automated ELF calculations will be done by ETABS and must closely match those calculated by hand before proceeding further. Dynamic analysis will investigate 12 mode shapes. The Dynamic results will be scaled to match ELF output before any Code allowable reduction in forces may be applied.

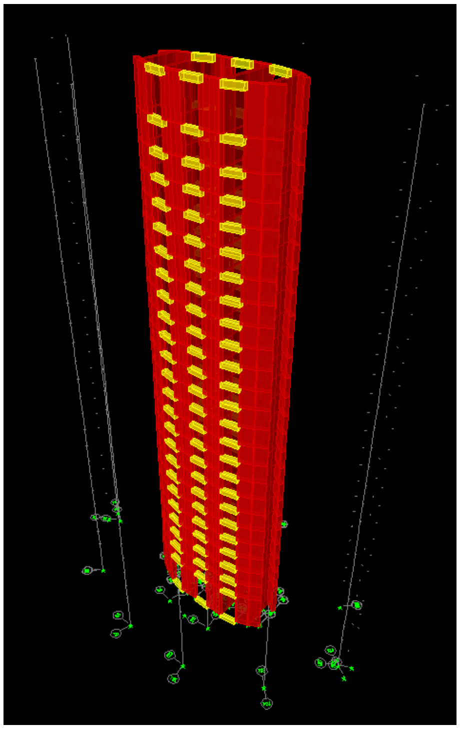

**Figure 2. Etabs model**

**Preliminary Design:**

An estimated moment of inertia, about the weak axis, was prepared using an industry proven formula. Equation 1 estimates the period of a real building by approximating it as a uniformly loaded prismatic beam. This equation takes into account the weight, height, typical floor area, and lateral system inertia of a building. The period may then be manipulated by increasing or decreasing the inertia to attain an acceptable amount of interstory drift.

$$T = \frac{2\pi}{3.52}\sqrt{\frac{\mu \times H^4}{EI}} \qquad \text{(Equation 1.)}$$

Due to the unique shape of the proposed core**,** shown in Figure 3, a shear force investigation was carried out. Shear stresses were redistributed according to stiffness and the maximum stiffness value, “K”, was determined using Equation 2. This value should be kept less than or equal to 4 in order to satisfy any peer review dilemma. The shear K value was then penalized by the redundancy factor, ρ, and the distribution factor (determined from relative stiffness). Equation 2 is derived from Section 21.7.4 of ACI-318.

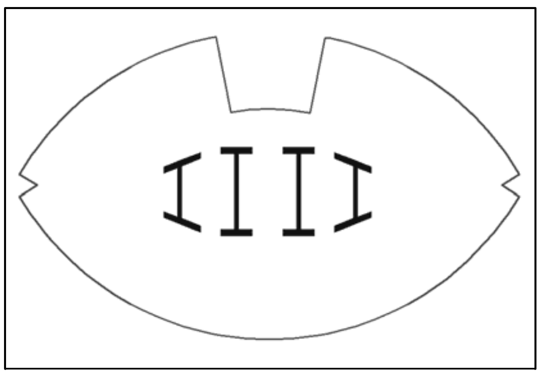

**Figure 3. Preliminary core design**

$$K_{Vave} = \left[ \frac{V_u / \phi}{A_v \sqrt{f'_c}} \times \rho \times DF \right] \leq 4 \qquad \text{(Equation 2.)}$$

**Static Force Analysis:**

The equivalent lateral force method was used in conjunction with ETABS modeling to determine static forces, story shears, and base shear. The ELF results were attained in agreement with Section 12.8 of ASCE7-05. The ELF results are presented in Table 2 below. ETABS static analysis was to verify manual findings. An acceptable error of less than 2% was found between these automated and manual methods. Complete results and output are available upon request.

**Table 2. Equivalent Lateral Forces**

| Floor | $Wxhx^k$ | h (ft) | Cvx | k | Fx (k) | M (ft-k) | Story V (k) |
|---|---|---|---|---|---|---|---|
| 1 | 0 | 0 | 0 | 1.35 | 0 | | |
| 2 | 82269 | 19.67 | 0.0022 | 1.35 | 5.7 | 113 | 2547 |
| 2m | 60054 | 29.17 | 0.0016 | 1.35 | 4.2 | 122 | 2541 |
| 3 | 257827 | 38.67 | 0.0070 | 1.35 | 17.9 | 694 | 2537 |
| 4 | 347146 | 48.17 | 0.0095 | 1.35 | 24.2 | 1164 | 2519 |
| 5 | 442962 | 57.67 | 0.0121 | 1.35 | 30.8 | 1778 | 2495 |
| 6 | 544553 | 67.17 | 0.0149 | 1.35 | 37.9 | 2545 | 2464 |
| 7 | 651375 | 76.67 | 0.0178 | 1.35 | 45.3 | 3475 | 2426 |
| 8 | 762998 | 86.17 | 0.0208 | 1.35 | 53.1 | 4575 | 2381 |
| 9 | 879070 | 95.67 | 0.0240 | 1.35 | 61.2 | 5853 | 2328 |
| 10 | 999302 | 105.17 | 0.0273 | 1.35 | 69.5 | 7314 | 2267 |
| 11 | 1123445 | 114.67 | 0.0307 | 1.35 | 78.2 | 8965 | 2197 |
| 12 | 1251286 | 124.17 | 0.0342 | 1.35 | 87.1 | 10812 | 2119 |
| 13 | 1382640 | 133.67 | 0.0378 | 1.35 | 96.2 | 12862 | 2032 |
| 14 | 1517343 | 143.17 | 0.0415 | 1.35 | 105.6 | 15118 | 1936 |
| 15 | 1655249 | 152.67 | 0.0452 | 1.35 | 115.2 | 17586 | 1830 |
| 16 | 1796229 | 162.17 | 0.0491 | 1.35 | 125.0 | 20271 | 1715 |
| 17 | 1940165 | 171.67 | 0.0530 | 1.35 | 135.0 | 23178 | 1590 |
| 18 | 2086949 | 181.17 | 0.0570 | 1.35 | 145.2 | 26312 | 1455 |
| 19 | 2236484 | 190.67 | 0.0611 | 1.35 | 155.6 | 29676 | 1310 |
| 20 | 2388681 | 200.17 | 0.0653 | 1.35 | 166.2 | 33274 | 1154 |
| 21 | 2543458 | 209.67 | 0.0695 | 1.35 | 177.0 | 37112 | 988 |
| 22 | 2700738 | 219.17 | 0.0738 | 1.35 | 187.9 | 41192 | 811 |
| 23 | 2860450 | 228.67 | 0.0782 | 1.35 | 199.1 | 45519 | 623 |
| 24 | 3900413 | 239.67 | 0.1066 | 1.35 | 271.4 | 65055 | 424 |
| Roof | 2189525 | 256.13 | 0.0598 | 1.35 | 152.4 | 39027 | 152 |
| | Total | | | | | | |
| | 36600611 | | | | | | |

| | |
|---|---|
| Base Shear, $V_b$ = | 2547 |
| Overtruning M, $M_o$ = | 453591 |

**Dynamic Force Analysis:**

The modal response spectrum analysis was completed in accordance with Section 12.9 of ASCE7-05. In order to attain the required 90% of mass participation in orthogonal response, a minimum of 7 modes needed consideration. 12 modes were investigated thus exceeding minimum requirements. Story drifts were concluded in accordance with Section 12.9.2; an additional response spectrum case was created to consider only drift. Scaling design values of dynamic base shear were prepared per Section 12.9.3. The code allowable 85% reduction was applied to ELF:Dynamic base shear ratio to determine forces. Accidental torsion affects were included in the model, thus, torsion amplification was not required. The soil structure interaction reduction, per Section 12.9.7, was not used in evaluating the model.

**Cases/Combinations:**

AUTOEXZ1: Automated ETABS static force analysis. Forces act along the X axis with a positive eccentricity along the Y axis. This creates a moment with a positive sign convention.

AUTOEXZ2: Automated ETABS static force analysis. Forces act along the X axis with a negative eccentricity along the Y axis. This creates a moment with a negative sign convention.

AUTOEYZ1: Automated ETABS static force analysis. Forces act along the Y axis with a positive eccentricity along the X axis. This creates a moment with a positive sign convention.

AUTOEYZ2: Automated ETABS static force analysis. Forces act along the Y axis with a negative eccentricity along the X axis. This creates a moment with a negative sign convention.

XSPECD: Spectral response in the X direction using a 5% damping ratio. This response case is to be used in determining displacements, not forces. The modal combination uses the Complete Quadratic Combination (CQC) method. This combination considers coupling between closely spaced modes caused by modal damping. The directional combination uses the Square Root of the Sum of Squares method (SRSS). This method does not take into account any modal coupling. Amplification of torsion is accounted for by modifying the eccentricity ratio. The input response spectra scale factor was determined by using equation 3, shown below.

$$\text{Scale Factor} = \frac{386 \times R}{C_d} \qquad (\text{Equation 3.})$$

XSPECF: Spectral response in the X direction using a 5% damping ratio. This response case is to be used in determining forces, not displacements. The modal combination uses the Complete Quadratic Combination (CQC) method. This combination considers coupling between closely spaced modes caused by modal damping. The directional combination uses the Square Root of the Sum of Squares method (SRSS). This method does not take into account any modal coupling. Amplification of torsion is accounted for by modifying the eccentricity ratio. The input response spectra scale factor was determined by using equation 4, shown below.

$$\text{Scale Factor} = (\text{S.F.}_{\text{DYN}}) \times \left( \frac{V_{b\,\text{ELF}}}{V_{b\,\text{DYN}}} \right) \times 0.85 \qquad (\text{Equation 4.})$$

YSPECD: Spectral response in the Y direction. This response case uses identical methods and scaling factors as does XSPECD. Displacements in the Y direction are given by this response case.

YSPECF: Spectral response in the Y direction. This response case uses identical methods and scaling factors as does XSPECF. Forces in the Y direction are given by this response case.

**Lateral Forces:**

A sample of the controlling lateral forces is presented herein. Controlling forces were selected from the various spectral response cases. A more detailed summary of forces may be found in Appendix B, complete results are available upon request. A sample of story forces and story shears has been presented from ELF and dynamic analysis. The results from dynamic analysis were used to design the lateral system, per Section 13.9.3 of ASCE7-05.

**Accidental Torsion and Torsion Amplification:**

The torsion amplification factor, $A_x$, has been determined in accordance with Section 12.8.4.3. $A_x$ was determined from assuming an eccentricity ratio of 5%. The eccentricity ratio was then manipulated by $A_x$ for a more accurate ratio of 6.72%. Table 3 shows sample calculations of $A_x$. The controlling spectral response case, AutoEYZ2, was checked at floors 8, 12, 18, and the Roof. All four spectral response cases were checked at floor 12 and the Roof.

**Table 3. Torsion Amplification Factor**

| Level | Load Case | Point 39 | Point 47 | | | |
|---|---|---|---|---|---|---|
| | | $\delta_1$ | $\delta_2$ | $\delta_{average}$ | $\delta_{max}$ | $A_{x\ story}$ |
| Roof | AutoEXZ1 | 0.891 | 0.891 | 0.891 | 0.891 | 0.694 |
| | AutoEXZ2 | 2.954 | -2.953 | 2.954 | 2.954 | 0.695 |
| | AutoEYZ1 | 14.31 | 7.933 | 11.122 | 14.31 | 1.150 |
| | AutoEYZ2 | 7.9350 | 14.3085 | 11.122 | 14.3085 | 1.149 |
| Floor 12 | AutoEXZ1 | -0.3988 | 0.398 | 0.398 | 0.3988 | 0.696 |
| | AutoEXZ2 | 1.322 | -1.3221 | 1.322 | 1.3221 | 0.694 |
| | AutoEYZ1 | 5.0814 | 2.226 | 3.654 | 5.0814 | 1.343 |
| | AutoEYZ2 | 2.228 | 5.0790 | 3.654 | 5.079 | 1.342 |

| $A_{x\ max}$ | ecc. ratio |
|---|---|
| 1.343 | 0.0672 |

**Pier and Spandrel Labeling:**

The pier labeling convention is presented in Figure 4. Two separate labeling systems were created in order to properly analyze flexural and shear limit states. Flexural resistance is a property dependent upon the entire section, yet shear resistance is dependent on properties of the parts (web and flanges). Thus piers 1 and 2 were used to gather flexural output while piers 3-8 were used for shear output. The coupling beams are labeled B2-B7 as shown.

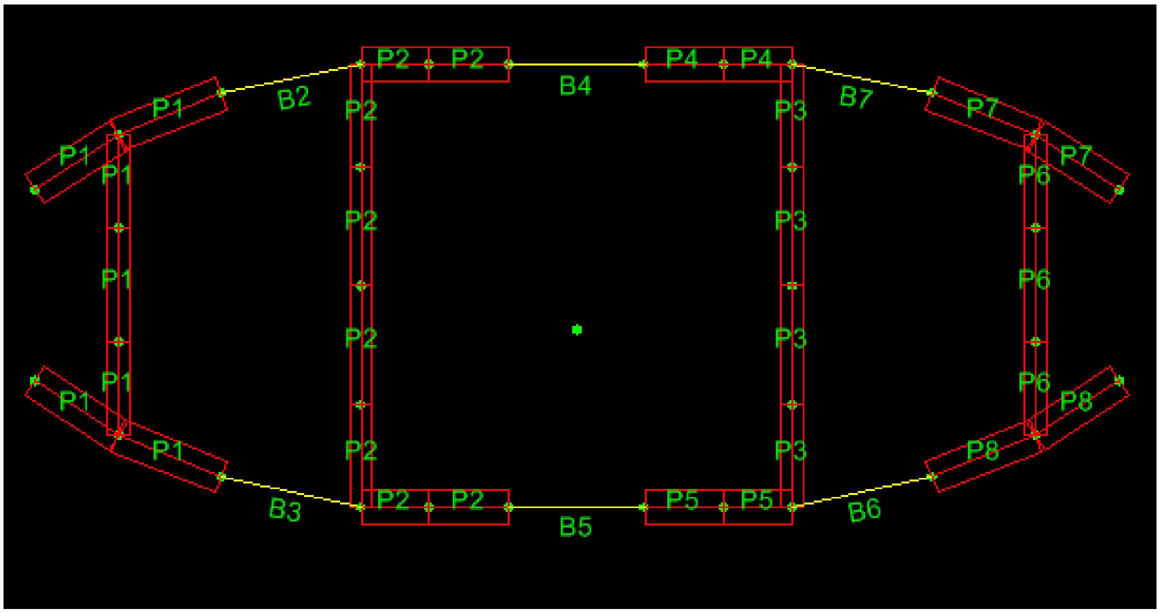


**Figure 4. Pier and spandrel labeling convention**

**Coupling Beams:**

*Preliminary Beam Design:*

Coupling beams are a vital part of the lateral resisting system with a reiterative design process. Proper data must be gathered in order to create a sound final design. In order to do so, it was essential to limit the shear K value to an acceptable one using Equation 3. Equation 3 is derived from Section 21.7.4 of ACI-318.

$$\mathrm{K}_{Vave} = \left[ \frac{V_u / \phi}{\mathrm{bd} \times \sqrt{f'_c}} \times \rho \right] \leq 8 \qquad \text{(Equation 3.)}$$

The beams' K value may be manipulated by decreasing their effective moment of inertia in the ETABS model. This simulates beam cracking, as the beam cracks the shear will decrease. With this in mind, the beams may be designed to resist a lower ultimate shear. The effective inertia has a lower bound limit described in Equation 4 (Paulay & Priestley). For the JWM the beams should not be cracked to less then 1/6 of $I_{gross}$. It should be noted that decreasing the effective inertia will increase the dynamic period, dynamic drift, and shear wall forces.

$$I_{effective} \geq \frac{0.2}{\left(1+3(h/l_n)^2\right)} \times I_{gross} \qquad \text{(Equation 4.)}$$

There must not be excessive drift or shear wall overload in order to justify decreasing the ultimate shear in the coupling beams. For the purposes of this thesis, the drift must be less the 1.0% of the height. This is more conservative than the usual limit of 1.2%. The shear walls must be rechecked for overload using Equation 2. If these criteria cannot be matched than the entire system must be strengthened.

**Table 4. Coupling Beam Ultimate Shear**

| Level | Output Shear (K) | Shear 0.8 $V_{max}$ (K) | Shear $V_{average}$ (K) | Ultimate Shear 0.8 $V_{max}$ (K) | Ultimate Shear $V_{average}$ (K) |
|---|---|---|---|---|---|
| Roof | 90 | **122** | 122 | **122** | 122 |
| 24 | 111 | | | | |
| 23 | 113 | | | | |
| 22 | 121 | | | | |
| 21 | 131 | | | | |
| 20 | 142 | | | | |
| 19 | 152 | | | | |
| 18 | 162 | 161 | **182** | 161 | **182** |
| 17 | 171 | | | | |
| 16 | 179 | | | | |
| 15 | 187 | | | | |
| 14 | 194 | | | | |
| 13 | 201 | | | | |
| 12 | 208 | 186 | **221** | 186 | **213** |
| 11 | 214 | | | | |
| 10 | 220 | | | | |
| 9 | 225 | | | | |
| 8 | 229 | | | | |
| 7 | 232 | | | | |
| 6 | 231 | 186 | **205** | | |
| 5 | 228 | | | | |
| 4 | 220 | | | | |
| 3 | 205 | | | | |
| 2 Mezz | 181 | | | | |
| 2 | 163 | | | | |

*Flexural and Shear Reinforcing Design:*

The reinforcing for the beams was determined after all preliminary design criteria had been met. Beam B5 shown in Figure 4 proved to be the critical beam. Spandrel forces were then taken from the dynamic output. The ultimate shear was chosen to be the greater of $0.8V_{max}$ and $V_{average}$ for the beam group under consideration. Good practice dictates designing beams at every 6 levels. Table 4 shows the determination of ultimate shears for their respective beam groups. As shown in Table 4, it was prudent to design the beams for levels 2-12 for the same ultimate shear.

The reinforcing was then designed using ACI-318 with special provisions from Chapter 21. The Beam Schedule has been reproduced below. A pictorial representation of Beam B1 can be seen in Figure 5. The figure shows a longitudinal and cross section of the coupling beam used on levels 2-12. Due to ease of construction and minimal economic gain from varying designs, only two different designs were used throughout the JWM. Detailed calculations and data to justify all reinforcing and final designs may be found in Appendix B.

**Table 5. Coupling Beam Schedule**

| Level | Beam Type | Size | | Reinforcement | | | Stirrups | | $f'_c$ (PSI) |
|---|---|---|---|---|---|---|---|---|---|
| | | W (IN) | H (IN) | Top Bars Cont | Bottom Bars Cont | Side Bars Cont | Size & Bar No. | Spacing (IN) | |
| Roof | B2 | 24 | 30 | 4 # 10 | 4 # 10 | - | 4 # 5 | 6 | 9000 |
| 24 | B2 | 24 | 30 | 4 # 10 | 4 # 10 | - | 4 # 5 | 6 | 9000 |
| 23 | B2 | 24 | 30 | 4 # 10 | 4 # 10 | - | 4 # 5 | 6 | 9000 |
| 22 | B2 | 24 | 30 | 4 # 10 | 4 # 10 | - | 4 # 5 | 6 | 9000 |
| 21 | B2 | 24 | 30 | 4 # 10 | 4 # 10 | - | 4 # 5 | 6 | 9000 |
| 20 | B2 | 24 | 30 | 4 # 10 | 4 # 10 | - | 4 # 5 | 6 | 9000 |
| 19 | B2 | 24 | 30 | 4 # 10 | 4 # 10 | - | 4 # 5 | 6 | 9000 |
| 18 | B2 | 24 | 30 | 4 # 10 | 4 # 10 | - | 4 # 5 | 6 | 9000 |
| 17 | B2 | 24 | 30 | 4 # 10 | 4 # 10 | - | 4 # 5 | 6 | 9000 |
| 16 | B2 | 24 | 30 | 4 # 10 | 4 # 10 | - | 4 # 5 | 6 | 9000 |
| 15 | B2 | 24 | 30 | 4 # 10 | 4 # 10 | - | 4 # 5 | 6 | 9000 |
| 14 | B2 | 24 | 30 | 4 # 10 | 4 # 10 | - | 4 # 5 | 6 | 9000 |
| 13 | B2 | 24 | 30 | 4 # 10 | 4 # 10 | - | 4 # 5 | 6 | 9000 |
| 12 | B1 | 24 | 30 | 4 # 11 | 4 # 11 | - | 4 # 5 | 6 | 9000 |
| 11 | B1 | 24 | 30 | 4 # 11 | 4 # 11 | - | 4 # 5 | 6 | 9000 |
| 10 | B1 | 24 | 30 | 4 # 11 | 4 # 11 | - | 4 # 5 | 6 | 9000 |
| 9 | B1 | 24 | 30 | 4 # 11 | 4 # 11 | - | 4 # 5 | 6 | 9000 |
| 8 | B1 | 24 | 30 | 4 # 11 | 4 # 11 | - | 4 # 5 | 6 | 9000 |
| 7 | B1 | 24 | 30 | 4 # 11 | 4 # 11 | - | 4 # 5 | 6 | 9000 |
| 6 | B1 | 24 | 30 | 4 # 11 | 4 # 11 | - | 4 # 5 | 6 | 9000 |
| 5 | B1 | 24 | 30 | 4 # 11 | 4 # 11 | - | 4 # 5 | 6 | 9000 |
| 4 | B1 | 24 | 30 | 4 # 11 | 4 # 11 | - | 4 # 5 | 6 | 9000 |
| 3 | B1 | 24 | 30 | 4 # 11 | 4 # 11 | - | 4 # 5 | 6 | 9000 |
| 2 Mezz | B1 | 24 | 30 | 4 # 11 | 4 # 11 | - | 4 # 5 | 6 | 9000 |
| 2 | B1 | 24 | 30 | 4 # 11 | 4 # 11 | - | 4 # 5 | 6 | 9000 |

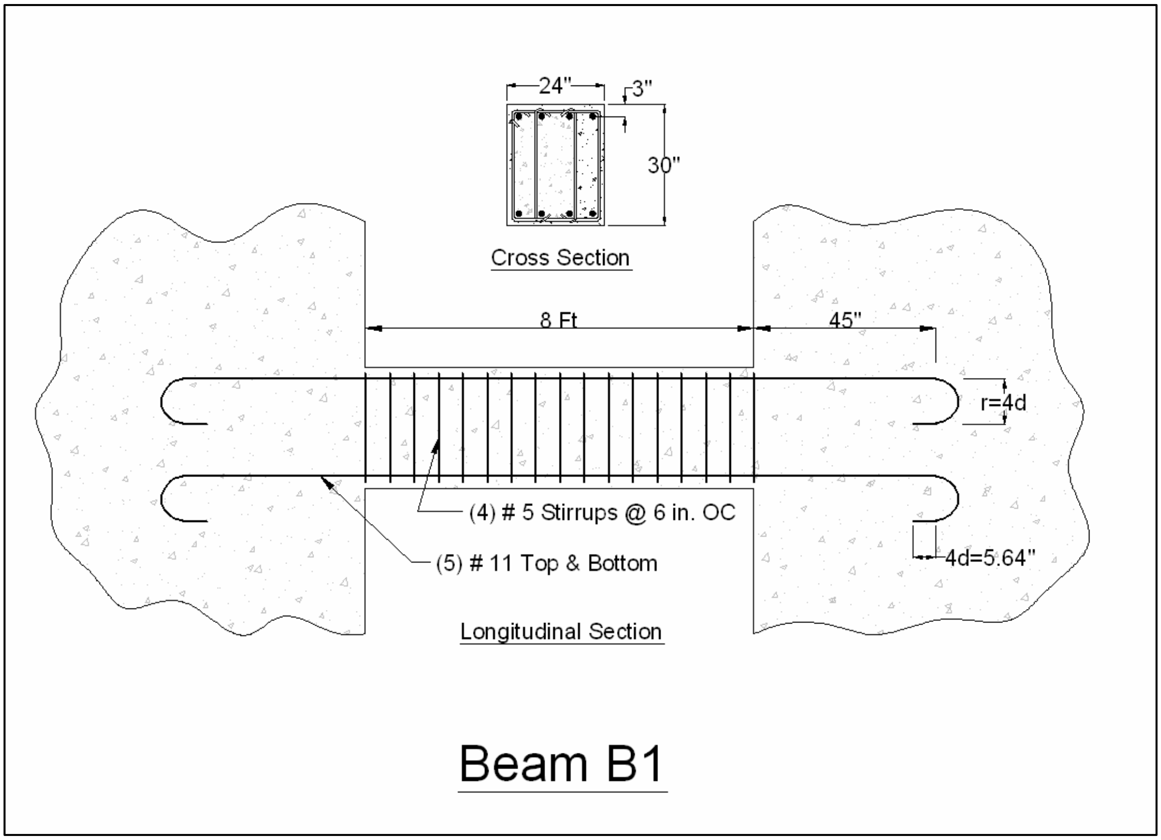


**Figure 5. Beam B1**

**Story Drift:**

The design story drift (Δ) has been determined has been determined using the proper dynamic analysis techniques. After designing piers and spandrels for the necessary capacities allowable story drift was checked. From Table 12.12-1 of ASCE7-05, the allowable story drift shall be $0.020h_{sx}$ , where $h_{sx}$ is the story height below Level x. The drifts were compared to those allowable and if necessary, the system was revised to further limit story drifts. Results for story drift are presented in Tables 5 and 6, complete results are available upon request. The drifts for the JWM were limited to less than 1.0% in order to meet the peer reviewer's approval more easily.

| Table 5. Drift in X | | |
|---|---|---|
| Story | Displacement (in) | Drift (%) |
| Roof | 16.57 | 0.425 |
| Story 24 | 15.81 | 0.455 |
| Story 23 | 15.28 | 0.483 |
| Story 22 | 14.80 | 0.513 |
| Story 21 | 14.28 | 0.542 |
| Story 20 | 13.74 | 0.270 |
| Story 19 | 13.17 | 0.594 |
| Story 18 | 12.58 | 0.615 |
| Story 17 | 11.96 | 0.633 |
| Story 16 | 11.32 | 0.647 |
| Story 15 | 10.66 | 0.660 |
| Story 14 | 9.97 | 0.670 |
| Story 13 | 9.27 | 0.678 |
| Story 12 | 8.55 | 0.685 |
| Story 11 | 7.82 | 0.690 |
| Story 10 | 7.07 | 0.694 |
| Story 9 | 6.31 | 0.695 |
| Story 8 | 5.54 | 0.694 |
| Story 7 | 4.76 | 0.688 |
| Story 6 | 3.99 | 0.676 |
| Story 5 | 3.22 | 0.653 |
| Story 4 | 2.48 | 0.617 |
| Story 3 | 1.78 | 0.561 |
| 2nd Mez | 1.14 | 0.477 |
| 2nd | 0.60 | 0.254 |

| Table 6. Drift in Y | | |
|---|---|---|
| Story | Displacement (in) | Drift (%) |
| Roof | 21.37 | 0.970 |
| Story 24 | 19.49 | 0.971 |
| Story 23 | 18.24 | 0.970 |
| Story 22 | 17.16 | 0.967 |
| Story 21 | 16.09 | 0.961 |
| Story 20 | 15.02 | 0.953 |
| Story 19 | 13.97 | 0.941 |
| Story 18 | 12.90 | 0.927 |
| Story 17 | 11.9 | 0.909 |
| Story 16 | 10.9 | 0.889 |
| Story 15 | 9.91 | 0.865 |
| Story 14 | 8.95 | 0.839 |
| Story 13 | 8.02 | 0.810 |
| Story 12 | 7.12 | 0.779 |
| Story 11 | 6.25 | 0.744 |
| Story 10 | 5.41 | 0.707 |
| Story 9 | 4.62 | 0.666 |
| Story 8 | 3.87 | 0.621 |
| Story 7 | 3.16 | 0.573 |
| Story 6 | 2.52 | 0.520 |
| Story 5 | 1.93 | 0.462 |
| Story 4 | 1.41 | 0.399 |
| Story 3 | 0.95 | 0.330 |
| 2nd Mez | 0.57 | 0.254 |
| 2nd | 0.28 | 0.121 |

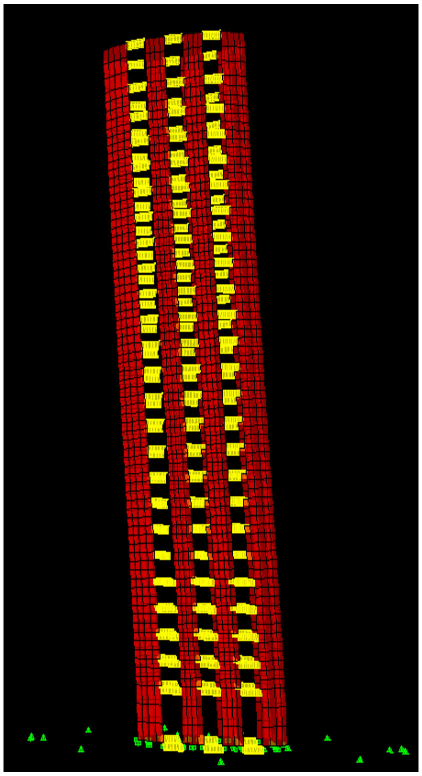

**Figure 6. Drift in the X**

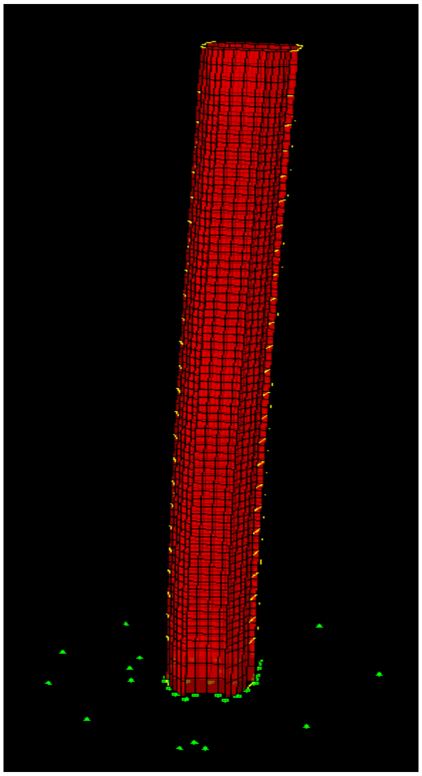

**Figure 7. Drift in the Y**

## Shear Walls:

*Direct Shear:*

Direct shear capacity was verified using Equation 3. However, in the shear walls the K value in Equation 3 is limited to less than or equal to 4. The code allows a K value of 8 but the pier review will be easily passed if the shear stresses are kept to half those allowed by code. The forces were gathered from ETABS output and checked for the worst case load, located at the ground floor. Only piers 3, 4, 6, and 7 were checked due to the inherent symmetry. A reproduction of the calculations may bee seen below. Pier 6 yields a K value of 4.11; this is much less than the code allowable value of 8 and therefore acceptable.

$$\text{Pier } 3: \quad 1100\,\text{k} = \phi \times 3.59\sqrt{f'_c} \times A_v, \quad \text{where } A_{v3} = 37.33\,\text{ft}^2$$

$$\text{Pier } 4 \,\&\, 5: \quad 343\,\text{k} = \phi \times 2.24\sqrt{f'_c} \times A_v, \quad \text{where } A_{v4/5} = 18.67\,\text{ft}^2$$

$$\text{Pier } 6: \quad 876\,\text{k} = \phi \times 4.11\sqrt{f'_c} \times A_v, \quad \text{where } A_{v6} = 26.0\,\text{ft}^2$$

$$\text{Pier } 7 \,\&\, 8: \quad 1100\,\text{k} = \phi \times 3.59\sqrt{f'_c} \times A_v, \quad \text{where } A_{v7/8} = {}^*19.5\,\text{ft}^2$$

$$^*A_{v7/8} = 9.77\text{ft}^2 \times \cos(33°) + 12.2\text{ft}^2 \times \cos(22°)$$

*Load Combinations for Flexure:*

A total of eight combinations were used to generate the worst case loading condition for the shear walls. The formulation of those combinations was from the controlling ASCE7 combination (the formulation is shown below). When the location on the P-M interaction curve is below the balance point, as is the case, any reduction in compressive force will reduce the flexural capacity. Thus, worst case loading will occur when the vertical earthquake effects reduce the compressive force on the shear wall. These load combinations were investigated using PCA Column.

$$\text{ASCE7}: 0.9D + 0.5L \pm 1.0E + 1.0E_v$$

$$\text{where } E_v = 0.2S_{DS}D \text{ , } S_{DS} = 0.97 \text{ , and } L = 0$$

$$\text{ASCE7 then becomes:}$$

$$(1)\ 0.7D + 1.0E_x + 0.3E_y$$

$$(2)\ 0.7D + 1.0E_x - 0.3E_y$$

$$(3)\ 0.7D - 1.0E_x + 0.3E_y$$

$$(4)\ 0.7D - 1.0E_x - 0.3E_y$$

$$(5)\ 0.7D + 0.3E_x + 1.0E_y$$

$$(6)\ 0.7D - 0.3E_x + 1.0E_y$$

$$(7)\ 0.7D + 0.3E_x - 1.0E_y$$

$$(8)\ 0.7D - 0.3E_x - 1.0E_y$$

$$\text{Where } E_x = M_x \text{ due to Earthquake in Y}$$

$$E_y = M_y \text{ due to Earthquake in X}$$

$$D = P_{SELF\ WT} + P_{GRAVITY}$$

*PCA Column Analysis and Reinforcement Layout:*

The reinforcing for piers 1 and 2 at the ground floor were designed utilizing PCA Column computer analysis. PCA is a powerful tool that can accurately analyze several load combinations and create an interaction diagrams based on rebar size and layout.

The results from PCA were taken and a reinforcing schedule was created from the results. The vertical and horizontal reinforcing schedules are reproduced in Table 7 and Table 8, respectively.

**Table 7. Pier Vertical Reinforcing Schedule**

| Pier | Rebar # | Rows | Spacing (in) | As (sq. in.) | ρ, per Pier (%) | ρ, per section (%) |
|---|---|---|---|---|---|---|
| 3 | 9 | 1 | 16 | 18 | 0.26 | 0.94 |
| 4 | 9 | 2 | 6 | 38 | 1.41 | |
| 5 | 9 | 2 | 6 | 38 | 1.41 | |
| 6 | 9 | 1 | 11 | 15 | 0.40 | 1.16 |
| 7 outer | 9 | 3 | 3.5 | 30 | 1.13 | |
| 7 inner | 9 | 2 | 6.5 | 24 | 1.70 | |
| 8 outer | 9 | 3 | 3.5 | 30 | 1.13 | |
| 8 inner | 9 | 2 | 6.5 | 24 | 1.70 | |

| Table 8. Pier Horizontal Reinforcing Schedule | | | | | |
|---|---|---|---|---|---|
| Pier | Rebar # | Rows | Spacing (in) | As ($in^2$/ft) | ρ, per Pier (%) |
| 3 | 4 | 2 | 10 | 0.48 | 0.25 |
| 4 | 5 | 2 | 10 | 0.74 | 0.26 |
| 5 | 5 | 2 | 10 | 0.74 | 0.26 |
| 6 | 4 | 2 | 10 | 0.48 | 0.25 |
| 7 | 5 | 2 | 10 | 0.74 | 0.26 |
| 8 | 5 | 2 | 10 | 0.74 | 0.26 |

It was found that the highest stresses occurred toward the edges of the flanges and required the most reinforcing. This is expected when a combination of moments in X and Y axis are placed on the section. Figure 8 shows one of the worst case loading combinations on pier 1. Blue indicates low tension stresses while red indicates high tension stresses due to the applied moment. If the moments were reversed, the opposite flange would become the critical area and require more reinforcing than the rest of the section.

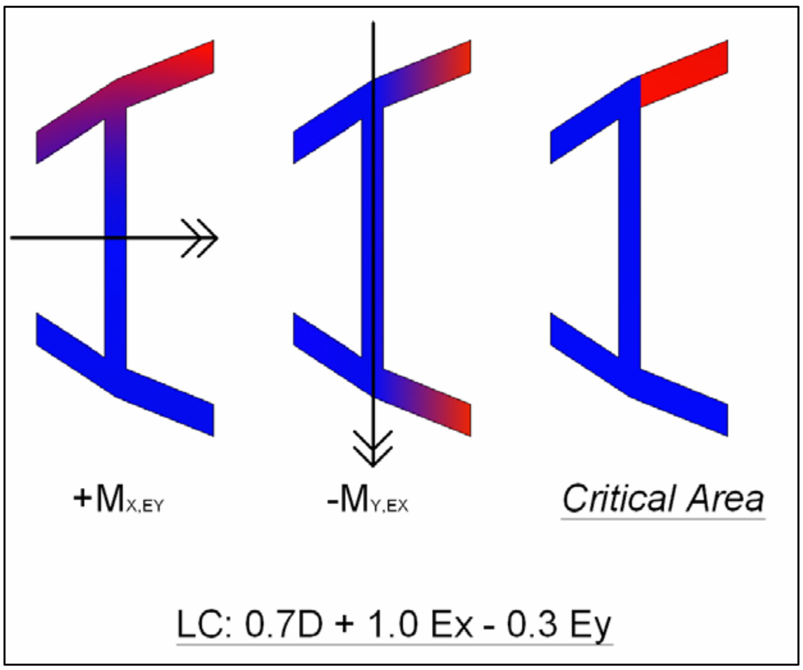


**Figure 8. Tension stress concentrations due to applied moments**

**Conclusion:**

The nonlinear response of the JWM matches the desired response. Plastic hinging occurs only in the coupling beams. They will act as the primary seismic force dissipation mechanism. The detailing of the designated nonlinear elements (coupling beams) provides sufficient ductility capacity. The beams will deform plastically and significantly depredate seismic forces. Shear walls have been designed for elastic response and will experience only flexural yielding. The walls maintain their lateral force resistance through large displacements and deform in a manner that distributes displacements evenly over the entire height.

The peer reviewer will find the seismic performance of the JWM exceeds code-prescriptive design requirements. The inherent uncertainties of the assumptions to define seismic performance-deformation capacity, nonlinear demands, etc- are offset by the stern constraints under which the JWM was designed. The JWM surpasses an equivalent level of performance to the code and would smoothly pass peer review.

# Depth: Post Tension Floor System

**Introduction:**

A flat plate post tension (PT) floor system, $f'_c$= 4000psi, was chosen to replace the current flat plate system with wall columns based on two primary reasons. First, California construction techniques and typical design practice dictate that such a structure would be designed using a PT system. Second, in order to meet industry standards of $\frac{\text{Vertical Wall Area}}{\text{Floor Area}} \times 100\% \leq 3\%$ the perimeter blade-columns were removed in favor of circular columns. Spans were increased to twice those of the original flat plate system. The PT system spans approximately 35 ft from column to column and 35 ft from column to core. Preliminary design estimated a 9.5 in. slab based on the standard span to thickness ratio $\left(\frac{L}{45}\right)$ for flat plate PT systems.

The goal of this section is to prove that a 9.5 inch PT slab is a viable solution for a floor system in the JWM. The lateral system has been designed using this assumption and must now be verified. If a 9.5 in. slab has been used to determine seismic forces on the lateral system. If the slab requires greater thickness, then the lateral system will experience higher forces and will need to be redesigned. The column layout for the PT system is shown below in Figure 9.

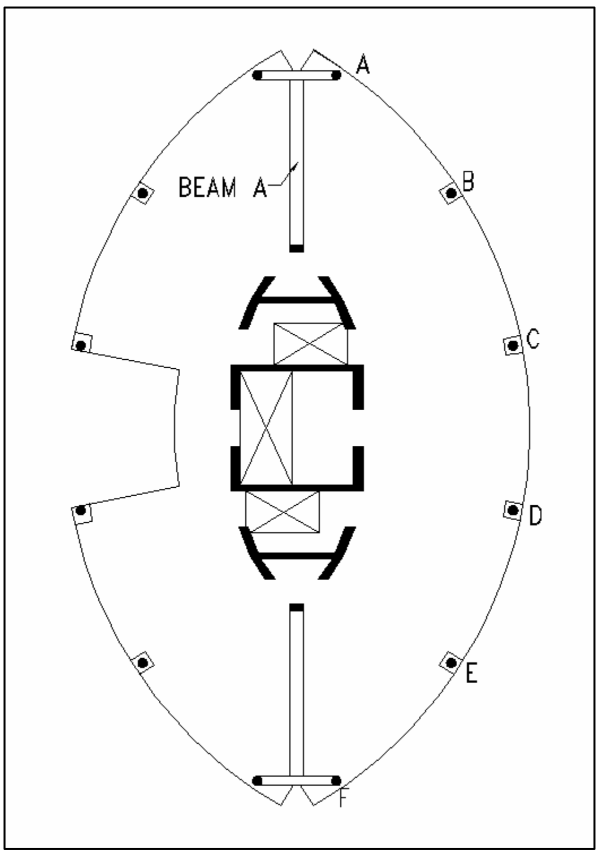


**Figure 9. Post tension floor plan**

## Analysis Methods and Assumptions:

Manual analysis with the assistance of Ram Concept computer analysis has been used to verify the integrity of the PT system. Average concrete stresses and midspan deflections will be investigated for the largest span, 34 ft. 8 in. It is in this span (span CD) that the largest stresses will occur from the PT tendons. The end span (span AB) will experience the greatest deflections and will be checked for excessive deflections. Long term deflection will be checked for the beam A. If possible, beam A will be limited to the existing partition width of 11 in. and needs to be limited to a maximum depth of 30 in. to allow for patron passage in the hallway. The labeling for spans, beams, and columns can be seen in Figure 9. Average stresses will be compared to the limits for Class U-unbonded tendon system. The balance load will be 85% of the dead load including self weight. The ACI moment coefficients are to be used to determine all positive and negative moments. With five interior spans any form of moment distribution is not necessary, ACI coefficients will yield acceptable values.

## Design Loads:

The loads for the JWM are in accordance with the original structure and ASCE7. The design loads and ASCE7 counterparts have been reproduced in Table 9. These loads will be used to verify the typical floor PT system.

**Table 9. PT Floor Loads**

| | Design Loads (psf) | ASCE 7 (psf) |
|---|---|---|
| Dead | 20 | 20 |
| Live | 40 | 40 |

## Average Stresses:

Average stresses were calculated by hand for span CD. This span has the largest drape profile and will cause the most stress on the concrete. The stresses were based on 85% dead load balance, ½ Ø 270 ksi tendons with 30 kip in losses, and 4 tendons grouped at every 4 ft. on center. The stresses were found to be well below those allowed by Chapter 18 of ACI. The detailed hand calculations of stresses have been included in Appendix C.

## Punching Shear:

Punching shear is a probable failure mode considering large 35 ft exterior spans being supported by 24 in. diameter columns. Ram Concept was used to check the need for drop panels around the circular columns. The drop panels were not designed by hand because the presence of drop columns is not viewed a system failure.  The results have been presented in Table xx.

**Table 10. Punching Shear Results**

| Column | Drop Panel | Approx Panel Size (in) | Shear Stud Reinforcing |
|---|---|---|---|
| 1 | No | - | - |
| 2 | Yes | 42x42 | No |
| 3 | Yes | 42x42 | No |
| 4 | Yes | 42x42 | No |
| 5 | Yes | 42x42 | No |
| 6 | No | - | - |
| 7 | No | - | - |
| 8 | Yes | 42x42 | No |
| 9 | Yes | 45x45 | Yes |
| 10 | Yes | 45x45 | Yes |
| 11 | Yes | 42x42 | No |
| 12 | No | - | - |

## Tendon Layout:

The Proposed latitudinal and longitudinal tendon layout is presented in Figures 10 and 11, respectively. A typical 1.25 in. of cover for PT tendons were used throughout all calculations. The tendon drape can be seen in Figure 12. The larger tendons will span from the core to the column while the smaller tendons will span in a radial pattern and be spaced at every 4 ft (in groups of 4).

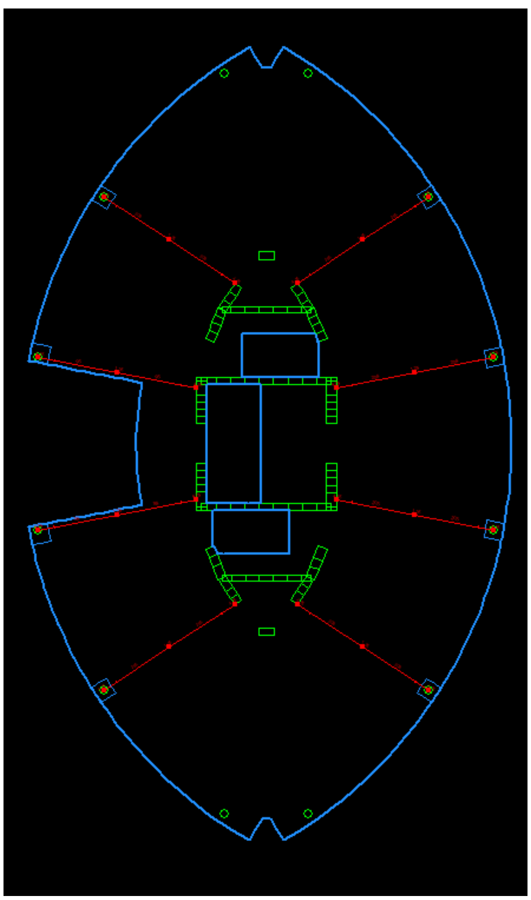

**Figure 10. Latitudinal tendon layout**

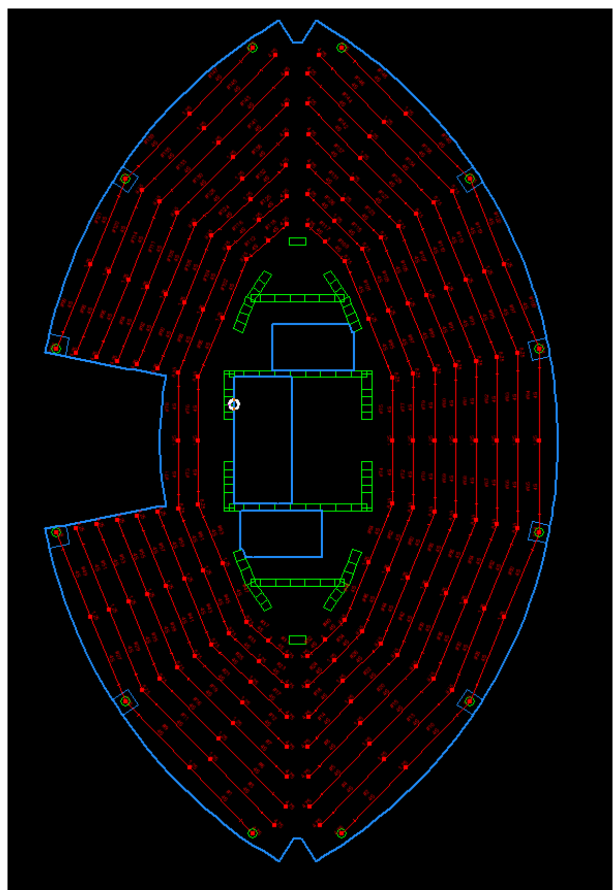

**Figure 11. Longitudinal tendon layout**

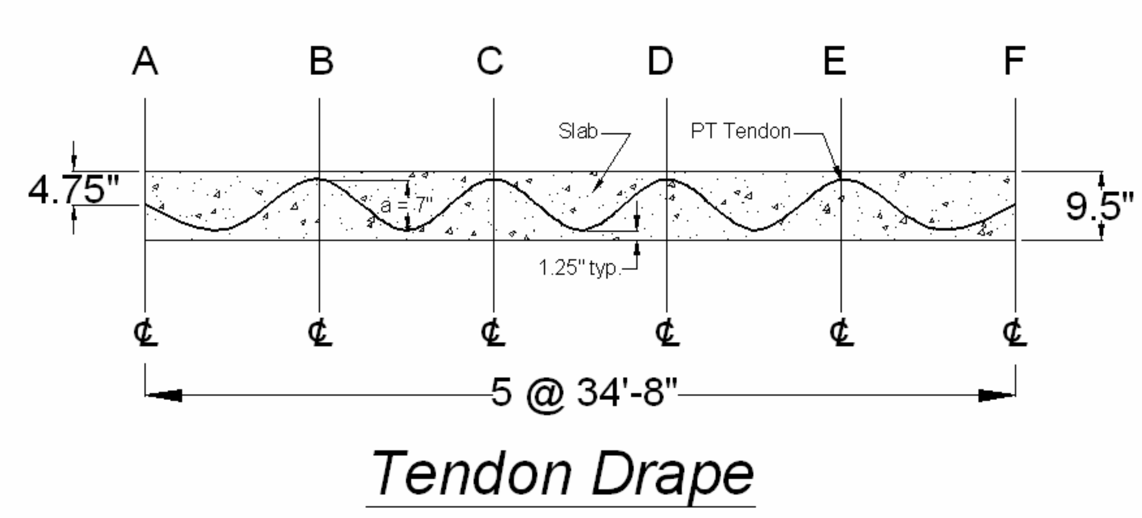


**Figure 12. Post tension tendon drape**

## Reinforcement:

The reinforcing for span CD was designed for a 1 ft. strip. The reinforcing was designed in accordance with Chapter 18 of ACI. Sufficient strength can be achieved using #5 @ 24 in. on center to resist the positive moment and #5 @ 20 in. on center to resist negative moments. Detailed hand calculations can be viewed in Appendix C.

## Deflections:

The results for deflections have been presented in Table 11. The deflection was checked for spans AB and CD. Long term deflection was checked for beam A. Fixed end supports were assumed in the long term deflection calculations. Using Equation 5 it was found that the L/360 deflections limit could not be met by a beam with a mere 11 in. width. Detailed calculations for long term deflection and span deflections may be found in Appendix C.

**Table 11. Span Deflections**

| Span | Deflection (in) | Deflection Equivalent |
|---|---|---|
| AB | 0.268 | L/1150 |
| CD | 0.361 | L/1150 |

$$\Delta_{long\ term} = \frac{5}{384} \times \frac{w_{sus}\ell^4}{E_c I_{effective}} \qquad \text{(Equation 5)}$$

$$\text{where } I_{effective} = \left[\left(\frac{M_{cr}}{M_a}\right)^3 \times I_g\right] + \left[\left(1 - \left(\frac{M_{cr}}{M_a}\right)^3\right) \times I_{cr}\right]$$

**Conclusion:**

It is evident that a 9.5 in. PT slab is sufficient for the JWM redesign. The average stresses in the slab are well within acceptable the ACI required limits. The midspan deflections are satisfactory for a typical interior and exterior bay. However, the long term deflection requirement cannot be met with a beam limited to 11 in. wide. This is not an event that would determine the entire system a “failure”. There are two simple solutions to limit the deflection of beam A. First, a wider beam with multiple layers of reinforcement will increase the cracked moment of inertia and effective inertia. This adjustment would likely decrease the deflection to acceptable amounts. Second, an addition of a column at half span would decrease deflections and likely allow a beam that may fit within existing partitions.

With these results in mind, it is likely that a thinner slab could be used if a PT system was to become the floor system of choice. The stresses may be further reduced by the addition of edge beams if a thinner slab were desired. The assumed concrete strength of 4000 psi may also be increased to increase the allowable stress limits.

The formulation of seismic forces based upon the assumed weight of a 9.5 in. PT slab has been proven accurate. A thinner, lighter slab would likely prove acceptable. Therefore the lateral forces used in design have been shown to be conservative.