SEISMIC EVALUATION OF CHABOT TOWER

FINAL REPORT

Prepared for

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1. EXECUTIVE SUMMARY

This report summarizes the findings and recommendations of a seismic performance evaluation of Chabot Tower for the maximum design earthquake (MDE) and the maximum credible earthquake (MCE) ground motions. The MDE was chosen by the East Bay Municipal District as a ground motion having a 10 percent probability of exceedance in 50 years (a return period of 475 years). The MCE is estimated as a moment magnitude M_w 71/4 event on the nearby Hayward Fault 0.5 km west of the tower. The seismic evaluation consisted of simplified code calculations and three-dimensional (3D) linear-elastic finite-element analyses. The material properties for the analyses were established using published data and observed physical conditions of the materials.

Chabot Tower is a multi-level entry portal structure constructed against the Chabot Dam left abutment rock, on the west shore of Lake Chabot. Inflow from the tower is passed to Tunnel No. 2 through an 8-foot-diameter brick-lined outlet shaft behind the tower. The tower is approximately 23 feet square in plan and 48 feet tall. It is made primarily of plain stone masonry and cast against the rock along its back side and base with no anchors. At the top, the tower is capped with a 13-foot high reinforced concrete pavilion. The pavilion roof slab is supported on reinforced concrete perimeter beams, which in turn are supported by 18 hollow circular concrete columns. The pavilion is connected to the abutment rock through a concrete slab bridge at the roof level.

The tower was modeled using 3D solid elements to represent the masonry and a portion of the foundation and abutment rock that support the tower. The pavilion was modeled using frame elements for columns, shell elements for the roof slab, and 3D solid elements for beam girders and the slab bridge. The inertia forces of the surrounding and inside water due to earthquake shaking were represented by added hydrodynamic mass coefficients. The material properties for the concrete and masonry were assessed and established in accordance with FEMA 356 and the Uniform Building Code as well as the observed physical conditions of the materials. The elastic properties of the abutment rock were estimated using measured seismic velocities in the dam foundation and consideration of the rock condition and the level of ground shaking at the site. The tower was analyzed for the gravity and hydrostatic loads plus the effects of seismic loads. The evaluation for seismic loads was based on the 3D response-spectrum mode-superposition method using three components of the earthquake response spectra as the seismic input. The seismic performance of the tower was then assessed by comparing computed seismic force demands with section capacities of the reinforced-concrete pavilion, and seismic stress demands with tensile and shear strengths of the plain stone masonry. Such comparison tends to show the severity of damage and possible modes of failure from which the acceptability of the performance can be assessed.

The results indicate that the reinforced-concrete pavilion will suffer severe damage and probably collapse in the event of a major earthquake with ground motions at the level of the MDE. This finding is supported by the demand-capacity ratios of the pavilion columns that reach as high as 6.21 for moment and 2.7 for shear. The results also show that the masonry tower will experience extensive tensile and shear cracking that could lead to formation of disjointed blocks and complete separation of the tower from the abutment rock, as indicated by the tensile and shear stress demand-capacity ratios of 9 and 2.1, respectively. Although the tower may not collapse, formation of disjointed blocks and separation from the abutment rock could diminish its load resisting capabilities. The valve shafts or shaft supports could be damaged causing accidental blockage of the sluice valves, thus blocking release of water from the reservoir. The situation will be even worse for a postulated MCE event on the nearby Hayward Fault which is capable of producing 40% larger seismic forces than the MDE.

The estimated abutment stresses indicate that the 8-foot-diameter outlet shaft behind the tower would survive the MDE and MCE shaking, provided that the outlet is inspected to ensure that the brick liner is in good condition and that the gate operating steel gear has not corroded. However, a deteriorated brick liner could suffer damage in a major earthquake and the resulting earthquake debris could potentially block the outlet works at the tunnel entrance.

Based on the results of this study, the tower will respond in brittle mode, thus no further structural analysis or material testing is recommended. This is because nonlinear behavior is not permitted in brittle mode and the materials, even if tested, will not result in strengths as high as those demanded by the earthquake. However, depending on the operational needs and potential impacts on the release of water from the reservoir, additional efforts should be focused on retrofitting the structure to ensure it will remain functional in the event of a major earthquake. Strengthening the pavilion structure appears to be an expensive undertaking. Therefore, we recommend demolishing and removing the pavilion to eliminate the possibly of the pavilion collapsing on top of the masonry tower, especially since it offers no significant structural function. If desired a light steel frame structure may be designed as a replacement. With the pavilion removed, two options are proposed: 1) do not fix the masonry tower but remove the sluice gates (or the valve shafts) so that accidental blockage of the sluice valves will not occur, or 2) strengthen the masonry tower to stabilize and maintain its structural integrity by anchoring the tower into the foundation and abutment rock using external anchors. In Option 1, the outflow from the reservoir will be controlled by the sluice gate in the outlet shaft. However, the brick liner and the gate operating steel gear should be inspected and if necessary repaired for both options to preclude accidental blockage of the outlet shaft at the entrance to the tunnel. This may be accomplished by connecting the 30” lower inlet pipe to the tunnel.

2. INTRODUCTION

2.1 GENERAL

This report presents the results of a three-dimensional linear-elastic finite-element analysis conducted to assess the seismic performance of Chabot Tower. The study was performed for the East Bay Municipal Utility District (District) under a contract to the URS Corporation. This report was prepared by Yusof Ghanaat of Quest Structures and reviewed by Lelio Mejia, the URS Project Manager for this work. This report also includes work performed in support of this evaluation by Tennebaum-Manheim Engineers (TME) and OLMM Consulting Engineers, as Attachments I and II, respectively.

Built in 1923, the tower was designed before modern seismic resistance codes and methods were in use. In 1991, the tower was evaluated by preliminary hand calculations using the 1988 UBC standards and found to be at high risk from an earthquake on the nearby Hayward Fault. The current study was therefore undertaken to assess the earthquake performance of the tower more thoroughly with the most recent code requirements and then proceed with a more detailed three-dimensional finite-element analysis.

The 48 foot high stone masonry tower is a multi-level entry port for the 8-foot-diameter brick-lined outlet shaft behind the tower, which is completely surrounded by rock and soil. A reinforced concrete pavilion structure 13 feet tall is built on top of the masonry tower for operation of the lower and mid level sluice gates. The outlet works feed a 36-in pipe within a tunnel (Tunnel No. 2) that could be used as an emergency water supply from Chabot Reservoir. Chabot Reservoir is normally used for recreation and has a main spillway separate from the outlet works plus another tunnel (Tunnel No. 3) for an auxiliary spillway. The mid-level and lower inlet sluice valves are currently kept open. The outlet flow is regulated using a 36-in sluice valve located in the 8-foot-diameter outlet shaft at the entrance to Tunnel No. 2. The reservoir can be drained in about 36 days with the 30-in diameter lower inlet pipe that feeds the outlet shaft.

2.2 DESCRIPTION OF TOWER

Chabot Tower is a 48-foot-high multi-level entry portal structure constructed against the Chabot Dam left abutment rock, on the west shore of Lake Chabot in San Leandro, California. Figure 2-1 shows a photograph of the tower taken on September 12, 1924 prior to impoundment of the lake. Figures 2-2 and 2-3 show elevation and plan views with section elevation depicting multi-level flow entry. Inflow to the tower is provided by the 8-foot opening in the upstream face. The water is then passed to the outlet shaft by a 20-inch-square sluiceway at invert El. 214.3 ft, and also through an 8x10 ft discharge tunnel with invert El. 224.5. The discharge tunnel is partially blocked by stop timbers except for an opening in the center of the tunnel (Figure 2-2). A third inlet to the outlet shaft is provided by a 30-inch steel pipe buried at the bottom of the tower. In 1991, a short section was added to the 30-in pipe to prevent the lower inlet from being blocked from falling material. The sluice valves for both the 30-in and 20-in inlets are maintained in open position. The inflow from tower first enters the outlet shaft, and then passes to

Tunnel No. 2 at the bottom of the shaft through a 36-inch pipeline located inside the tunnel and regulated by a 36-inch sluice valve. The flow out of the reservoir can be controlled by either the 36-in sluice valve located at the entrance to the tunnel or by the two 36-in butterfly valves downstream near the blow-off structure.

The tower is approximately 23 feet square in plan but it is slightly narrower on the upstream or north side (Figure 2-3). It is made primarily of plain stone masonry, except for the top part, which includes layers of dressed stone, bricks, and concrete. The slightly embedded tower is simply cast against the abutment rock along its back side and the base with no anchors. Any tension and shear resistance at the contact surfaces are therefore limited to tensile and shear strengths of the mortar. At the top the tower is capped with a 13-foot-high reinforced concrete pavilion which houses the lower and mid-level inlet sluice gate operators. The pavilion roof slab is reinforced concrete and is supported on reinforced-concrete perimeter beams. These beams in turn are supported on 18 hollow circular reinforced-concrete columns with outside and inside diameters of 15 and 11 inches, respectively. The columns and slotted reinforced-concrete floor (pre-cast concrete floor) rest on about 4.5 feet of concrete above the masonry tower. The total height of the tower including the pavilion is around 53 ft. The ground level is at an elevation of 203 ft and the spillway is at an elevation of 227.25 ft.

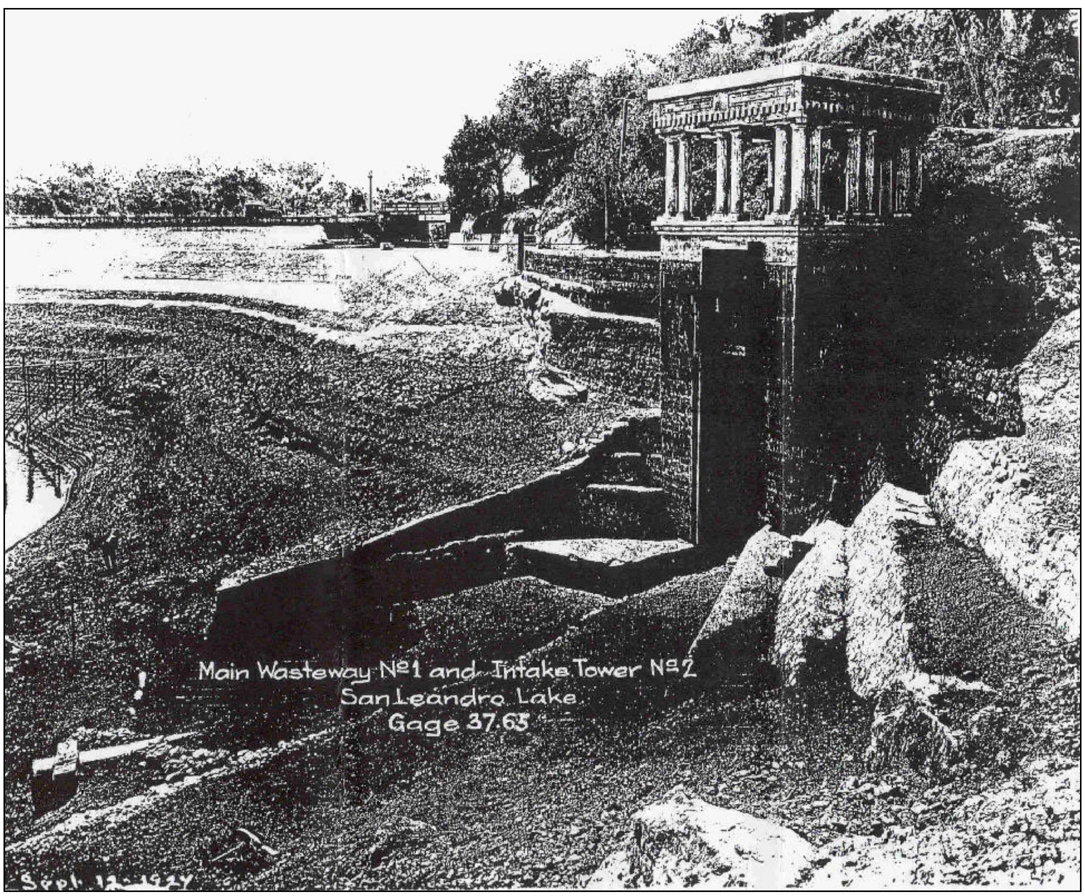
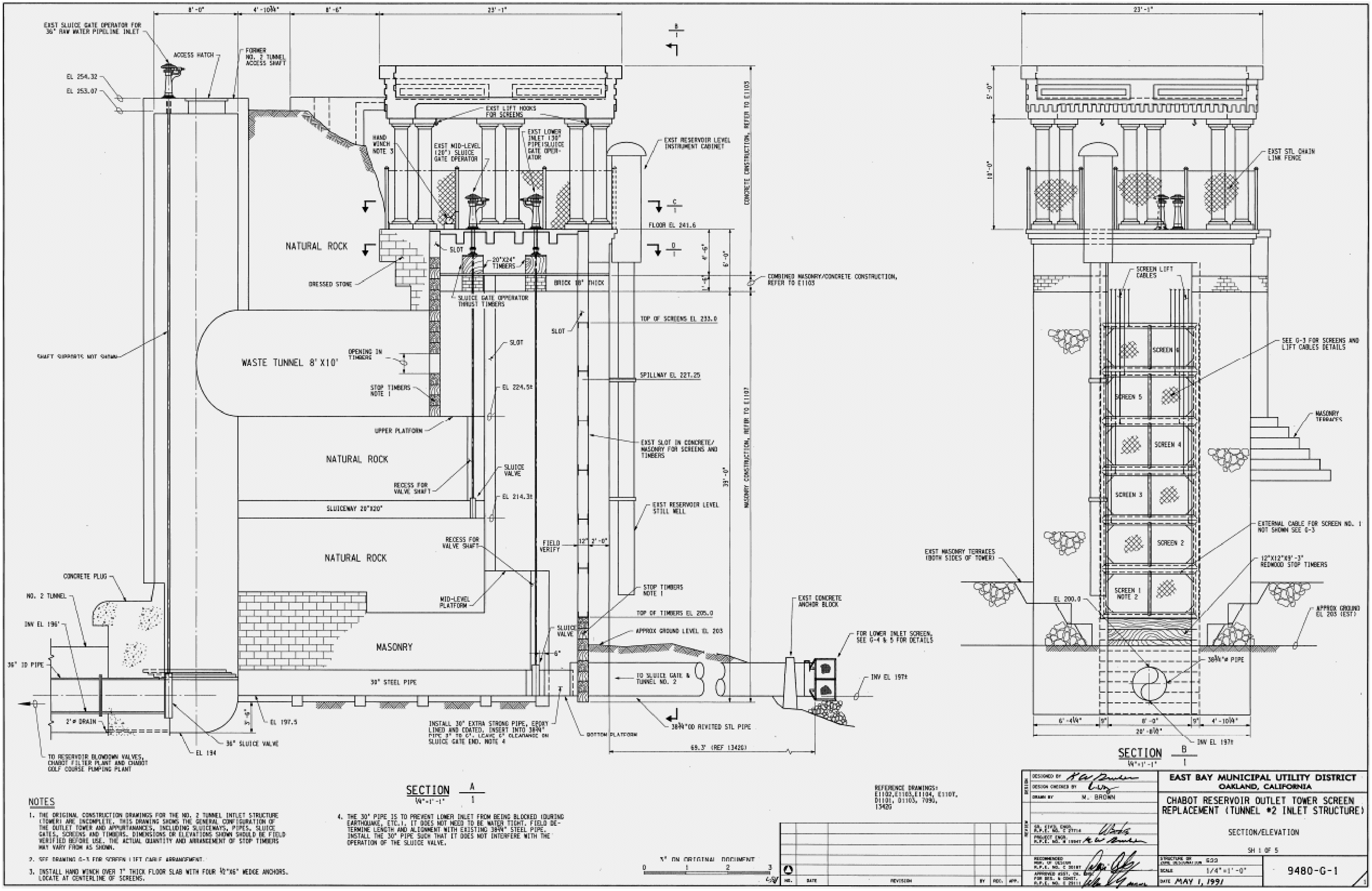


Figure 2-1: Construction photo taken on September 12, 1924



SECTION A/1
1/4"=1'-1"
SECTION B/1
1/4"=1'-1"
NATURAL ROCK
WASTE TUNNEL 8' X10'
MASONRY
30" STEEL PIPE
SLUICEWAY 20"X20"
NOTES
1. THE ORIGINAL CONSTRUCTION DRAWINGS FOR THE NO. 2 TUNNEL INTLET STRUCTURE (TOWER) ARE INCOMPLETE. THIS DRAWING SHOWS THE GENERAL CONFIGURATION OF THE OUTLET TOWER AND APPURTANANCES, INCLUDING SLUICEWAYS, PIPES, SLUICE GATES, SCREENS AND TIMBERS. DIMENSIONS OR ELEVATIONS SHOWN SHOULD BE FIELD VERIFIED BEFORE USE. THE ACTUAL QUANTITY AND ARRANGEMENT OF STOP TIMBERS MAY VARY FROM AS SHOWN.
2. SEE DRAWING G-3 FOR SCREEN LIFT CABLE ARRANGEMENT.
3. INSTALL HAND WINCH OVER 7" THICK FLOOR SLAB WITH FOUR 1/2"X6" WEDGE ANCHORS. LOCATE AT CENTERLINE OF SCREENS.
4. THE 30" PIPE IS TO PREVENT LOWER INLET FROM BEING BLOCKED (DURING EARTHQUAKE, ETC.). IT DOES NOT NEED TO BE WATER TIGHT. FIELD DETERMINE LENGTH AND ALIGNMENT WITH EXISTING 38¾" STEEL PIPE. INSTALL THE 30" PIPE SUCH THAT IT DOES NOT INTERFERE WITH THE OPERATION OF THE SLUICE VALVE.
REFERENCE DRAWINGS: E1102, E1103, E1104, E1107, D1101, D1103, 7090, 13420
EAST BAY MUNICIPAL UTILITY DISTRICT
OAKLAND, CALIFORNIA
CHABOT RESERVOIR OUTLET TOWER SCREEN REPLACEMENT (TUNNEL #2 INLET STRUCTURE)
SECTION/ELEVATION
SH 1 OF 5
DRAWN BY M. BROWN
SCALE 1/4"=1'-0"
DATE MAY 1, 1991
9480-G-1

Figure 2-2: Section/elevation view showing multi-level entry portal structure and 8-foot brick-lined outlet shaft

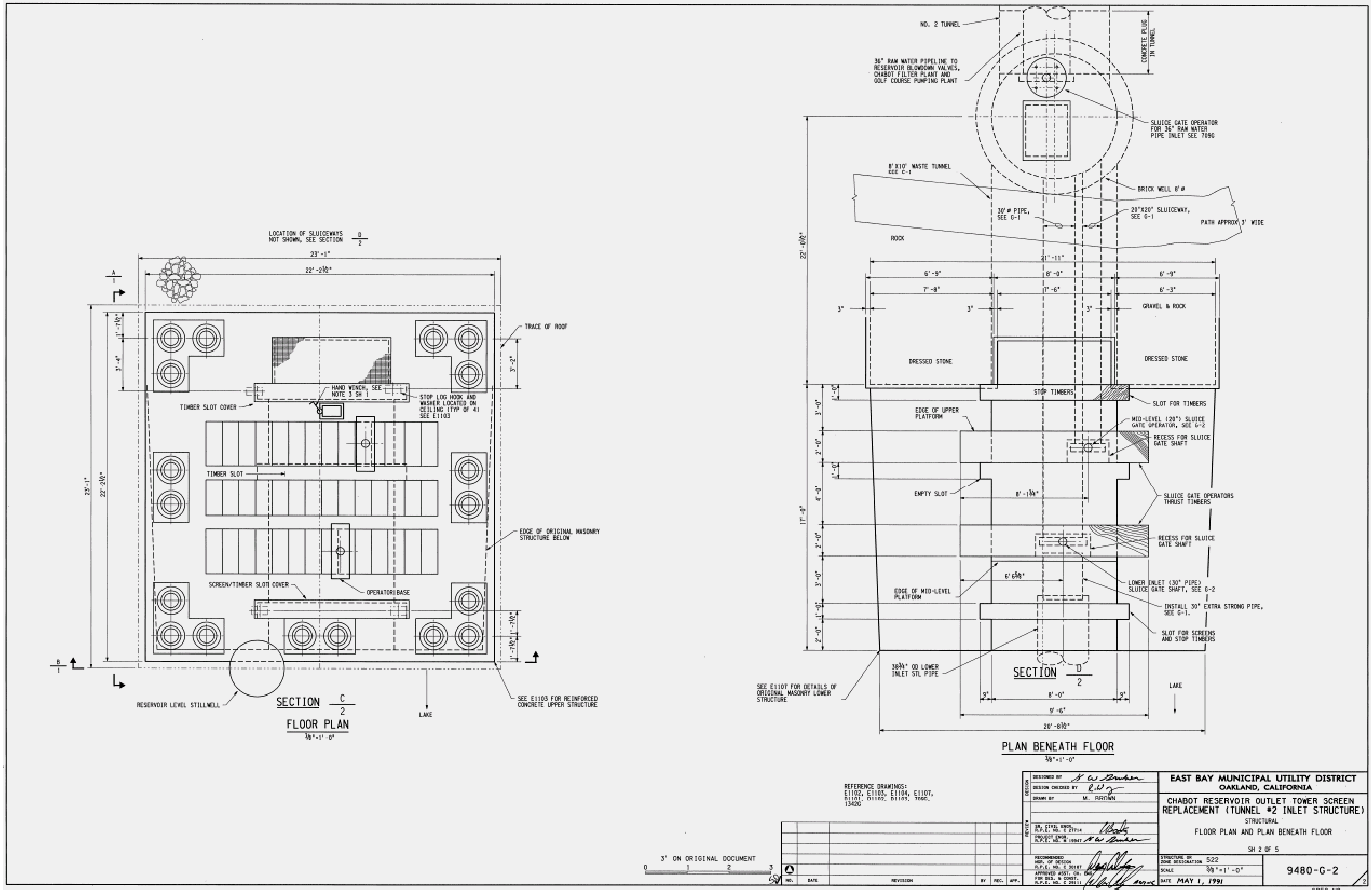


Figure 2-3: Floor plan and plan beneath the floor of tower structure

2.3 SCOPE OF WORK

The scope of work for seismic performance evaluation of Chabot Outlet Tower consisted of the following tasks:

Task 1: Material properties and condition assessment

This task involved a review of existing data and site visits to assess the physical condition of the concrete and masonry. The field conditions and observations were documented with photographs and sketches. The existing drawings and historical photographs were retrieved to establish the as-built geometry and modifications. Default lower-bound and expected material properties for the concrete and masonry were established in accordance with FEMA 356 and also based on the field observations and inspection. This task was performed by Tennebaum-Manheim Engineers (TME) and reviewed by Quest Structures. A summary of the findings of this task is reported by TME as Attachment I to this report.

Task 2: Simplified baseline analysis

This task included a review and updating of the 1991 District's calculations. The updated analysis consisted of equivalent-lateral-force calculations in accordance with the 2001 California Building Code (CBC). Both the reinforced-concrete pavilion and the masonry tower were analyzed and section capacities for the reinforced-concrete and masonry members were calculated using the material properties established under Task 1. The demand-capacity ratios for various members were computed to assess seismic performance of the tower and to compare with the results of finite-element analysis. The simplified analysis was carried out by OLMM Consulting Engineers and reviewed by Quest Structures. The results and findings of this task are reported in Attachment II.

Task 3: Three-dimensional finite-element analysis

The task of 3D linear-elastic response-spectrum analysis was performed by Quest Structures using the material properties established under Task 1. This task consisted of the following activities.

a. Conduct a site visit and review existing data, design and construction drawings, and previous calculations to establish the geometry and evaluation methodology.

b. Develop a SAP2000 3-D linear model consisting of the masonry tower, reinforced-concrete pavilion, and the abutment and foundation rock. The added hydrodynamic mass of the surrounding and contained water were to be estimated using standard procedures.

c. Perform linear-elastic analysis using the 3D model with three components of earthquake response spectra applied along principal axes of the tower. The seismic input was to include the 5%-damped response spectra for the MDE and MCE developed by URS.

Task 4: Evaluation of seismic performance of tower

This task was also performed by Quest Structures with the following subtasks:

a. Evaluate the results by comparing stress and force demands with strength and force capacities to assess the seismic performance of the tower. Depending on the severity of damage, assess the need for additional work or retrofit fixes.

b. Prepare a detailed engineering report to summarize the results of the tower evaluation including the data review, the analysis methodology and conclusions, and recommendations for further work, if necessary.

3. EVALUATION CRITERIA

The seismic evaluation criteria for Chabot Outlet Tower were established based on force and stress demand capacity ratios and consideration of potential failure modes. The evaluation criteria were formulated considering the following:

- An approach based on demand-capacity ratios
- Review of existing data and available drawings and historical photographs to establish geometry and method of construction
- Site visit to assess physical condition of the concrete and masonry
- Establishment of design/evaluation earthquakes
- Establishment of material properties in accordance with FEMA 356 and the UBC as well as the visual assessment of structure
- Evaluation loads including static and seismic
- Methods of analysis including both simplified code procedures and a more detailed three-dimensional finite-element structural analysis

3.1 METHODOLOGY AND APPROACH

The earthquake performance of Chabot Outlet Tower is assessed by comparing seismic force demands with section capacities of the reinforced-concrete pavilion, and seismic stress demands with tensile and shear strengths of the plain stone masonry. Such comparisons tend to show what region of the tower will suffer damage in the form of yielding of reinforcing steels and cracking and/or crushing of the concrete and masonry. For this purpose, the seismic force and stress demands are obtained from the 3D linear-elastic finite-element analysis using the established material properties. The shear and moment capacities of the reinforced-concrete members are estimated in accordance with the ACI specifications and the US Army Corps of Engineers EM 1110-2-2400 (USACE, 2003). For reinforced-concrete columns the moment capacity is obtained from the axial force-bending moment interaction diagrams. The shear, tensile, and compressive stress capacities of the brick and stone masonry are established from the FEMA and UBC specified strength values.

If the results of linear-elastic analysis indicate that the force and stress capacities are not exceeded, the tower is judged to perform satisfactorily. Otherwise, the magnitudes and spatial extent of demand-capacity ratios are used to assess severity of the damage and probable modes of failure. The demand-capacity ratios for brittle mode of behavior involving shear should not exceed 1, while the demand-capacity ratios for flexural behavior of reinforced-concrete members could reach a value of 2. The tensile and shear demand-capacity ratios for the plain masonry should also not exceed 1.

3.2 REVIEW OF EXISTING DATA

Existing information including site plans, structural drawings, and historical photographs were reviewed to establish the geometry and method of construction for seismic evaluation of the tower. A list of all drawings retrieved for this review is given in Attachment I. The data show that the stone masonry tower was embedded and cast against the abutment rock. The tower walls are mainly made of stone masonry, except that concrete and bricks were also used in the top of the walls. In addition, bricks were employed on the inside faces of the walls. Based on this information, the three-dimensional model of the tower was arranged accordingly and material properties were assigned consistent with distribution of the stone masonry, concrete, or bricks.

The 1991 District analysis of the Chabot Outlet Tower was reviewed and is discussed in Section 4.0 of Attachment II. The 1991 analysis was based on the 1988 UBC assuming that the pavilion is a Special Moment Resisting Space Frame (SMRSF) and that the tower walls are cantilevered at the base. The tower was analyzed for two levels of seismic forces and found to be severely damaged in both cases.

3.3 DESIGN/EVALAUTION EARTHQUAKES

The Chabot Tower is evaluated for the maximum design earthquake (MDE) and checked for the maximum credible earthquake (MCE). The MDE is defined as the maximum level of ground motion for which the structure is designed or evaluated (USACE, 2003). The MDE was chosen by the District as a ground motion having a 10 percent probability of exceedance in 50 years (a return period of 475 years). Since the selected MDE ground motion is lower than the level of ground motion at 10 percent in 100 years (a return period of 950 years) recommended by USACE (2003), the tower is also checked for the MCE ground motion. In the period range of interest (< 0.2sec), the MCE ground motion corresponds to the 1300- to 1500-year motion. In this period range of interest, the 950-year motions are 25 to 30% higher than the MDE motion per URS memorandum (2004a and b).

By definition the MDE ground motion is estimated probabilistically by considering contributions from all significant seismic sources of different magnitudes and distances. The MDE ground motions in the form of equal hazard response spectra are given in Section 3.6.3.

The MCE ground motions at the site were estimated for stability analysis of Chabot Dam using a deterministic approach. Among several seismic sources considered, the Hayward fault, located 0.5 km west of the dam, was found capable of generating the strongest ground motion at the site and was selected as the controlling MCE. The estimated maximum magnitude for the Hayward fault is M_w 71/4.

3.4 MATERIAL PROPERTIES AND CONDITION ASSESSMENT

Material properties and condition assessment of Chabot Tower are described by Tennebaum-Manheim Engineers in Attachment I. Concrete was assessed in accordance with FEMA 356-6.3.3.2.1. Overall, the visual inspection indicates that the tower structure is in good condition. The pavilion roof shows signs of spalling and rust jacking, but the remainder of the concrete appears to be in good condition. Based on these assessments a knowledge factor of 0.75 was assigned to the pavilion roof and 1.0 to the concrete below the roof. The knowledge factor, as required by FEMA 356, is used to account for uncertainty in the collection of as-built concrete or masonry data. For example, default strength values for the pavilion roof were reduced by 25 percent to account for the spalling and rusting damage.

Masonry was assessed in accordance with FEMA 356-7.3.3.1. Only masonry above the water level could be examined, which appeared to be in good condition.

Table 3-1 lists the lower bound and expected material properties established for the concrete and masonry based on default values and visual examination of the structure. As discussed later in Section 5-3, Chabot Tower is a short-period structure with force-controlled seismic behavior. Thus its seismic response is governed by the magnitudes of forces and stresses rather than deflections caused by flexural response. As a result the lower bound material properties will be used in the analysis and evaluation.

In addition to material properties of the concrete and masonry, the elastic properties of the abutment rock were also needed for the 3D analysis of the tower. The elastic modulus and Poisson's ratio of the abutment rock were estimated by URS based on the measured seismic velocities at the dam site, the rock condition at the tower, and the anticipated level of ground shaking. A rock modulus of 720 ksi and a Poisson's ratio of 0.43 were obtained, which are consistent with shear and compression velocities of 850 and 2500 m/s, respectively.

Table 3-1: Summary of material properties reported by TME

Material (density)	Location	Property	Lower Bound (psi)	Expected Strength (psi)
Concrete (150 pcf)	Roof beams & slab	Compressive Strength, f'_c	1875	2812
		Reinforcing Tensile Strength (& Yield)	41,250 (24,750)	51,560 (30,938)
		Elastic Modulus, E_c	2,850,000	2,850,000
	Columns, floor, slab, and beams	Compressive Strength	2500	3750
		Reinforcing Tensile Strength (& Yield)	55,000 (33,000)	68,750 (41,250)
		Elastic modulus, E_c	2,850,000	2,850,000
Brick (120 pcf)	Throughout	Compressive Strength	900	1170
		Tensile Strength	20	26
		Shear Strength	27	35
		Elastic Modulus	643,500	643,500
		Shear Modulus	257,400	257,400
Stone Masonry & Dressed Stone (160 pcf)	Throughout	Compressive Strength	1800	2340
		Tensile Strength	20	26
		Shear Strength	54	70
		Elastic Modulus	1,287,000	1,287,000
		Shear Modulus	514,800	514,800

3.5 METHOD OF ANALYSIS

Chabot Outlet Tower includes two unique structural features that significantly affect its seismic response. These include the abutment support on the downstream face and the plain stone masonry construction. The abutment provides additional support and excitation along the height of the tower. The tower is therefore not a freestanding cantilever and its behavior must be captured using a three-dimensional model. Similarly, the pavilion structure is attached to the abutment through a concrete slab bridge on the back of the structure, a condition that will subject the pavilion to torsion and must be treated in three dimensions. The plain masonry construction introduces modes of failure that to a large extent depend on the fracture of mortar joints due to tension and shear. Consequently, the 3D finite-element response-spectrum analysis has been adopted to more accurately address these issues. The 3D model described later includes the masonry tower, the pavilion, the effects of inside and outside water, as well as a portion of the foundation and abutment rock adjacent to the tower structure.

In addition to the 3D finite-element analysis, an equivalent lateral load calculation based on current code requirements was carried out to update the 1991 District analysis and also to provide baseline results for the more elaborate 3D finite-element analysis.

3.6 EVALUATION LOAD

The following loads are considered for the 3D response-spectrum analysis of Chabot Tower.

3.6.1 Dead Loads

Dead loads for concrete and stone and brick masonry are based on their respective unit weights listed in Table 3-1. The dead loads due to reinforced concrete that make up the pavilion were assumed the same as the weight of plain concrete and were applied the same way. The weight of the pre-cast floor was distributed as nodal loads depending on the tributary area.

3.6.2 Hydrostatic Loads

Hydrostatic pressures acting on the east (upstream), west (abutment), north and south faces of the tower were computed using a unit weight of 62.4 pcf for the impounded water. The water level was assumed at El. 227.25 feet, same as the spillway crest. These hydrostatic pressures were applied to the corresponding faces of the 8-node solid element. Note that although the horizontal hydrostatic loads in the north-south direction cancel out, there exists a net hydrostatic force in the abutment direction due to sloping and stepped construction of the outside faces of the walls.

3.6.3 Seismic Loads

Seismic loads for evaluation of Chabot Tower consist of inertia forces generated by horizontal and vertical components of the MDE response spectra. The 5%-damped MDE equal-hazard acceleration response spectra for the horizontal and vertical directions were developed by URS (2004b). They are listed in Table 3-2 and are also shown in Figure 3-1. The estimated peak horizontal and vertical accelerations for the MDE are 0.74g and 0.72g, respectively.

The 5%-damped response spectral accelerations for the horizontal component of the MCE are given in Table 3-3 and shown in Figure 3-2. Also provided in Table 3-3 are ratios of the MCE to MDE spectral accelerations for comparison. These ratios show that the MCE spectral accelerations are about 40 percent higher than those of the MDE in the period range of the tower structure (i.e. less than 0.3 sec). Accordingly, the linear-elastic seismic response due to the MCE will be about 40% higher than that estimated for the MDE.

Table 3-2: MDE Response Spectra at 5% damping

Period (sec)	Response Spectral Acceleration, S_a(g)	
	Horizontal	Vertical
0.02	0.74	0.72
0.05	1.10	1.59
0.07	1.28	1.84
0.10	1.49	1.86
0.15	1.70	1.43
0.20	1.76	1.21
0.30	1.59	0.90
0.50	1.20	0.62
0.75	0.89	0.47
1.00	0.66	0.37
1.50	0.43	0.26
2.00	0.30	0.19

Table 3-3: MCE Response Spectra at 5% damping

Period (sec)	Response Spectral Acceleration, S_a(g)	
	Horizontal	MCE/MDE Ratio
0.010	1.05	1.41
0.020	1.05	1.41
0.050	1.49	1.36
0.075	1.78	1.39
0.100	2.05	1.38
0.150	2.41	1.42
0.200	2.55	1.45
0.300	2.44	1.54
0.400	2.26	1.66
0.500	2.04	1.51
0.750	1.67	1.88
1.000	1.40	2.13
1.500	0.95	2.22
2.000	0.70	2.32
3.000	0.43	
4.000	0.30	

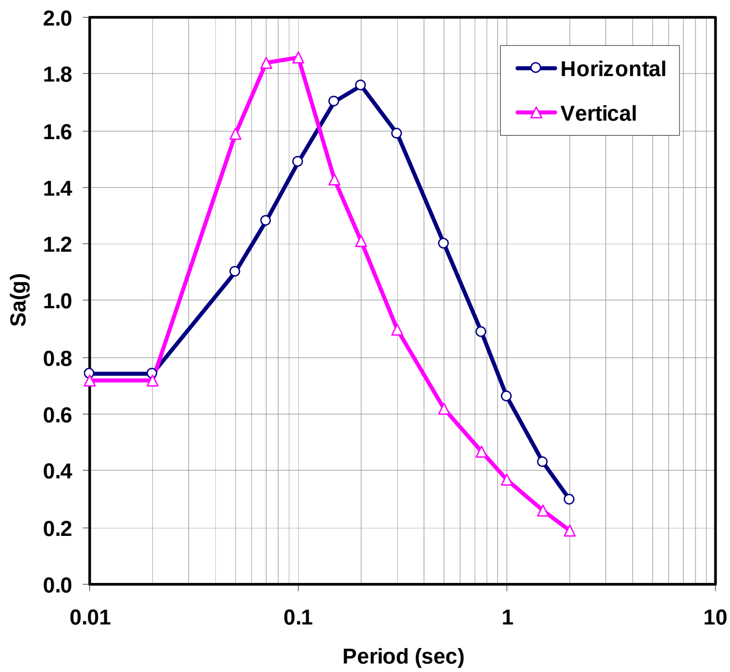


Figure 3-1: MDE Horizontal and Vertical Spectral Accelerations at 5% damping

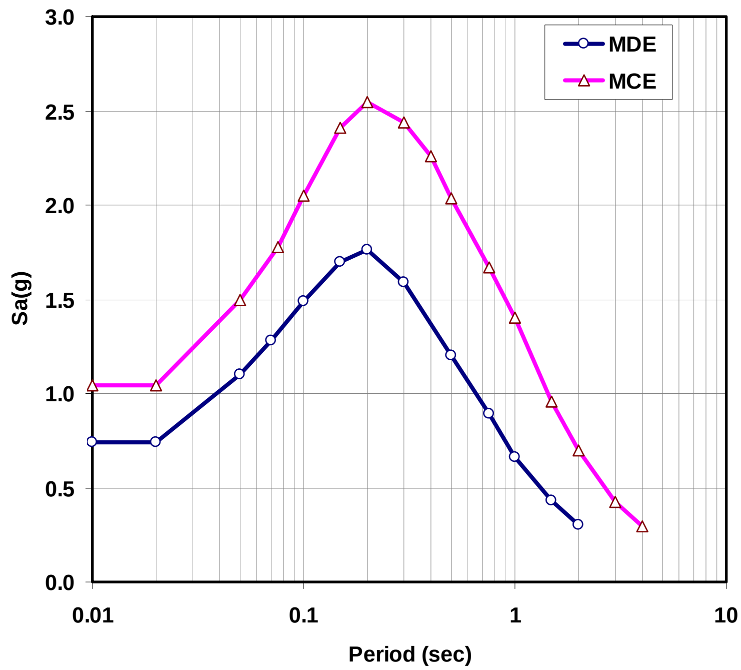


Figure 3-2: Comparison of MCE with MDE Horizontal Spectral Accelerations at 5% damping

3.6.4 Hydrodynamic Loads

The hydrodynamic effect of surrounding water due to seismic loading is represented by added mass terms using the Generalized Westergaard Added-Mass Method (Kuo 1982). For inside water, the total mass of water captured in the tower was distributed among the interior nodes in accordance with the tributary area.

3.6.5 Load Combinations

The 3-D finite-element analyses of Chabot Tower are performed for the usual static and seismic loading combinations. The self weight and hydrostatic loads are applied separately to check the model and then combined with the effects of seismic loads due to three components of ground motion as follows:

$$Q = Q_{SW} + Q_H \pm \sqrt{Q_{EX}^2 + Q_{EY}^2 + Q_{EZ}^2} \tag{3-1}$$

where

Q	=	Peak value of thrust, shears, and moments or stresses due to self weight, hydrostatic, and seismic loads
Q_{SW}	=	Effects resulting from self weight
Q_H	=	Effects resulting from hydrostatic pressures
Q_{Ex}	=	Effects resulting from the x (north-south) component of input response spectra
Q_{Ey}	=	Effects resulting from the y (east-west) component of input response spectra
Q_{EZ}	=	Effects resulting from the z (vertical) component of response spectra

4. SIMPLIFIED BASELINE ANALYSIS AND SECTION CAPACITIES

4.1 STRUCTURAL MODELING

The simplified analysis and computation of section capacities are reported by OLMM as Attachment II to this report. The simplified analysis was carried out based on equivalent lateral forces in accordance with the 2001 California Building Code. The pavilion and the tower were analyzed separately using an importance factor of 1.5 per 2001 CBC.

Two cases were analyzed. In Case I, the pavilion was assumed fixed at its lower level and was analyzed for a base shear of 1.125 times its weight. The base shear factor was obtained from the seismic Zone 4 specification with Fault Type A, Soil Type S_B, and near source distance of 0.5 km. Note that the resulting base shear factor of 1.125 is twice the 0.54 used in the 1991 District analysis per 1988 UBC. In Case II, the lower one-third of the tower is embedded was assumed to be embedded in the abutment rock. This assumption resulted in 25% reduction in the base shear, but tensile and shear stresses still exceeded corresponding capacities.

In both cases the masonry tower was analyzed as a cantilever structure, which resembled the Cantilevered Column Building Systems in 2001 CBC. The seismic forces included inertia forces due to the mass of walls and mass of the pavilion, but ignored water inertia forces caused by seismic shaking. The water inertia forces appear to be significant and could increase seismic base shear by as much as 25 to 50%.

4.2 SECTION CAPACITIES

Computation of section capacities is discussed in Appendix A of the OLMM report. The flexural, axial, and shear capacities for various members were computed using the material properties established in Section 3.3 and the current code standards. However, the resulting capacities for certain members were reduced to account for inadequate or lack of shear reinforcements and insufficient confinement and detailing that are necessary to develop full capacities. For example, the beam moment capacities were taken as 50% of the code-calculated values, while the moment and axial force capacities for columns were taken as 33% of those given by the code.

The flexural strength of pavilion columns subjected to both bending moment and axial load is characterized by the axial load-bending moment interaction diagram. Computation of the interaction diagrams was accomplished using the PCACOL computer program. The axial load reduction factor of $\varphi_p = 0.7$, and the bending moment reduction factor of $\varphi_M = 0.9$ were used in accordance with the ACI code. The resulting interaction diagram is displayed in Figure 4-1.

The masonry strength parameters in Table 3-1 were obtained from FEMA 356. However, a literature search indicated that other sources such as the UBC recommend significantly different values. For this evaluation, therefore, the tensile and shear strengths of brick and stone masonry were established as the average of the values given by FEMA and the UBC in Tale 4-1 below.

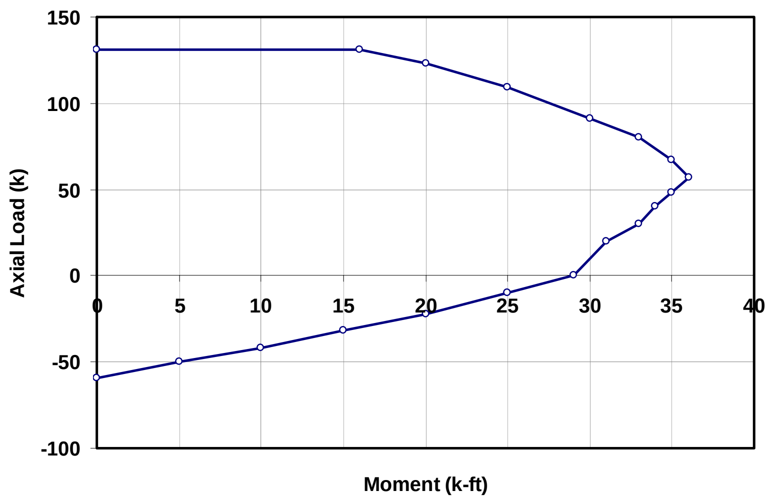


Figure 4-1: Axial load-bending moment interaction diagram for pavilion columns

Table 4-1: Summary of member capacities

Structure	Member	Moment Capacity (k-ft) f= 0.9		Shear Capacity (kips) f= 0.85
		At Supports	At Midspan	
Reinforced Concrete Pavilion	Roof Beam	32*	30*	51.01
	Roof slab bridge	--	--	80.09
	Columns	Varies with axial force, see Figure 4-1		8.93
	"L" Shape Floor Beam-1	24	32	19.51
	"L" Shape Floor Beam-2	40	32	25.76
	Interior Rectangular Floor Beam	23	30	10.46
	Beam Connecting Tower Walls	64*	71*	61.20

* Moments reduced by 50% due to lack of ties and detailing

Structure	Material Type	Compressive Strength[1] (psi)	Tensile Strength (psi)	Shear Strength (psi)
Masonry Tower	Concrete	2500	250[2]	100[2]
	Brick	900	17.5[3]	21[3]
	Dressed Stone	1800	14[4]	31[4]
	Stone Masonry	1800	14[4]	31[4]

1 Compressive strengths per TME as listed in Table 3-1

2 Tensile and shear strengths of concrete were taken equal to $0.1f_c'$ and $2(f_c')$^$^{1/2}$, respectively, per 2001 CBC.

3 Tensile and shear strengths of brick were adjusted by averaging values reported by TME with allowable working stresses given for joints by UBC Table 24-B (Tensile = (20+15)/2=17.5, shear = (27+15)/2=21 psi).

4 Tensile and shear strengths of stone were adjusted by averaging values reported by TME with allowable working stresses given for joints by UBC Table 24-B (Tensile = (20+8)/2=14, shear = (54+8)/2=31 psi).

4.3 RESULTS AND FINDINGS OF SIMPLIFIED ANALYSIS

Results of the simplified analysis are summarized in the form of demand-capacity ratios for various structural members in Tables 4-2 and 4-3. Based on the demand-capacity ratios (DCR) listed in Table 4-2, the OLMM analysis indicates that the roof beams could fail in bending and that in the absence of ties the longitudinal reinforcement might buckle. The pavilion columns show failure both in bending and shear. Again the lack of ties and adequate confinement in the columns is likely to lead to the collapse of the pavilion structure.

The DCR values for different material layers that make up the tower are summarized in Table 4-3. The results indicate that the masonry sections of the tower (i.e. 90% of height) are overstressed in tension; the stone masonry section (i.e. 80% of height) is also overstressed in shear. Based on these results, the simplified analysis indicates that severe damage could be expected across the walls leading to possible collapse of the tower.

Table 4-2: Summary of force demand-capacity ratios for pavilion

Structure	Member Type	DEMAND-CAPACITY RATIO (DCR)		
		Moment at Supports	Moment at Midspan	Shear Force
Reinforced Concrete Pavilion	Roof Beam	<u>**3.96**</u>	<u>**2.82**</u>	0.71
	Column	Moment + Axial:	<u>**5.85**</u>	<u>**1.23**</u>
	Floor Beam	0.67	0.41	0.42
	Roof Slab Bridge			0.92

Table 4-3: Summary of stress demand-capacity ratios for masonry tower

Layer No.	Height (ft)	Material Type	DEMAND-CAPACITY RATIO (DCR)*		
			Compressive	Tensile	Shear
1	4’-6”	Concrete	0.02	0.13	0.15
2	1’-6”	Brick	0.08	<u>**3.07**</u>	0.81
3	3’-3”	Dressed Stone	0.08	<u>**7.91**</u>	0.70
4	35’-9”	Stone Masonry	0.73	<u>**86.81**</u>	<u>**1.48**</u>
* Case-I: full embedment					

5. THREE-DIMENSIONAL FINITE-ELEMENT ANALYSIS

As discussed in Section 3.4, the seismic performance of Chabot Intake Tower should be evaluated using 3D finite-element analysis. The support and excitation provided by the abutment cannot be handled by code procedures, where the structure is assumed to be fixed at its base only. This is because, the code procedure applies the entire lateral force to the base of the structure in the form of overturning moment and shear, thus ignoring the fact that only a portion of the total lateral force reaches the base and the remainder is resisted by the abutment support. The 3D analysis of Chabot Intake Tower was conducted using the SAP2000 finite-element program. It involved developing a 3D structural model for the masonry tower, the pavilion structure, and a portion of the abutment and foundation rock supporting the tower, followed by application of static and seismic loads to assess earthquake performance of the tower to ground motion hazard dominated by the nearby Hayward Fault.

This chapter presents the 3D modeling, analysis procedures, and evaluation of the results. The evaluation begins with static analysis to check the 3D finite-element model by applying and examining the effects of each load separately. The 3D model is analyzed for the self weight and hydrostatic load cases. The evaluation then continues by performing linear-elastic response- spectrum analysis of the tower with and without the bridge support at the back of the pavilion.

5.1 DESCRIPTION OF FINITE ELEMENT MODEL

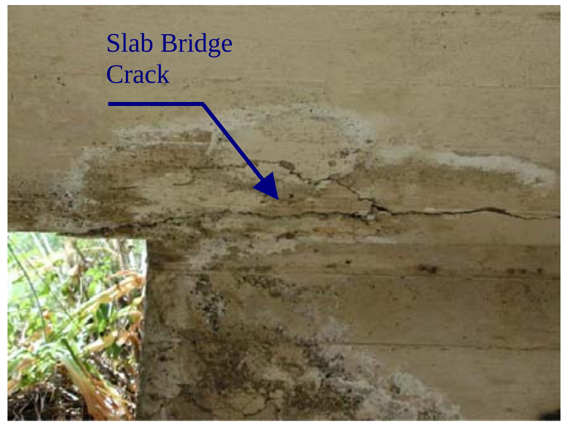
Figures 5-1 to 5-7 show an elaborate finite-element model developed for the masonry tower, the pavilion, the bridge, and the foundation and abutment rock supports. The geometry was obtained from the available drawings, historical photographs, and data collected during the site visit. The masonry walls and the natural foundation and abutment rock were discretized with an assembly of 8-node solid elements. The reinforced concrete pavilion was modeled by a combination of frame, shell, and 8-node solid elements. The complete finite element model consisted of 7,388 solid elements, 150 frame elements, 229 shell elements, and 9,636 nodal points. A refined model such as this was necessary to permit shear contribution from higher modes.

5.1.1 Masonry Tower

The finite-element mesh for the masonry tower was developed such that the four distinct material types including the concrete, brick, dressed stone, and the stone masonry could be grouped separately with its own properties, as given in Table 3-1. Brick layers on the inside face of the tower walls were also grouped separately so that brick properties could be assigned to these layers, thus distinguishing them from the adjacent concrete or stone masonry. The model also included a reinforced-concrete beam that connects the masonry walls at the top of the front face (Figure 5-3), a structural member that was not considered in the simplified analysis.

5.1.2 Reinforced Concrete Pavilion

Figure 5-6 shows the finite-element model for the reinforced-concrete pavilion separately. The columns and floor beams were represented by frame elements. The roof slab and parapet walls were modeled using shell elements, while the roof perimeter beams and the concrete footings were represented by 8-node solid elements. The pre-cast slabs covering the openings are not structurally significant, thus only their inertia forces in the form of nodal masses were represented and distributed according to the tributary area. An important structural feature of the pavilion is its connection to the abutment through the reinforced-concrete slab bridge in the back. The bridge not only restrains movements of the pavilion, but also excites the pavilion by the abutment motions. However, the bridge connection to the pavilion is vulnerable and could be severely damaged during the earthquake shaking. In fact, this connection has already cracked partially, as observed during the site visit (photo on right). Therefore, two cases were analyzed: 1) first the bridge was connected to the pavilion to assess its effects and vulnerability, and 2) it was not connected to the pavilion after it had been determined that it would completely crack.



5.1.3 Foundation and Abutment Rock

A portion of the foundation and abutment rock was included in the finite-element model of the tower structure to provide support for the structure, to account for flexibility of the surrounding rock, and to excite the structure from both the base and abutment supports. The foundation and abutment rock model was developed by extending a mesh of 8-node solid elements a distance equal to the tower dimensions in the downward, left and right, and backward directions. The foundation and abutment mesh were assumed massless, thus only flexibility of the rock was considered. The seismic input was applied at exterior foundation-abutment nodes, the boundary nodes that were assumed to be fixed in space.

5.1.4 Hydrodynamic Effects of Water

The hydrodynamic effects of outside water due to seismic loading were represented by added-mass terms using the Generalized Westergaard Added-Mass Method (James S.-H. Kuo, 1982). For the fully contained inside water, the added-mass was represented by the weight of water distributed among the interior nodes in accordance with the tributary area. The reservoir water elevation was assumed to be at the spillway crest elevation of 227.25 ft.

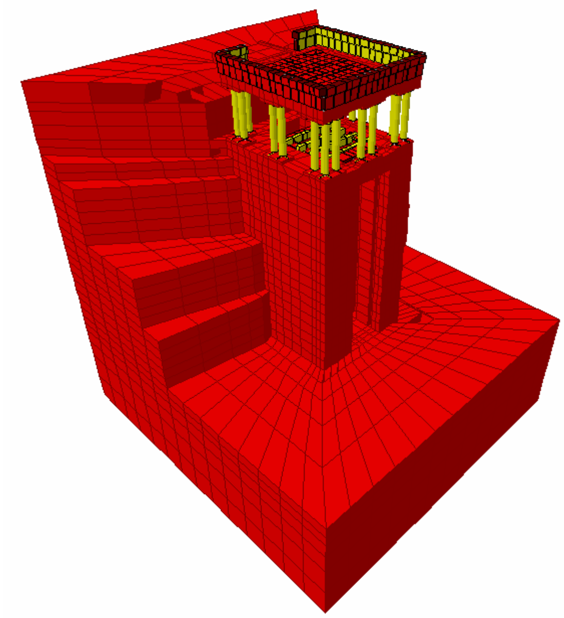


Figure 5-1: Front view of the tower, foundation, abutment, and pavilion model

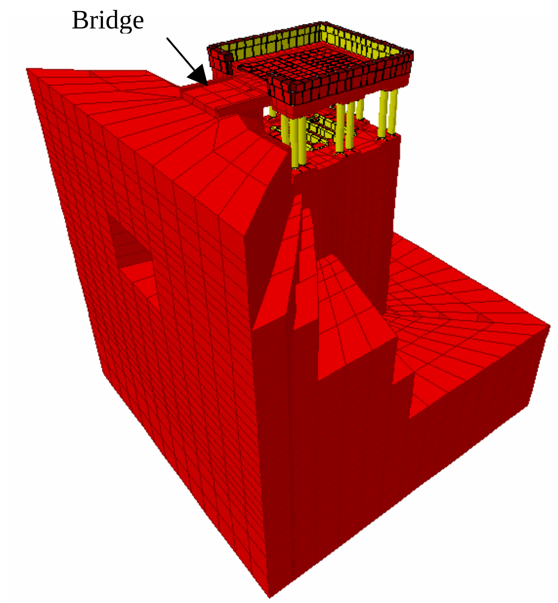


Figure 5-2: Back view of the tower, foundation, abutment, and pavilion model

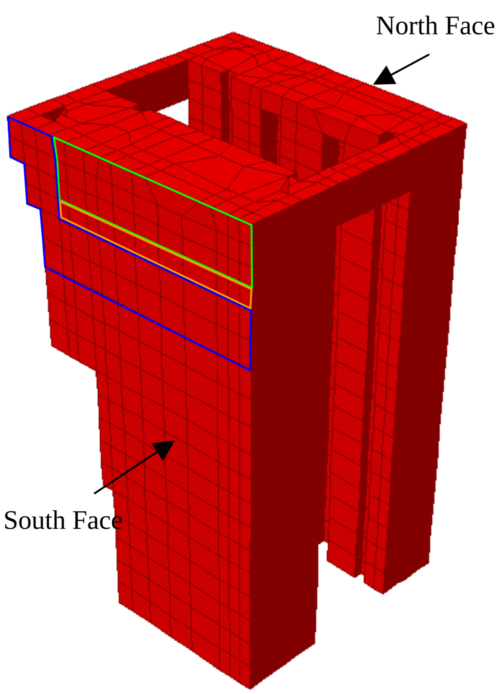


Figure 5-3: Front view of the tower model

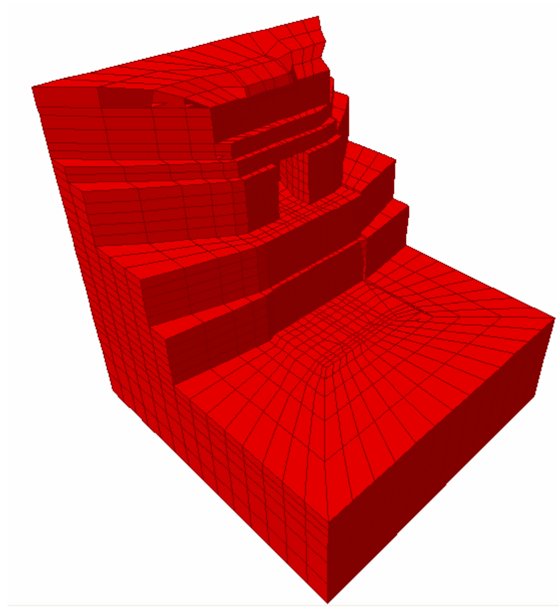


Figure 5-4: Front view of the foundation model

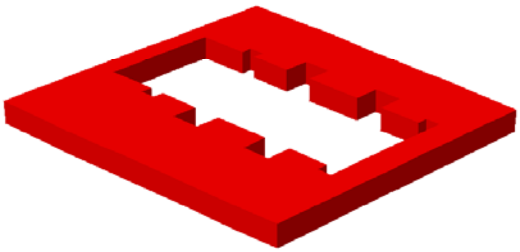


Figure 5-5: Top of Tower at El. 239 ft.

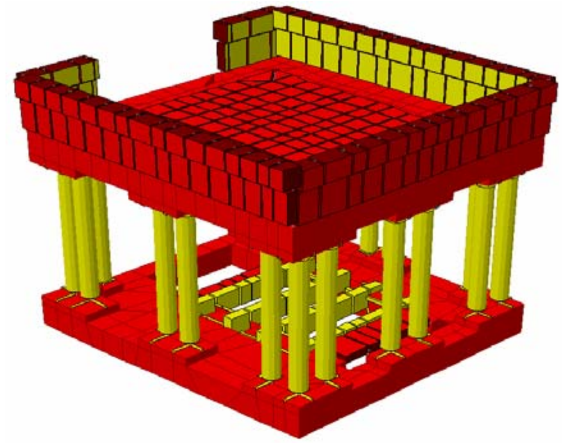


Figure 5-6: Pavilion model

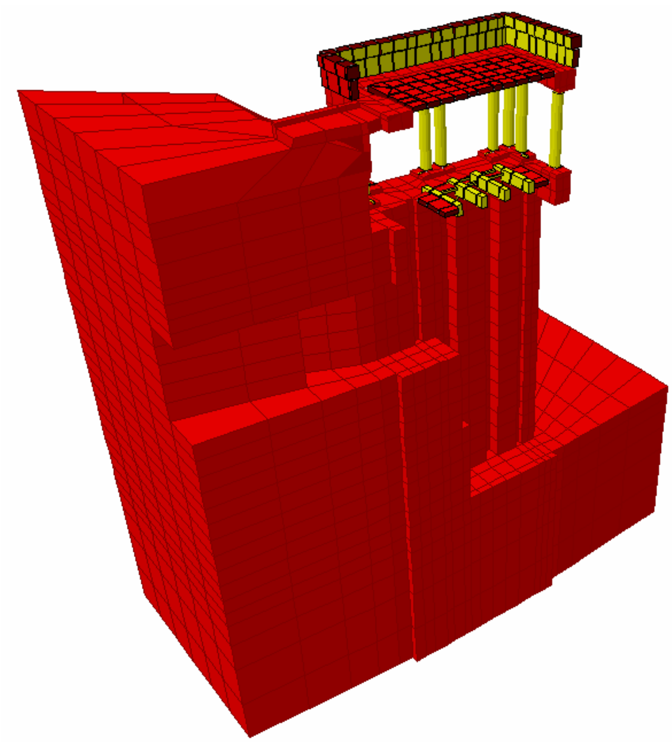


Figure 5-7: Vertical mid-section view showing stop timber slots and 8' x 10' waste tunnel. Also shown are pavilion roof beams and slab and floor beams and slab.

5.2 STATIC ANALYSES

The 3D finite-element model described in Section 5.1 was analyzed for gravity and hydrostatic loads to compute static stresses and forces that are required for combination with dynamic stresses and forces due to earthquake loading. The gravity and hydrostatic loads were applied separately so that the accuracy of the finite-element model could be verified, because gravity or hydrostatic stress patterns can easily be recognized and examined. The stresses are computed for all elements which include both the north and south walls. However, since the results are about the same for both walls, only the results for the south wall are discussed below.

The self weights of the pavilion and tower were determined and applied as described in Section 3.5.1. Figure 5-8 displays the self-weight vertical stresses on three faces of the south wall. The stresses range from -56 psi (compression) at the bottom of the wall to 0 psi at the top of the wall. As expected, the magnitudes of vertical stresses increase from top to bottom in accordance with the weight increase.

The hydrostatic loads were applied as surface pressures on appropriate faces of the tower walls as described in Section 3.5.2. Figure 5-9a shows the hydrostatic horizontal stresses on three faces of the south wall. The stress values range from 0 to -1.5 psi (compression) at the bottom to -12 psi at one element row above the base, and finally to 0 psi at the water surface. Note that the stress magnitudes are in close agreement with the hydrostatic surface pressures. The horizontal hydrostatic stresses parallel to the wall (y-direction) are shown in Figure 5-9b, while the vertical stresses caused by water pressures acting on the east face of the tower are shown in Figure 5-9c.

The north wall of the tower behaves similarly, except that deformations and the stresses for the thinner north wall are slightly higher than those shown for the south wall.

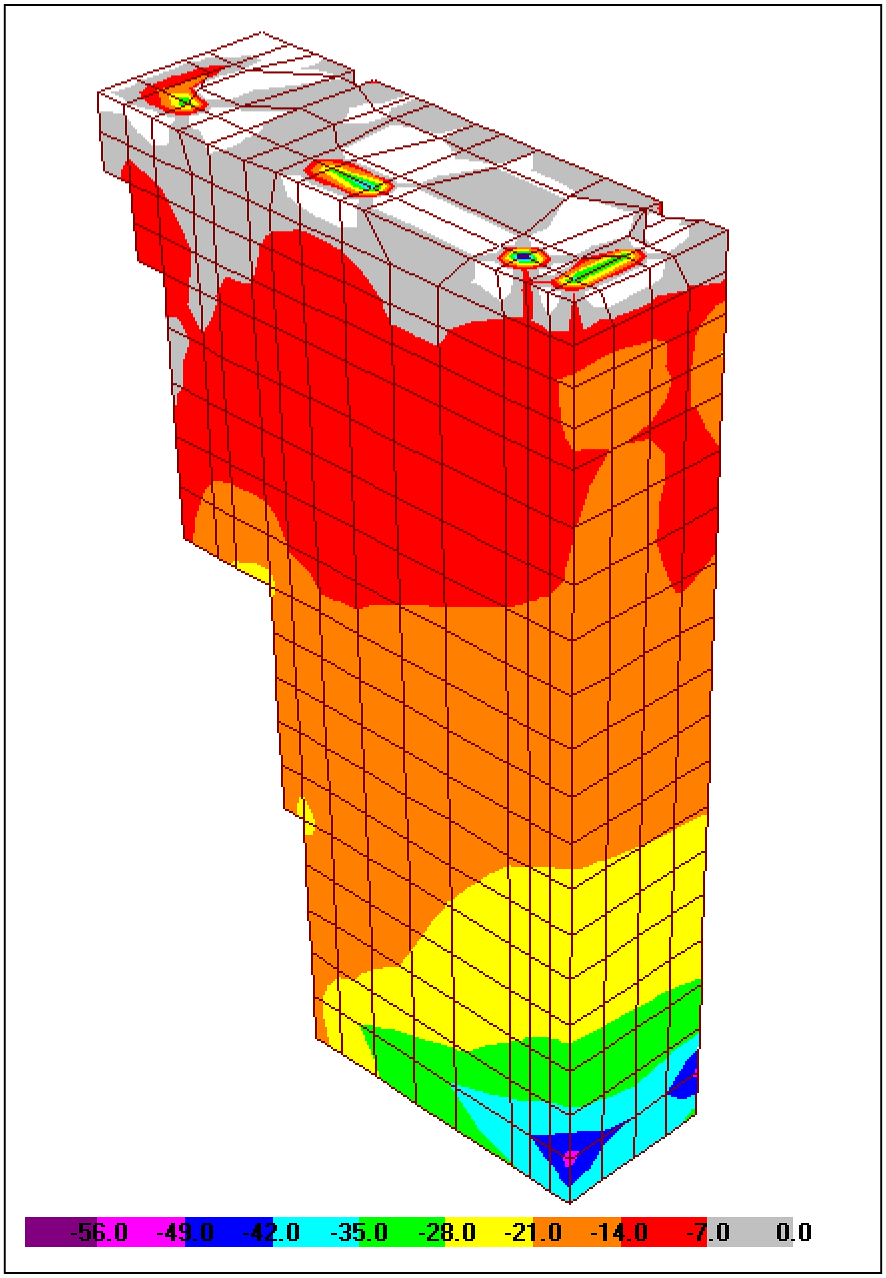


Figure 5-8: South-wall vertical stresses due to self weight (psi)

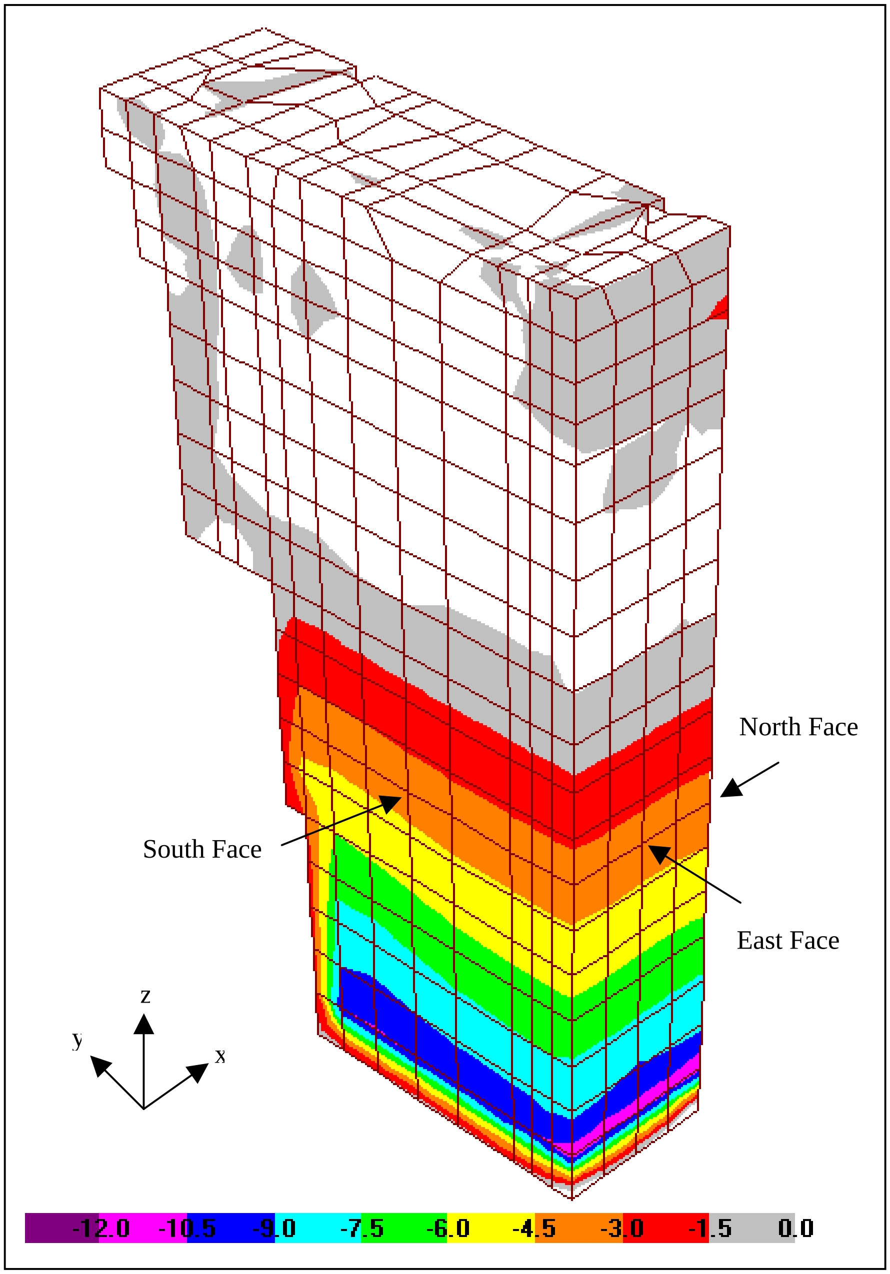


Figure 5-9a: South-wall horizontal stresses (σ_{xx}) due to hydrostatic pressures (psi)

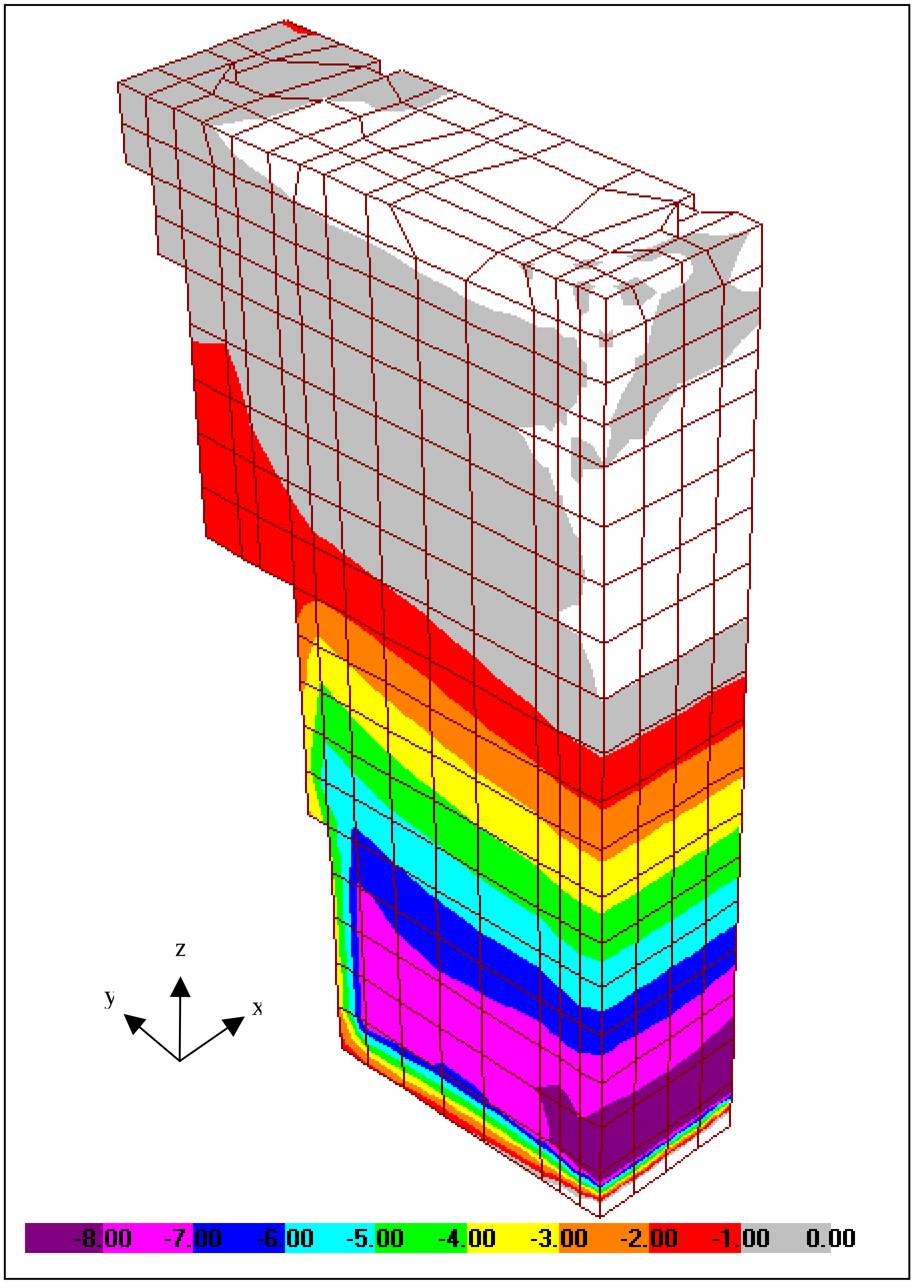


Figure 5-9b: South-wall horizontal stresses (σ_{yy}) due to hydrostatic pressures (psi)

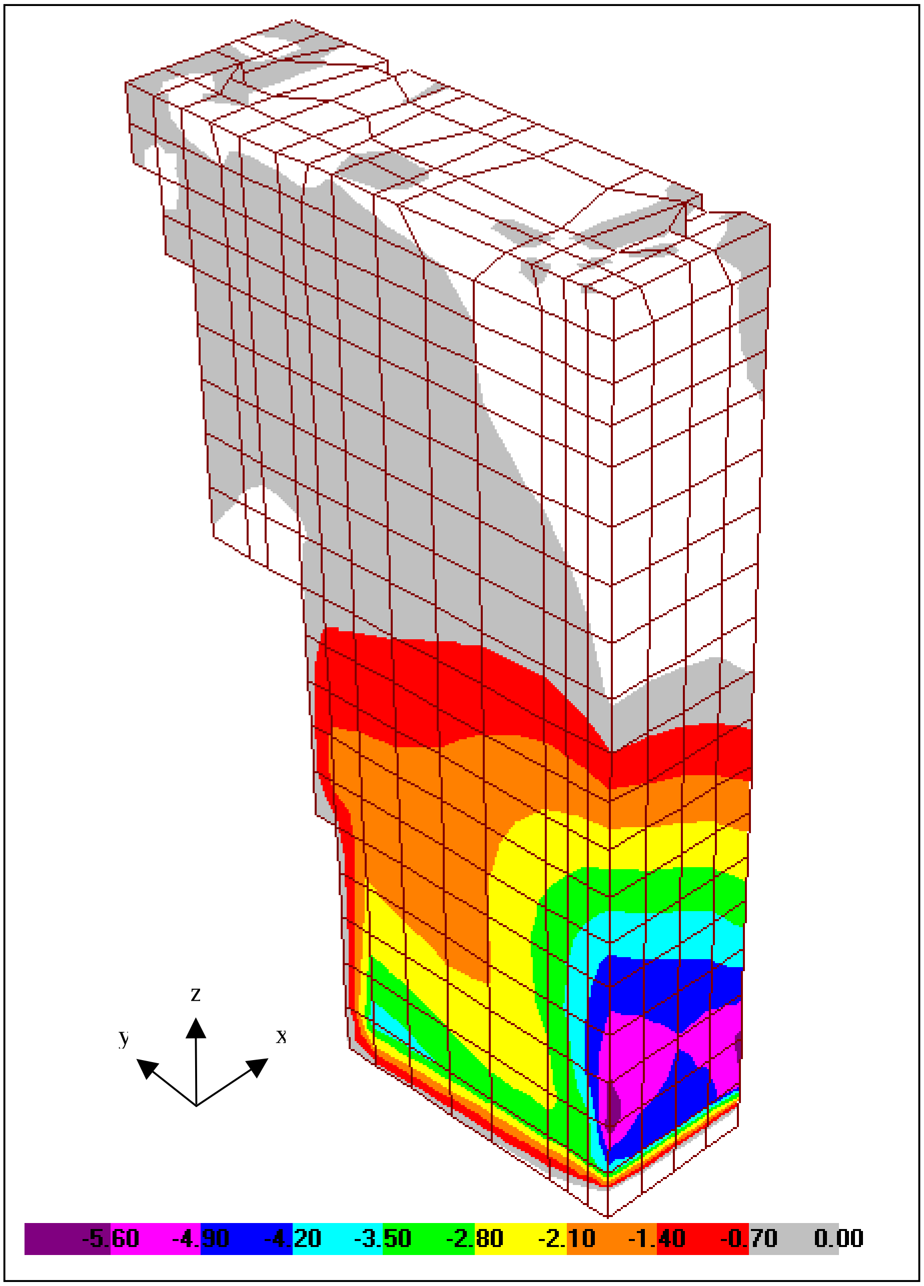


Figure 5-9c: South-wall vertical stresses (σ_{zz}) due to hydrostatic pressures (psi)

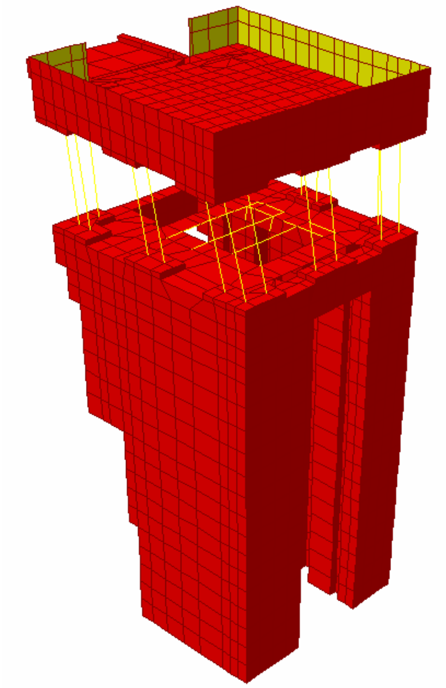
5.3 MODE SHAPES AND PERIODS

The vibration mode shapes and periods required for the earthquake response-spectrum analysis were computed using the finite-element model described in Section 5.1. The modal properties were estimated using Ritz vectors, which are more efficient than the eigenvectors. Results showed that superposition of 50 Ritz vectors accounted for more than 99% mass participation in each of the three directions. The 26 modes with 90% or more mass contribution in all three directions are listed in Table 5-1; the remaining modes contributed very little to the tower response and are not listed in the table. The periods range from 0.085 sec (11.76 Hz) to about 0.01 sec (100 Hz).

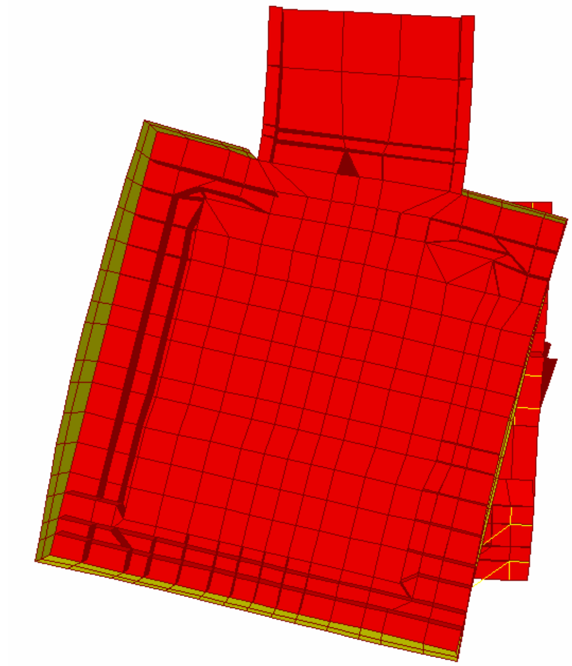
Figure 5-10 displays three of the most significant vibration modes of the Chabot Tower. The first mode with a period of 0.085 sec (11.76 Hz) is the pavilion bending mode, where the pavilion undergoes transverse deformations in the north-south direction with some noticeable amount of torsion caused by the bridge support, see Figures 5-10a and b. The second mode at 0.059 sec (16.98 Hz) represents a combined out-of-phase bending mode, where the pavilion and masonry tower bend transversely in opposite directions (Figure 5-10c). Note that this mode has a mass participation of 39.22 percent which is attributed to the mobilized mass of the masonry tower. The third mode with a period of 0.052 sec (19.6 Hz) and a mass participation primarily in the vertical direction (as compared to its mass participations in the N-S and E-W directions) is fundamental bending mode of the pavilion roof slab, as shown in Figure 5-10d. Based on these results, the tower structure is classified as a short-period (high-frequency) structure whose periods fall in the ascending region of the response spectra. This indicates a force-controlled (force capacity is attained prior to flexural capacity) behavior for which nonlinear deformations are not permitted.

Table 5-1 Vibration periods and modal participating mass ratios

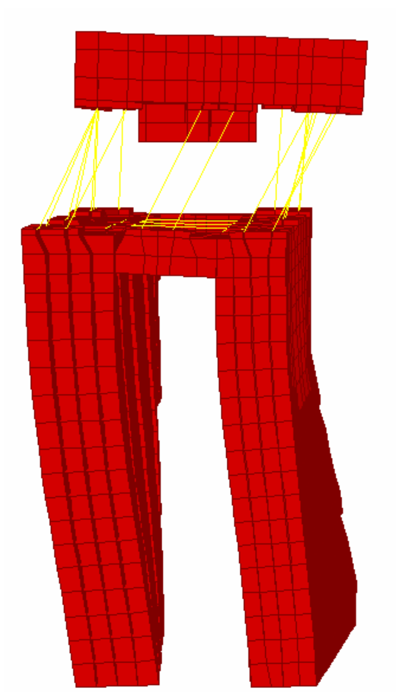
Mode	Period (sec)	Individual Mode (%)			Cumulative (%)		
		N-S	E-W	Vertical	N-S	E-W	Vertical
1	0.085	7.81	0	0	7.81	0	0
2	0.059	39.22	0	0.015	47.03	0	0.015
3	0.052	0	0.04	1.73	47.04	0.04	1.74
5	0.035	7.03	0.02	0.01	54.36	0.72	1.76
6	0.031	0	8.34	3.97	54.36	9.06	5.73
7	0.027	0.01	3.25	44.64	54.38	12.31	50.37
8	0.026	1.08	0.20	2.46	55.45	12.51	52.83
11	0.023	0	0.58	4.23	55.49	14.03	57.12
13	0.021	0	3.29	0.01	55.62	17.81	57.13
14	0.020	13.39	15.49	0.31	69.00	33.30	57.44
15	0.020	3.20	7.06	0.15	72.20	40.36	57.60
16	0.020	9.51	23.85	0.14	81.72	64.21	57.74
17	0.019	0.08	1.16	0.21	81.80	65.38	57.95
18	0.019	3.13	4.31	0.09	84.93	69.68	58.03
19	0.019	0.47	3.34	1.60	85.40	73.02	59.63
20	0.018	0.26	1.33	0.46	85.65	74.35	60.09
22	0.017	0.011	1.90	4.00	85.88	76.50	64.87
23	0.017	0	1.18	0.95	85.88	77.68	65.82
24	0.016	0.34	7.28	10.34	86.21	84.96	76.16
25	0.016	0.14	0.59	9.69	86.36	85.55	85.85
26	0.015	1.99	0.70	0.81	88.35	86.25	86.66
28	0.015	0.05	0.07	1.49	88.55	86.63	88.37
29	0.014	0.25	1.04	0	88.79	87.67	88.37
31	0.013	014	1.54	1.03	89.55	89.40	89.48
33	0.012	2.62	0.24	0.06	92.45	90.17	89.59
34	0.011	0.18	1.44	0.53	92.63	91.61	90.11



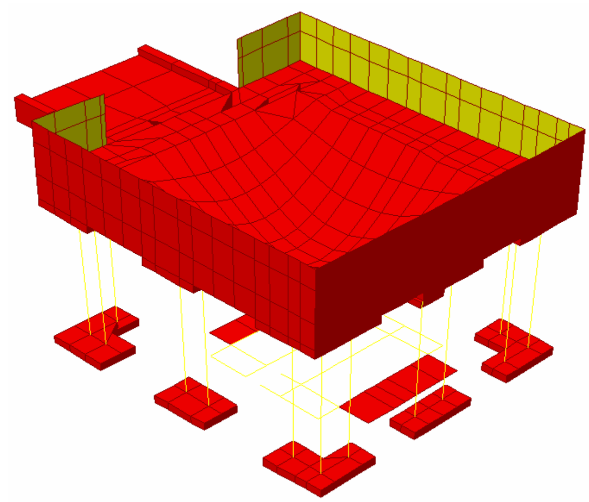
a) Mode-1 viewed from south east
T_1 = 0.0850 sec



(b) Mode-1 viewed from top
T_1 = 0.0850 sec



c) Mode-2 viewed from east (front face)
T_2 = 0.0589 sec



(d) Mode-3 viewed from south east
T_3 = 0.0522 sec

Figure 5-10: First three major mode shapes of Chabot tower

5.4 RESPONSE SPECTRUM ANALYSIS

Earthquake response analysis of Chabot Tower was carried out using the response-spectrum modal-superposition method. For this purpose, first the vibration mode shapes and periods of the tower-water-foundation system were calculated as discussed in Section 5.3; then the maximum stresses and forces for each mode (modal responses) were obtained for each component of the input response spectra. However, since each mode reaches its maximum response at a different time, the maximum response of the tower for each component (i.e. vertical and two horizontal components) of ground motion was obtained by combining the maximum modal responses for that component using the complete-quadratic-combination (CQC) method. In the final step, the maximum responses for the vertical and two horizontal components of the ground motion were combined by the square-root-of-the-sum-of-the-squares (SRSS) method to estimate the total dynamic response of the tower due to all three components of the earthquake response spectra. The input response spectra for earthquake analysis were those briefly described in Section 3.6.3. For each response-spectrum component, the spectral value at any period of vibration gives the maximum response of the mode having that period and the specified 5% damping.

The dynamic stress and force results obtained from the response-spectrum analysis represent the maximum stresses and forces that could develop in the masonry tower and pavilion at any time during the earthquake ground shaking. It should be noted that the response-spectrum stresses and forces are all positive and do not include contributions due to the static loads. Thus they are assumed to be either positive or negative when combined with the static responses to obtain the maximum and minimum total responses in the structure, as given by Equation 3-1.

5.4.1 Masonry Stress Results for MDE

Horizontal Normal Stresses (σ_{xx})

Figures 5-11a and 5-11b show the maximum and minimum horizontal normal stresses (σ_{xx}) in the north-south direction for the south wall. As discussed previously, the maximum values represent the static plus seismic stresses and the minimum values correspond to the static minus seismic stresses. Figure 5-11a indicates that the maximum stresses are concentrated at the back edges of the wall in the abutment region and also at the bottom edges in contact with the foundation. High tensile stresses exceed tensile strengths of the stone and brick masonry by more than a factor of 3, indicating that tensile cracks are likely to develop at the edges within the regions identified by dotted lines in Figure 5-11a. In other words, the north-south normal stresses (σ_{xx}) have the effect of breaking interface bonds and separating the walls from the foundation and abutment rock along the edges. However, the minimum stresses in the north-south direction (Figure 5-11b) are limited to -100 psi and remain well within the compressive strength of the masonry.

Horizontal Normal Stresses (σ_{yy})

Figures 5-12a and 5-12b display the maximum and minimum normal stresses (σ_{yy}) in the upstream-downstream (east-west) direction for the south wall. In Figure 5-12a, the overstressed regions with stresses exceeding the tensile strength of the masonry, are identified by dotted lines. The results show that high tensile stresses cover a significant portion of the wall. A comparison of Figure 5-12a with Figure 5-11a shows that both the magnitudes and overstressed regions for σ_{yy} are larger than those for the σ_{xx} stresses. The σ_{yy} tensile stresses develop predominantly due to bending of the wall about the vertical axis, as evident by large stresses on the back edges of the wall (Figure 5-12a). This suggests that vertical tensile cracks would develop parallel to the abutment. The cracks probably will occur at the abutment contact, but may not propagate to the entire overstressed region. This is because once the cracking occurs at the abutment contact, magnitudes of tensile stresses will drop in the walls and the extent of the overstressed region may be lower than indicated by the calculated stresses. However, it appears that the cracks at the abutment contact could be deep and might completely separate the walls from the abutment. Figure 5-12b indicates that compressive stresses are generally small and that with a peak value of -160 psi they are well within the compressive strength of the masonry.

Vertical Normal Stresses (σ_{zz})

The maximum and minimum vertical normal stresses (σ_{zz}) for the south wall are presented in Figure 5-13a and 5-13b. Unlike the horizontal normal stresses which are generated by the bending of the wall about the vertical axis, the vertical tensile stresses are predominantly caused by the bending of the walls with respect to horizontal axis. As expected, vertical stresses are highest at locations of the horizontal contact surfaces with the abutment and foundation. The results indicate that the vertical tensile stresses also exceed tensile strengths of the brick and stone masonry and could produce horizontal cracks within the dotted regions shown in Figure 5-13a, originating from the contacts with the abutment and foundation. The cracks could also occur in the upper front portion of the walls, especially if the beam connecting the two walls has failed. The vertical compressive stresses are moderate with the peak reaching -120 psi at the base of the tower.

Out-of-Plane Shear Stresses (σ_{xy})

The maximum and minimum out-of-plane shear stresses in the south wall are shown in Figures 5-14a to 5-14c. It can be seen from these figures that the static plus earthquake loads generate larger shear stresses than the static minus earthquake loads. High out-of-plane shear stresses with a peak value in excess of 35 psi occur along the back edges of the wall at about half height of the tower (Figures 5-14a and 5-14b). The out-of-plane shear stresses exceeding the shear strength of the masonry might lead to shear failure of the wall edges in contact with the abutment. The dotted regions in Figures 5-14a and 5-14b indicate the region with high shear stresses. However, the shear cracking may not extend beyond the contact regions with the abutment, mainly because initiation of cracking at the contact corners would decrease shear stresses in the walls.

In-plane Shear Stresses (σ_{yz})

Figures 5-15a and 5-15b display the maximum and minimum in-plane shear stresses for the south wall. The results show that in-plane shear stresses exceed shear strengths of brick (21 psi) and stone masonry (31 psi) over 75% of the walls' surface areas. Figure 5-15a indicates the possible diagonal cracking that might develop as a result of excessive in-plane shear stresses. Note that actual diagonal cracks probably will trace the joints and will be stepped as opposed to straight lines. Furthermore, the exact number of diagonal cracks is not known. It is quite possible that only two to three diagonal cracks may develop due to lack of reinforcement. Figure 5-15b shows that minimum in-plane shear stresses due to static minus earthquake loads also exceed shear strength of the masonry and could lead to additional stepped cracking in the lower part of the tower. Overall, the in-plane and out-of-plane shear stresses exceeding the shear strengths cover more than 75% of the masonry wall, an indication that shear failure will occur.

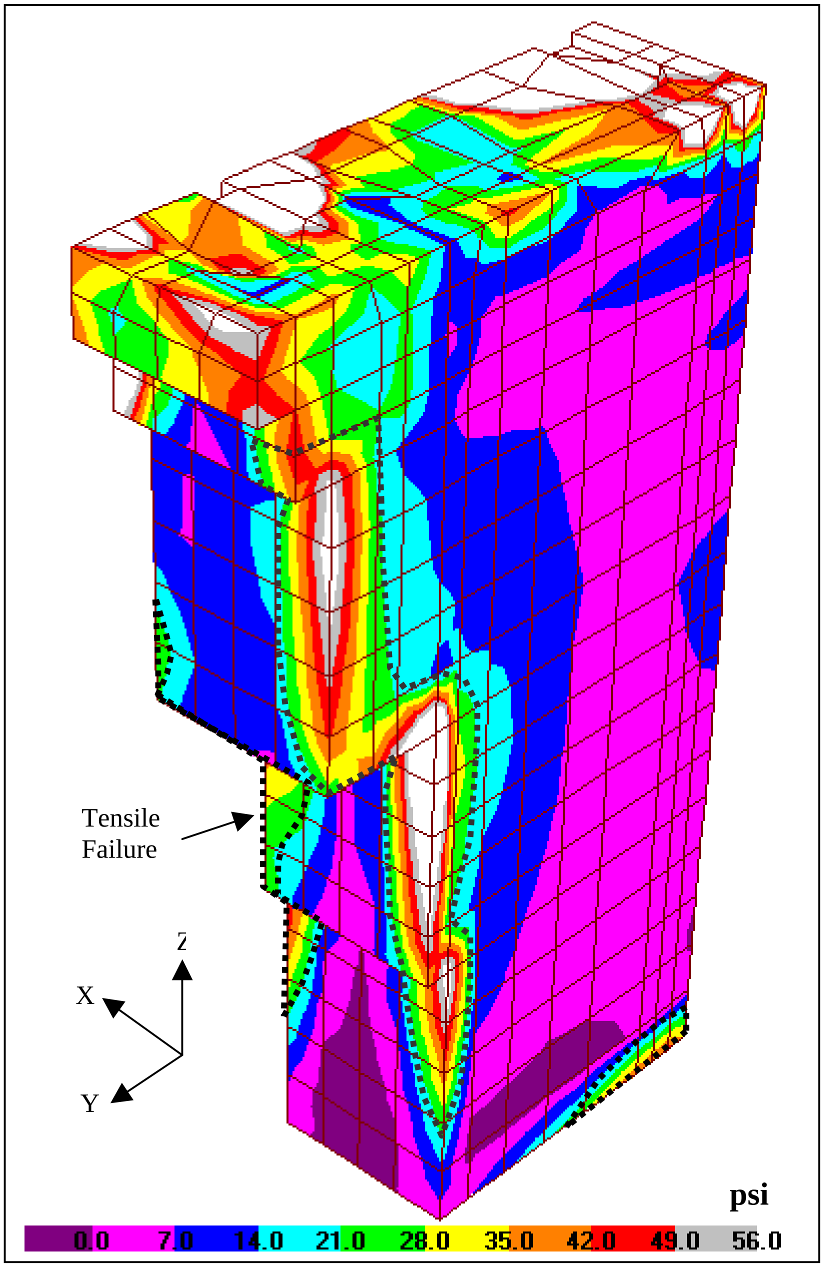
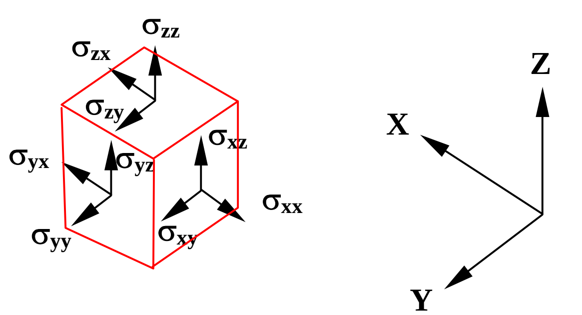


Figure 5-11a: Maximum horizontal normal stresses (σ_{xx}) for the south wall due to static plus earthquake loads. Regions within the dotted lines indicate potential tension failure.



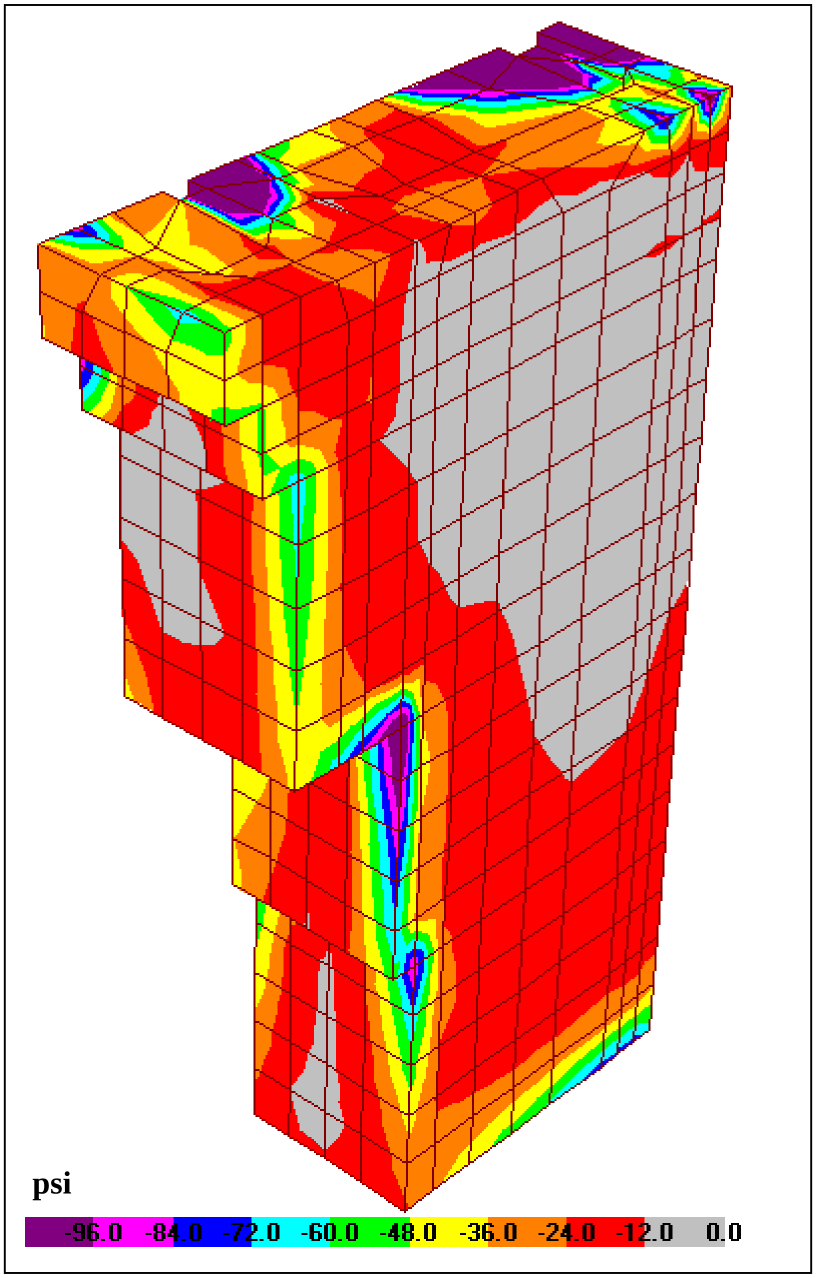
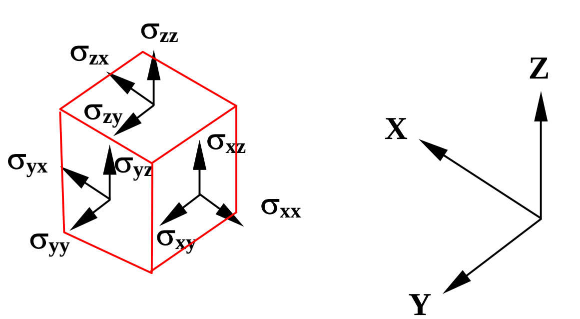


Figure 5-11b: Minimum horizontal normal stresses (σ_{xx}) for the south wall due to static minus earthquake loads.



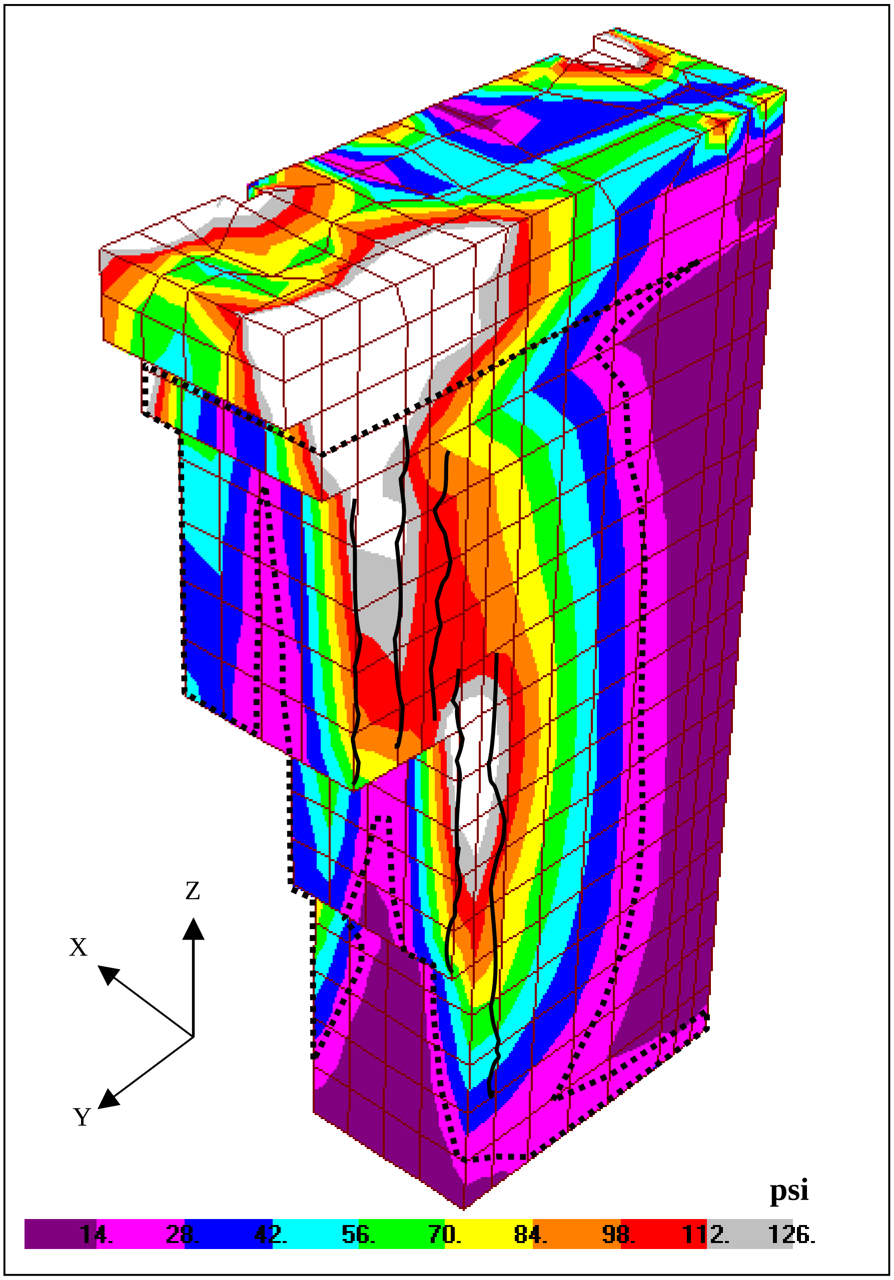
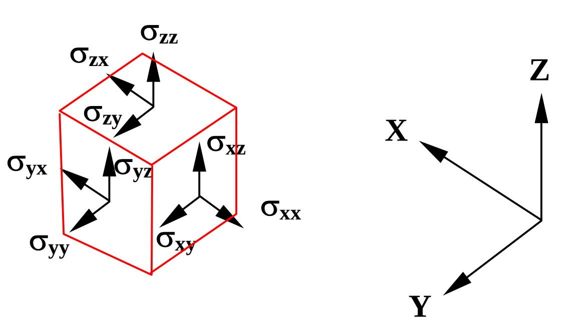


Figure 5-12a: Maximum horizontal normal stresses (σ_{yy}) for the south wall due to static plus earthquake loads. Regions within the dotted lines indicate potential tension failure.



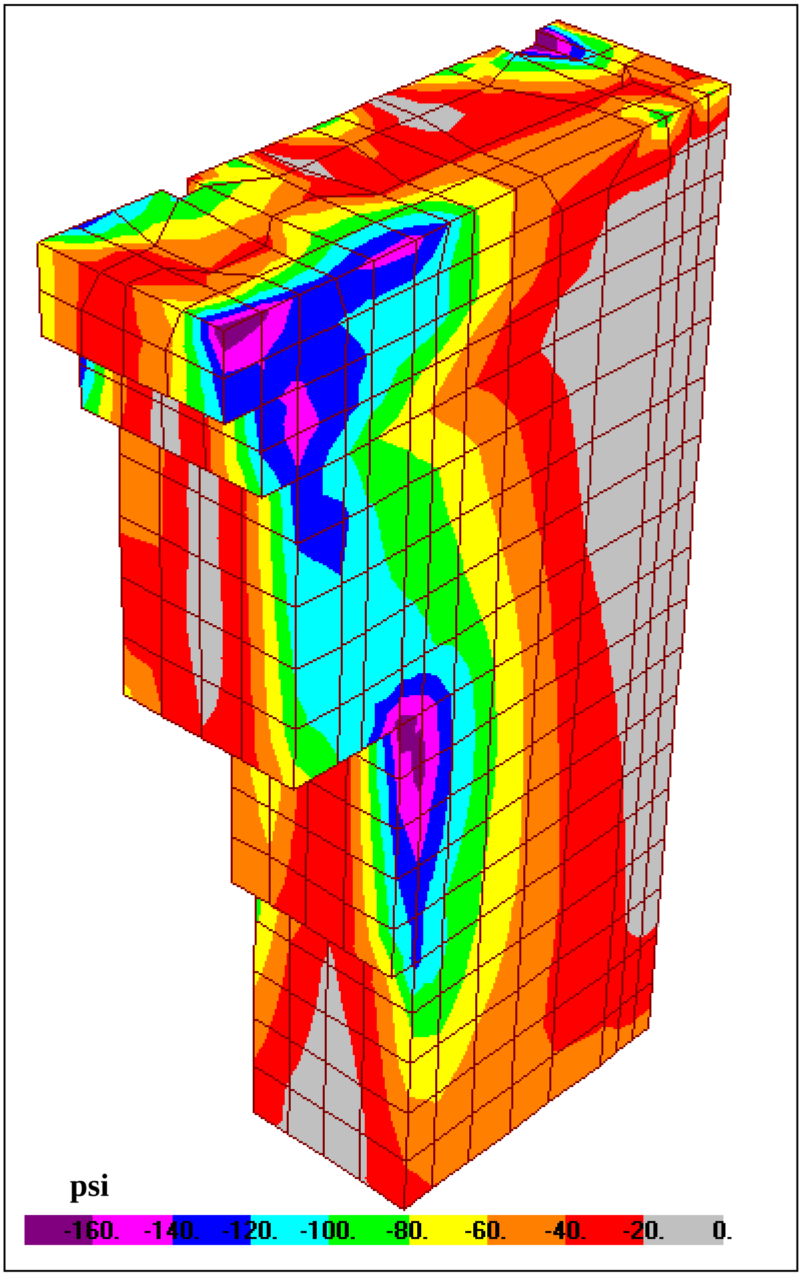
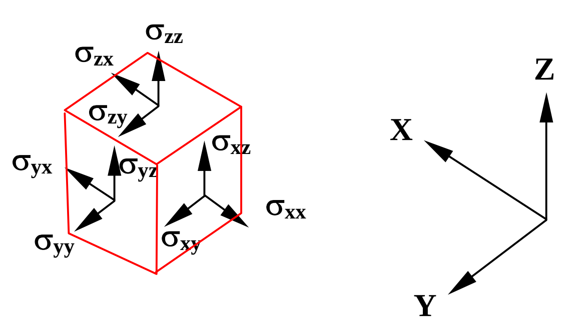


Figure 5-12b: Minimum horizontal normal stresses (σ_{yy}) in the south wall due to static minus earthquake loads.



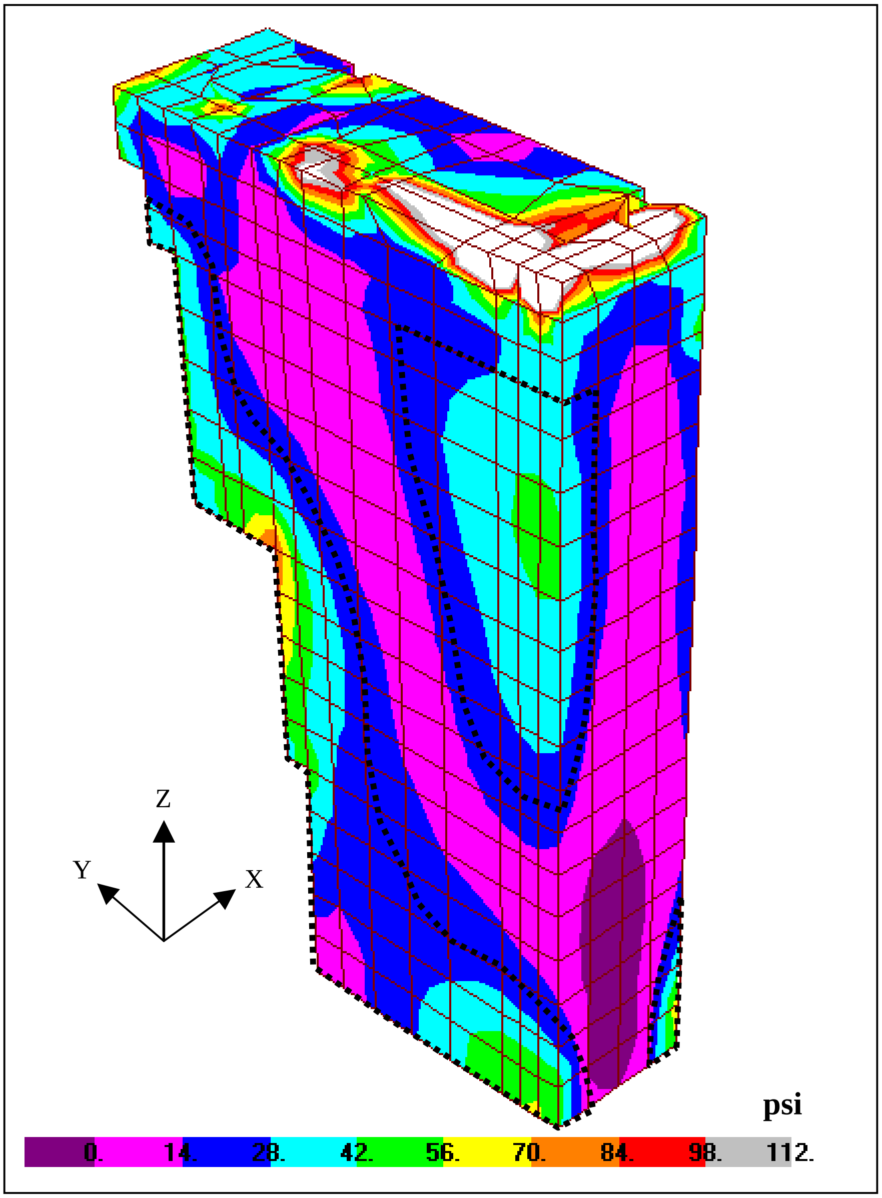
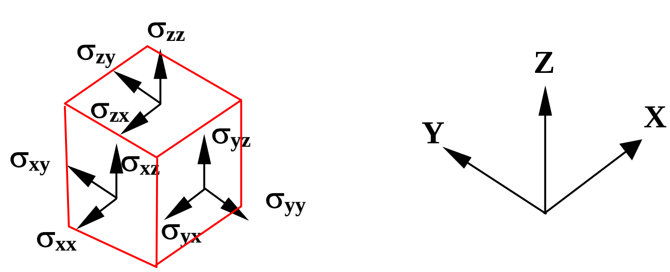


Figure 5-13a: Maximum **v**ertical stresses (σ_{zz}) in the south wall due to static plus earthquake loads. Regions within the dotted lines indicate potential tension failure.



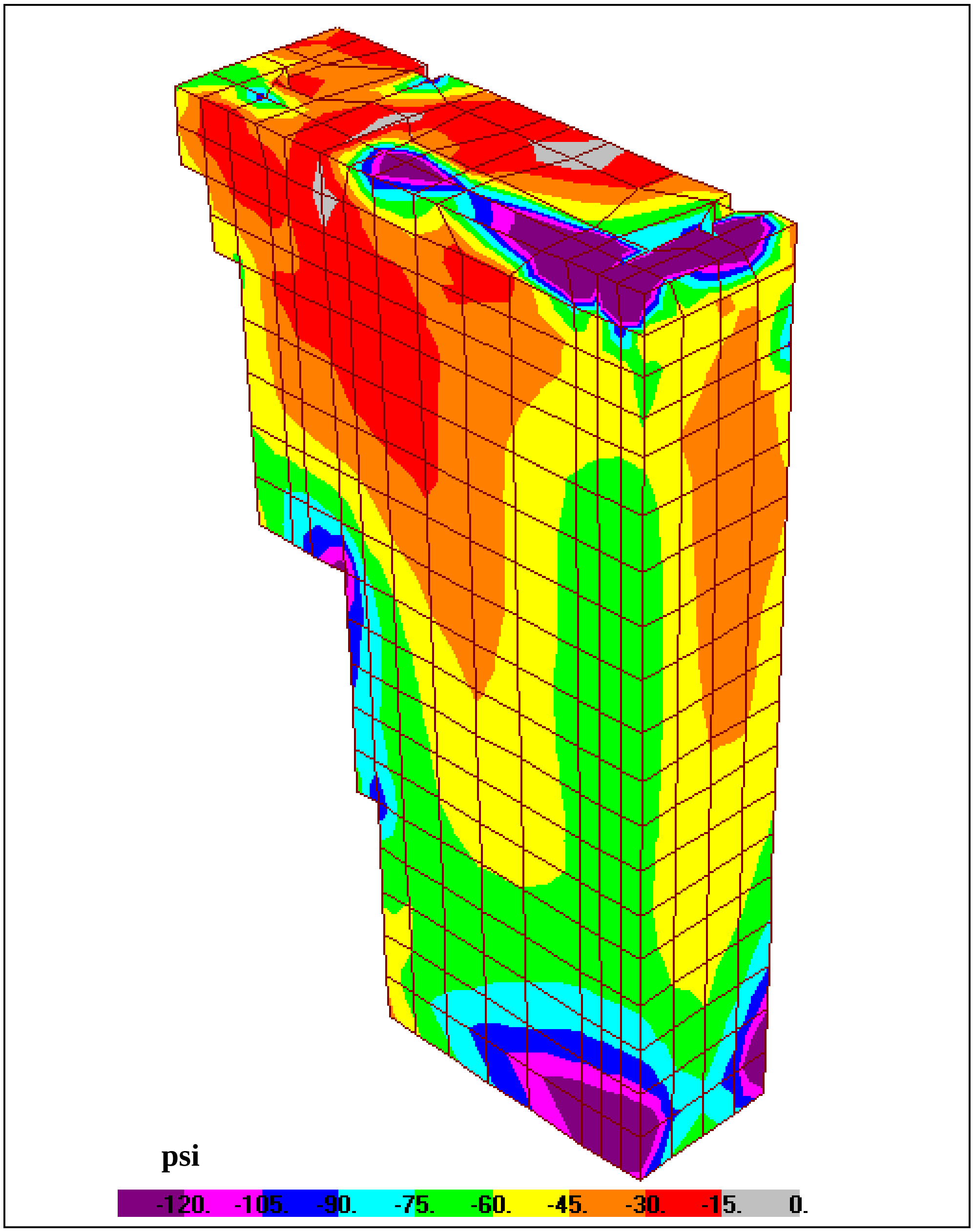
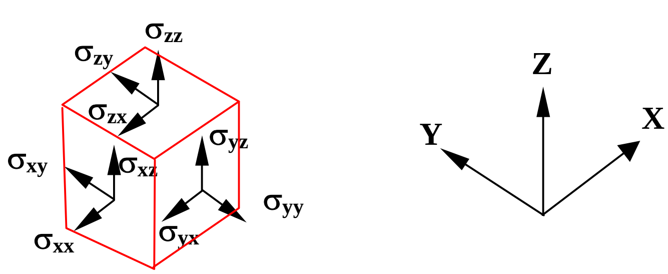


Figure 5-13b: Minimum **v**ertical stresses (σ_{zz}) in the south wall due to static minus earthquake loads.



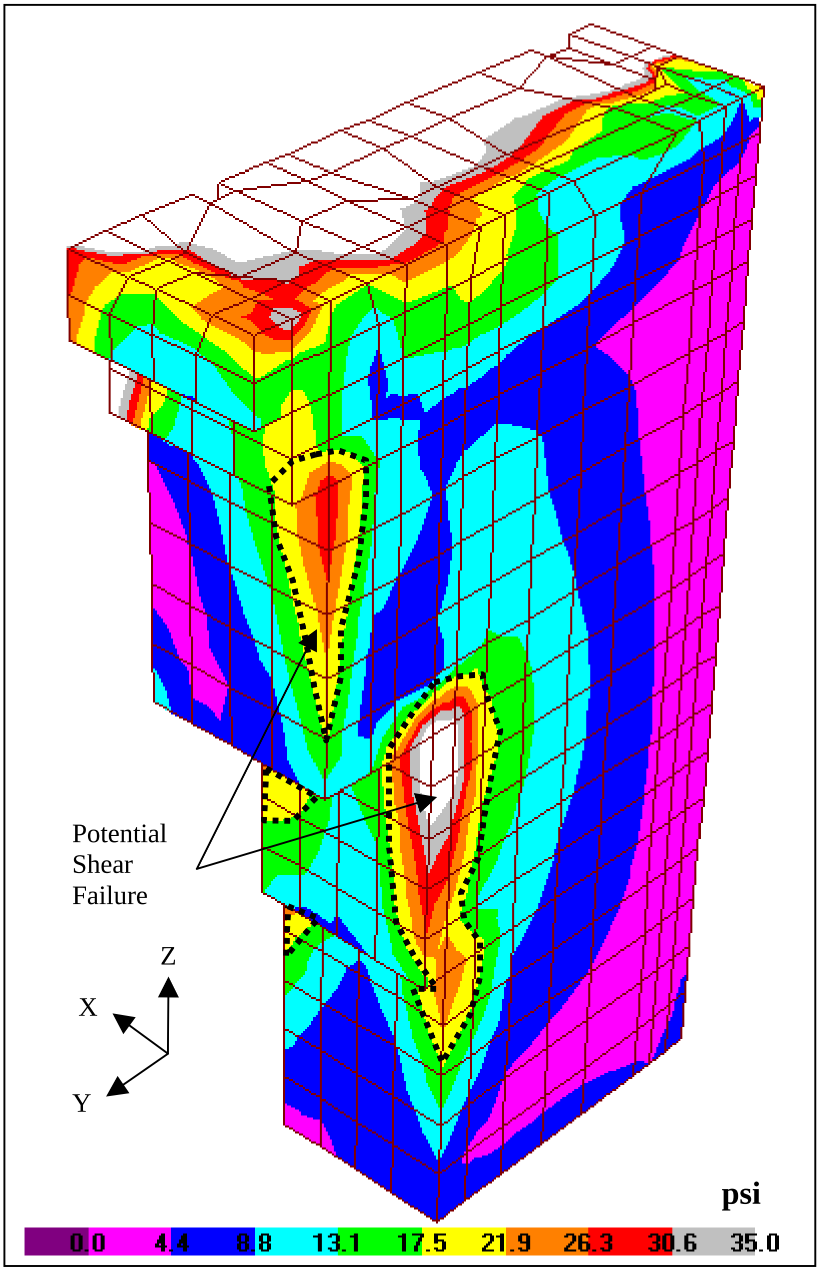
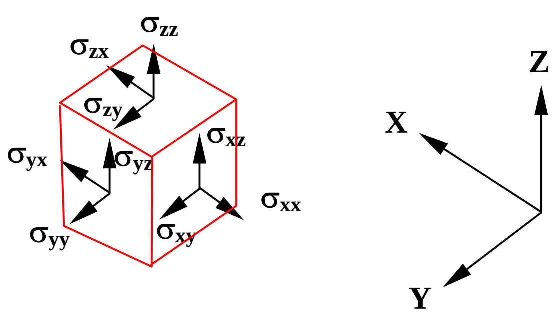


Figure 5-14a: Maximum out-of-plane shear stresses (σ_{xy}) for the south wall due to static plus earthquake loads.



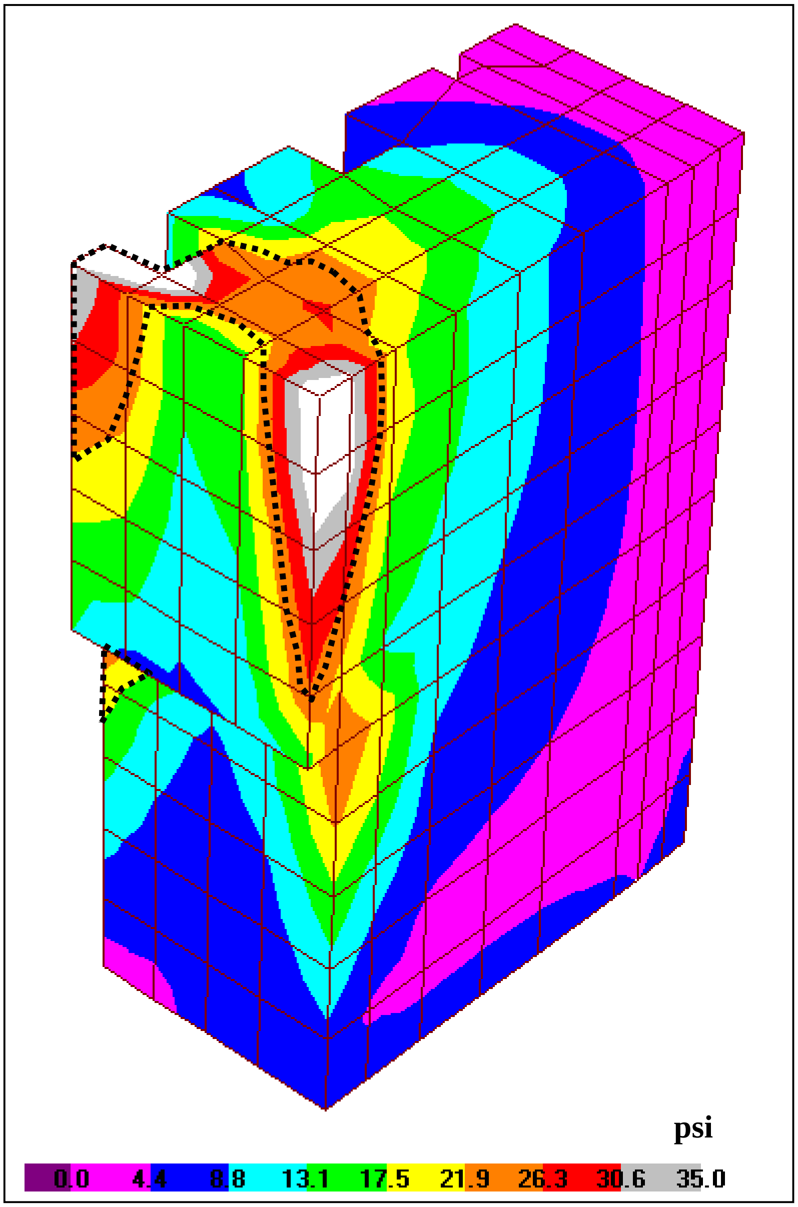
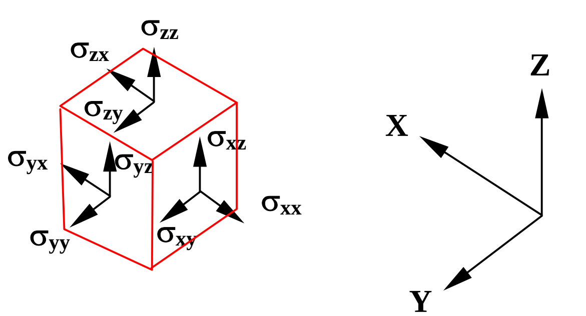


Figure 5-14b: Maximum out-of-plane shear stresses (σ_{xy}) on bottom half of the south wall due to static plus earthquake loads.



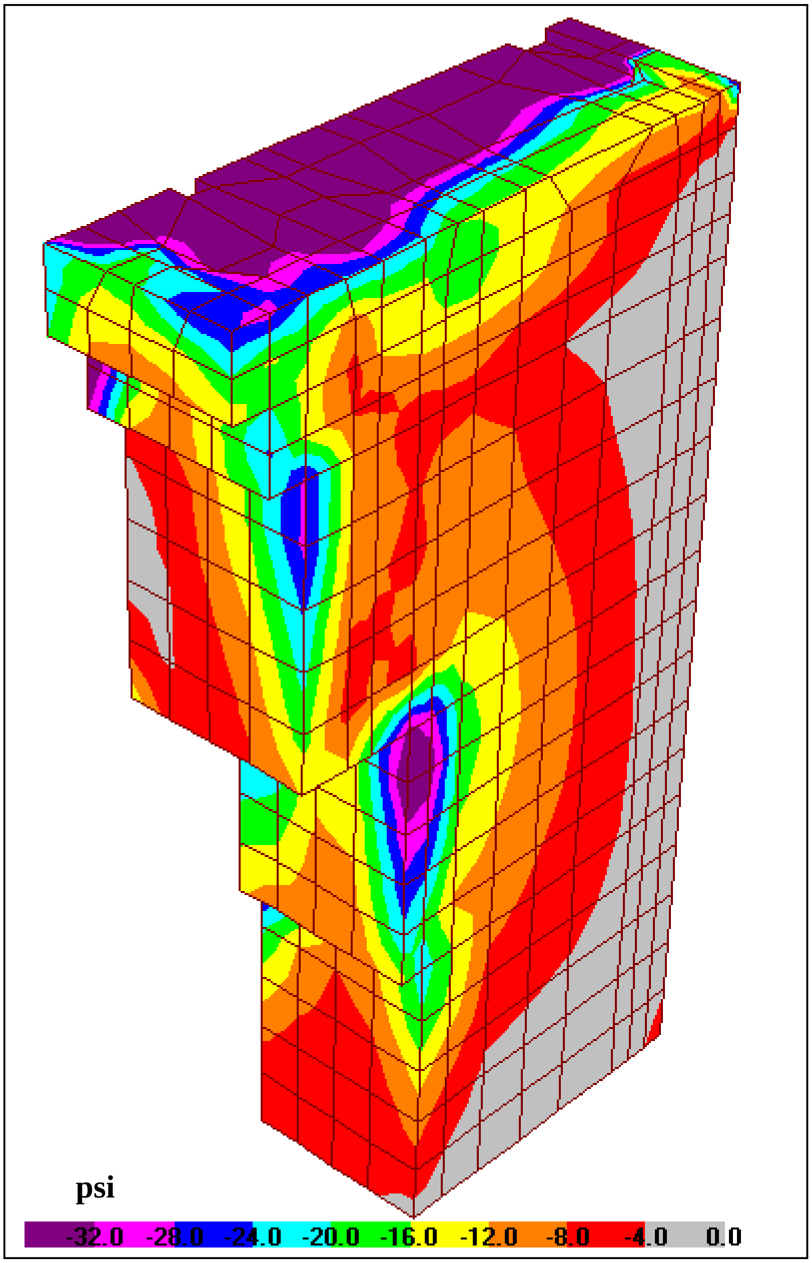
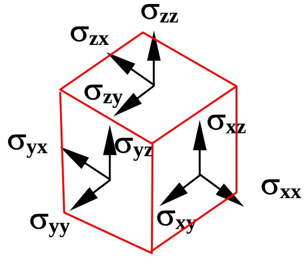
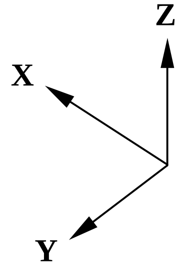


Figure 5-14c: Minimum out-of-plane shear stresses (σ_{xy}) for the south wall due to static minus earthquake loads





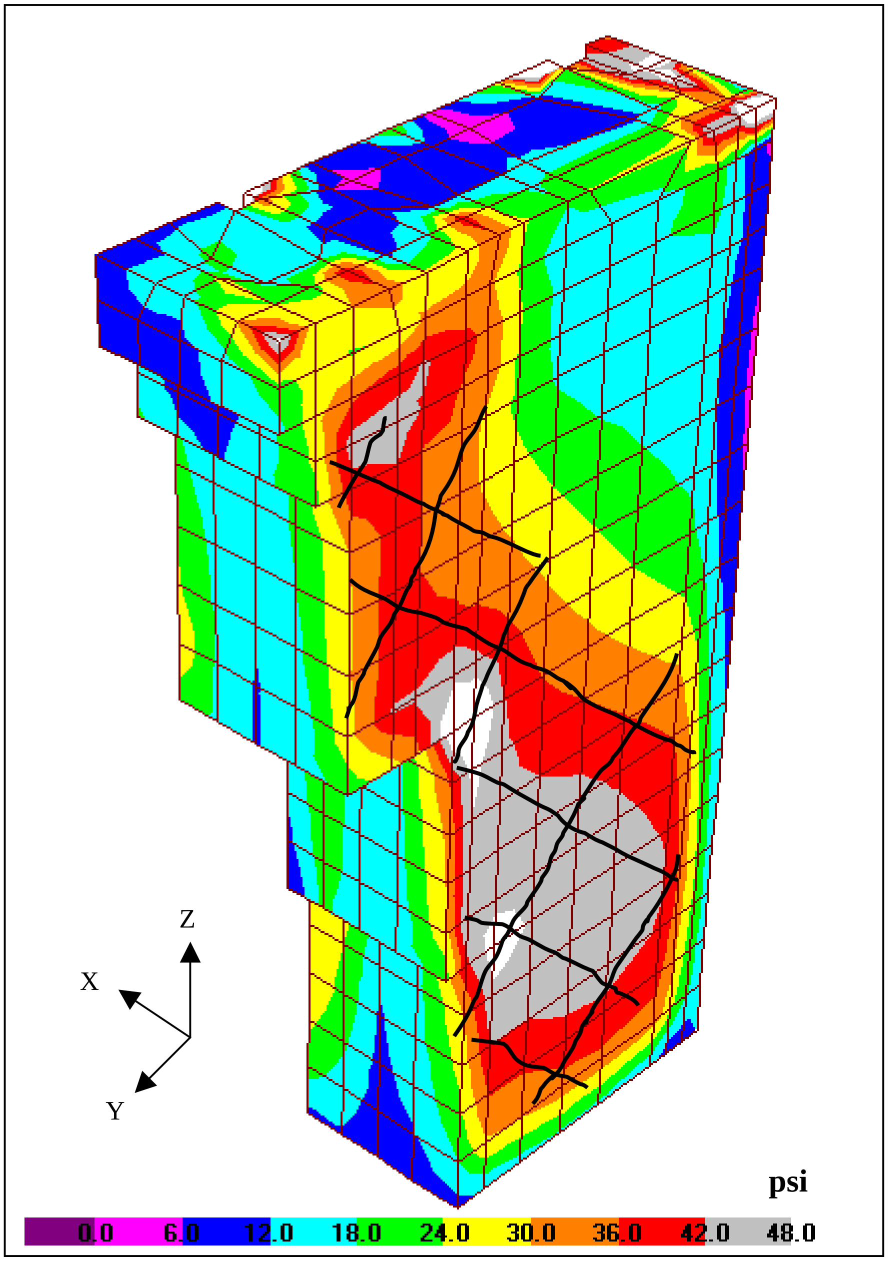
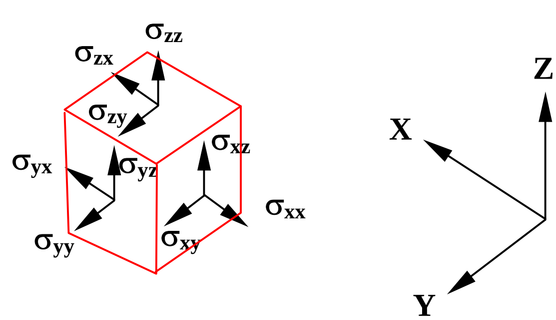


Figure 5-15a: Maximum in-plane shear stresses (σ_{yz}) in the south wall due to static plus earthquake loads



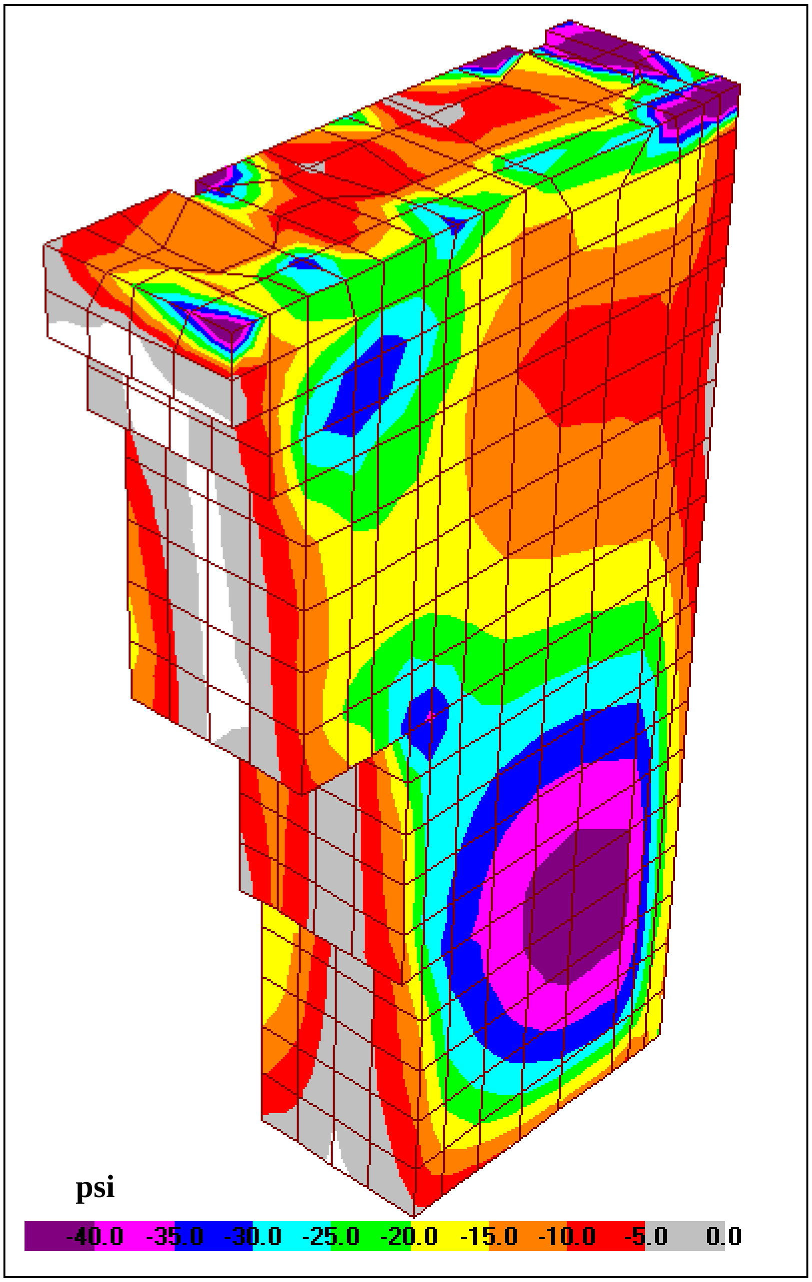
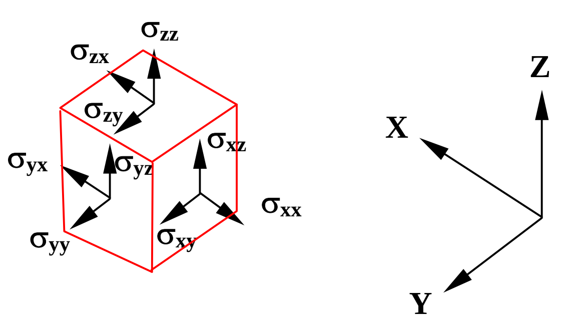


Figure 5-15b: Minimum in-plane shear stresses (σ_{yz}) in the south wall due to static minus earthquake loads



Stress Demand-Capacity Ratios

The maximum tensile, compressive, and shear stresses discussed above are now compared with the tensile, compressive, and shear strengths of the concrete and masonry in terms of demand-capacity ratios in Tables 5-2 to 5-4 below. Also included in these tables, when available, are the demand-capacity ratios computed by OLMM using code procedures. Both the 3D finite-element and simplified code calculations result in very high tensile and shear stress demand-capacity ratios, indicating that the masonry tower could suffer severe tensile and shear cracks leading to possible collapse of the tower. However, there are some differences between the two analyses that should be recognized. The main difference is that the code treats the tower as being cantilevered only at the base, thus producing much higher tensile stresses at the base of the tower than that predicted by the finite-element analysis. The finite-element element analysis, which accounts for the abutment support, distributes stresses along the height of the tower. Furthermore, the finite-element did not produce high compression stresses at the base of the tower as subjected by a DCR of 0.73 by the code calculations. Other differences are that the code calculations were based on one component of the ground motion and did not consider the added-mass of water. If these effects had been considered, the code calculations could have resulted in even higher stresses.

Table 5-2: Tensile stress demand-capacity ratios for the masonry wall

Material Type	Maximum Stress Location (ft)	Maximum Tensile Stress (psi)	Tensile Strength (psi)	DCR (Finite-element)	DCR (Code)**
Concrete	239	126	250	0.6	0.13
Brick	236	126	17.5	**7.2**	**3.07**
Dressed Stone	239	126	14	**9.0**	**7.91**
Stone	223	126	14	**9.0**	**86.81**
** The code values were obtained from the report by OLMM Consulting Engineers (Case-I embedment).					

Table 5-3: Compressive stress demand-capacity ratios for the masonry wall

Material Type	Maximum Stress Location (ft)	Maximum Compressive Stress (psi)	Compressive Strength (psi)	DCR (Finite-element)	DCR (Code)**
Concrete	239	160	2500	0.06	0.02
Brick	236	150	900	0.17	0.08
Dressed Stone	239	160	1800	0.09	0.08
Stone	223	170	1800	0.09	0.73
** The code values were obtained from the report by OLMM Consulting Engineers (Case-I embedment).					

Table 5-4: Shear stress demand-capacity ratios for the masonry wall

Material Type	Maximum Stress Location (ft)	Maximum Shear Stress (psi)	Shear Strength (psi)	DCR (FE)	DCR (Code)**
Concrete	239	40	100	0.4	0.15
Brick	236	45	21	**2.1**	0.81
Dressed Stone	239	45	31	**1.45**	0.70
Stone	223	48	31	**1.55**	**1.48**
** The code values were obtained from the report by OLMM Consulting Engineers (Case-I embedment).					

5.4.2 Pavilion Results for MDE

The earthquake performance evaluation of the pavilion structure is summarized in this section. The process involves comparison of the shear and moment capacities with the corresponding demands for critical members of the structure. The critical members include the beam connecting the masonry walls at the top, interior rectangular and "L" shape beams which make up the pavilion floor, the pavilion roof beams, the bridge connecting the pavilion roof to the abutment, and 18 hollow circular columns supporting the roof. Figure 5-16 shows the critical sections chosen for the beam connecting the two masonry walls. The force and moment demands at the end and mid sections of the beam are computed and compared with the shear and moment capacities estimated for the 2'x3' section with four 3/4-inch square bars on the top and four 3/4-inch bars on the bottom of the beam.

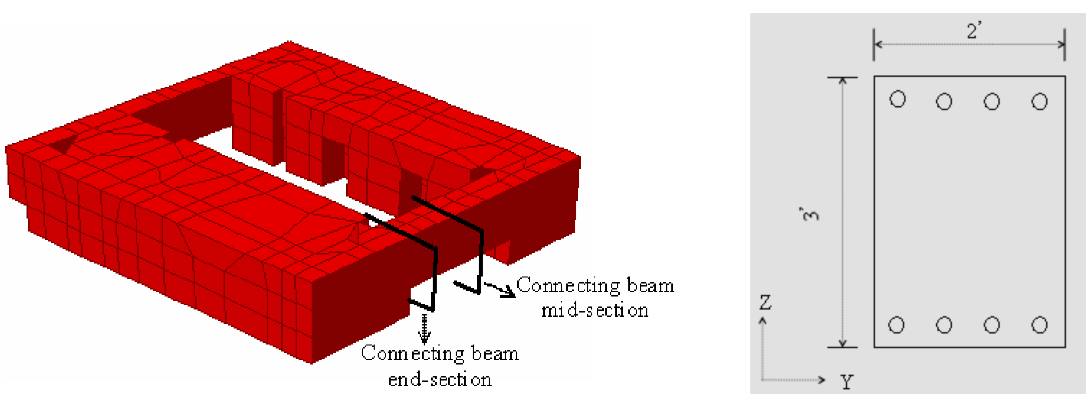


Figure 5-16: Sections chosen to assess the extent of damage for the connecting beam

The results for the pavilion are presented for two cases: 1) with the bridge connected to the pavilion roof (see Figure 5-2), and 2) with the bridge failed in shear and thus not connected to the roof. Tables 5-5 to 5-7 show the maximum shear forces and moments computed for the pavilion critical members with the bridge connected to the roof. Also listed in these tables are the force and moment capacities and the corresponding demand-capacity ratios. The finite-element results show that, with the bridge connected to the roof, the following members fail:

- Front "L" shape floor beam fails in shear
- Pavilion roof bridge fails in shear
- Beam connecting the tower walls fails in flexure
- 7 columns in front of the tower fail in flexure

Since the bridge fails in shear, a second finite-element model of the tower with no connection between the bridge and pavilion was analyzed to assess the performance after the bridge has been sheared off.

Tables 5-8 to 5-10 summarize the results for the model without the bridge. Since this condition is similar to the simplified analysis, which did not include the bridge, the results from the finite-element can directly be compared with those from the simplified analysis. The finite-element results indicate that in the absence of the bridge, all forces and moments increase, but the increase for the pavilion roof beams and columns is significantly greater. The moment DCR for the roof beams have increased from 0.22 to 1.72, indicating a possible flexural failure (see Tables 5-6 and 5-9). The shear demands on the columns have increased 3 to 12 times and the moment demands 3 to 18 times, with the peak values of the shear and moment DCR's reaching 2.70 and 6.21, respectively. At such high shear and moment demand-capacity ratios, all columns will probably fail, leading to a possible collapse of the pavilion structure. Furthermore, the collapse could be sudden due to high shear demands. Similar findings are reported by OLMM in Attachment II, which computed a moment DCR of 5.85 and a shear DCR of 1.23 for the columns.

Table 5-5: Shear demand-capacity ratios for critical sections of pavilion **with** bridge support

Member Type	V_{demand} (kips)	$V_{capacity}$ (kips)	V_d/V_c (F.E.)	V_d/V_c (Code)
Beam connecting walls	45	61.2	0.74	N/A
Interior rectangular floor beam	3	10.46	0.29	N/A
Front "L" shape floor beam	32	19.51	**1.64**	N/A
Back "L" shape floor beam	19	25.76	0.74	N/A
Roof Beam	6.6	51.01	0.13	N/A
Pavilion roof bridge	153	80.09	**1.91**	N/A

Table 5-6: Moment demand-capacity ratios for pavilion **with** bridge support

Member Type	Section Location	M_{demand} (kip-ft)	$M_{capacity}$ (kip-ft)	M_d/M_c (F.E.)	M_d/M_c (Code)
Beam connecting walls	Mid	13	71	0.18	N/A
	End	97	64	**1.52**	N/A
Interior rectangular floor beam	Mid	2	30	0.07	N/A
	End	8	23	0.35	N/A
Front “L” shape floor beam	Mid	6	32	0.19	N/A
	End	11	24	0.46	N/A
Back “L” shape floor beam	Mid	3	32	0.09	N/A
	End	7	40	0.18	N/A
Roof Beam	Mid	6	30	0.20	N/A
	End	7	32	0.22	N/A

Table 5-7: Demand-capacity ratios for the pavilion columns **with** bridge support

Column No.	Axial Force (kips)	Bi-axial Moment Demand (kip-ft)	Moment Capacity (kip-ft)	Moment DCR	Shear Demand (Kips)	Shear DCR
1	14	31.38	23.3	**1.35**	7.16	0.80
2	10	29.72	25	**1.19**	6.43	0.72
3	7	29.27	26.2	**1.12**	6.58	0.74
4	6	28.28	26.6	**1.06**	6.58	0.74
5	10	29.76	25	**1.19**	6.43	0.72
6	13	28.40	23.75	**1.20**	7.16	0.80
7	11	27.51	24.6	**1.12**	6.15	0.69
8	10	19.68	25	0.79	6.26	0.70
9	5	21.10	27	0.78	4.58	0.51
10	4	17.56	27.4	0.64	4.58	0.51
11	4	16.64	27.4	0.61	3.84	0.43
12	3	16.64	27.8	0.60	3.84	0.43
13	3	10.56	27.8	0.38	2.64	0.30
14	2	11.45	28.2	0.41	2.82	0.32
15	4	9.39	27.4	0.34	2.50	0.28
16	2	6.80	28.2	0.24	1.88	0.21
17	2	7.55	28.2	0.27	2.06	0.23
18	4	10.04	27.4	0.37	2.67	0.30

Table 5-8: Shear demand-capacity ratios for critical sections of pavilion **without** bridge support

Member Type	V_{demand} (kips)	$V_{capacity}$ (kips)	V_d/V_c (F.E.)	V_d/V_c (Code)
Beam connecting walls	47	61.2	0.77	N/A
Interior rectangular floor beam	3	10.46	0.29	0.42
Front "L" shape floor beam	35	19.51	**1.80**	N/A
Back "L" shape floor beam	21	25.76	0.82	N/A
Roof Beam	25	51.01	0.49	0.71
Pavilion roof bridge	--	80.09	--	0.92

Table 5-9: Moment demand-capacity ratios for the different sections of the pavilion **without** bridge support

Member Type	Section Location	M_{demand} (kip-ft)	$M_{capacity}$ (kip-ft)	M_d/M_c (F.E.)	M_d/M_c (Code)
Beam connecting walls	Mid	34	71	0.48	N/A
	End	108	64	**1.69**	N/A
Interior rectangular floor beam	Mid	2	30	0.07	0.41
	End	8	23	0.35	0.67
Front "L" shape floor beam	Mid	8	32	0.25	N/A
	End	14	24	0.58	N/A
Back "L" shape floor beam	Mid	5	32	0.16	N/A
	End	8	40	0.20	N/A
Roof Beam	Mid	8	30	0.27	**2.82**
	End	55	32	**1.72**	**3.96**

Table 5-10: Demand-capacity ratios for the pavilion columns **without** bridge support

Column No.	Axial Force (kips)	Bi-axial Moment Demand (kip-ft)	Moment Capacity (kip-ft)	Moment DCR	Shear Demand (Kips)	Shear DCR
1	31	91.97	15.5	**5.93**	20.09	**2.25**
2	19	89.20	21.3	**4.19**	19.48	**2.18**
3	8	85.70	25.8	**3.32**	18.75	**2.10**
4	7	85.98	26.2	**3.28**	18.73	**2.10**
5	19	88.53	21.3	**4.16**	19.35	**2.17**
6	32	91.29	15	**6.09**	20.03	**2.24**
7	19	90.61	21.3	**4.25**	19.88	**2.23**
8	19	90.69	21.3	**4.26**	19.82	**2.22**
9	5	93.38	27	**3.46**	19.94	**2.23**
10	5	92.02	27	**3.41**	19.87	**2.23**
11	4	94.85	27.4	**3.46**	20.22	**2.26**
12	4	93.51	27.4	**3.41**	20.16	**2.26**
13	18	104.48	21.7	**4.81**	22.70	**2.54**
14	18	99.25	21.7	**4.57**	21.67	**2.43**
15	27	108.71	17.5	**6.21**	23.51	**2.63**
16	7	102.94	26.2	**3.93**	22.43	**2.51**
17	6	110.00	26.6	**4.14**	24.08	**2.70**
18	25	103.59	18.5	**5.60**	22.62	**2.53**

5.4.3 Outlet Shaft Response to MDE

The 8-foot-diameter brick-lined outlet shaft behind the tower was not included in the finite-element model of the tower structure. However, the estimated abutment rock stresses in the vicinity of the shaft are quite small. As shown in Figure 5-17, the peak normal stresses in the bottom half of the outlet shaft are expected to be 1 to 3 psi and near the top of the shaft in the range of 8 to 12 psi. Note that the peak values of 8 to 12 psi are influenced by the rigid boundary of the abutment model. In reality, the actual stresses may be even smaller. Overall, such low stress levels are unlikely to induce damage in the outlet shaft and its brick liner, provided that the liner is maintained in good condition and free of deterioration.

The outlet shaft was inspected only briefly from the top during the site visit. The photograph on the right, taken from the top, shows steel rod and channel supports inside the outlet shaft. The sign of rusting in the steel rod coupling, rod bearings, and the channel supports is evident. It is therefore recommended that the outlet shaft be inspected and rusting damage be repaired before it can adversely affect operation of the tunnel gate at the bottom of the outlet.

5.5 RESPONSE TO MCE

As discussed in Section 3.6.3, in the period range of the tower structure, the MCE produces about 40% higher seismic loads than the MDE. Accordingly, section forces and element stresses for the MCE will be 40% higher than those computed for the MDE. Therefore, the MCE undoubtedly causes more severe damage and a higher probability of collapse than the MDE.

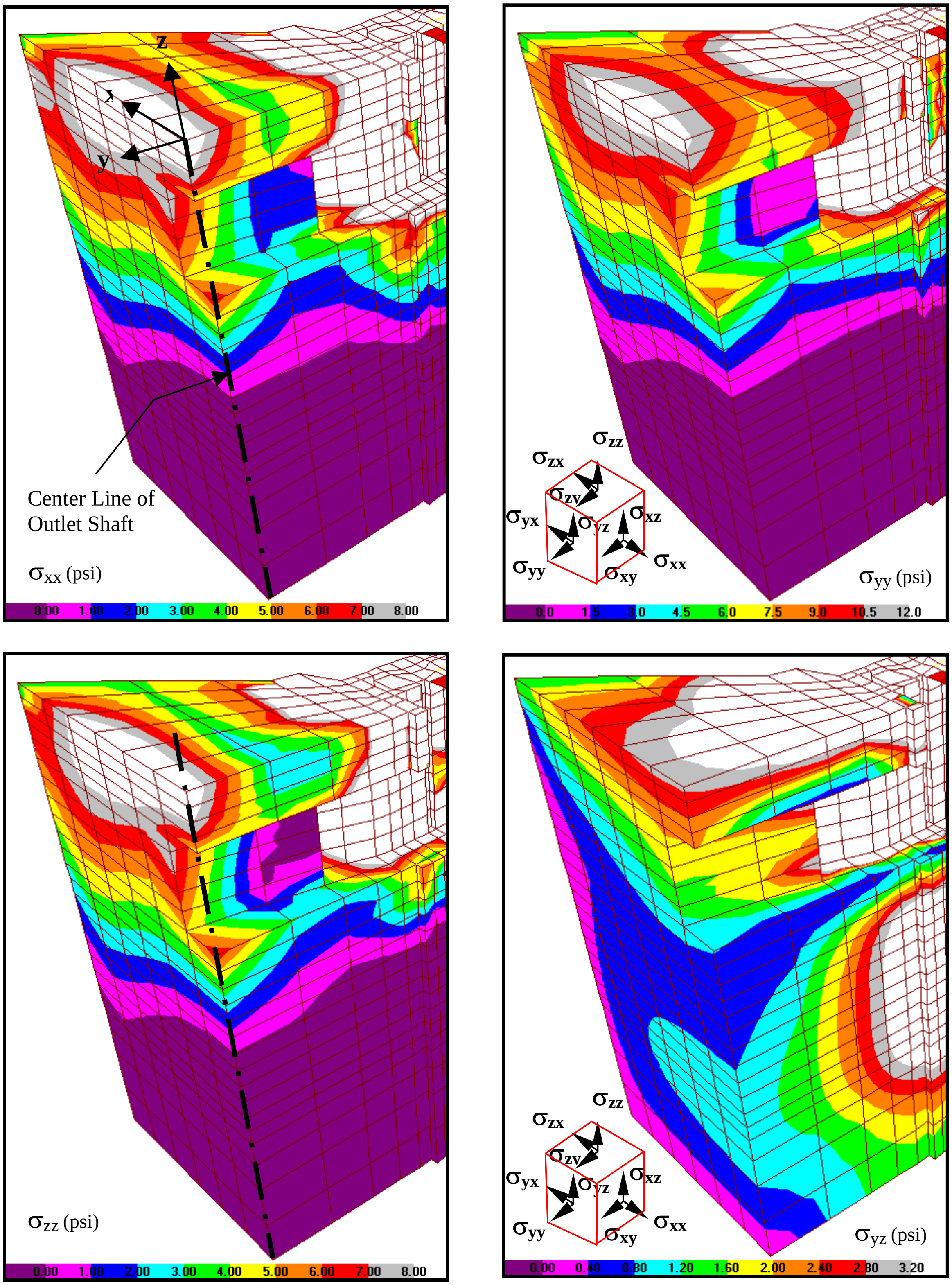


Figure 5-17: Normal and shear stresses in the vicinity of the outlet shaft

6. FINDINGS AND CONCLUSIONS

The results of the simplified and three-dimensional linear-elastic analyses indicate that both the reinforced-concrete pavilion and the masonry tower will suffer severe damage. In the event of a major earthquake with ground motions at the level of the MDE, the reinforced-concrete pavilion probably will collapse and the masonry tower is likely to suffer extensive cracking. The cracking could lead to separation of the tower from the abutment rock and formation of disjointed blocks that could fall with the collapse of pavilion. The situation will be even worse in the event of a postulated MCE on the nearby Hayward Fault that is capable of producing 40% larger seismic forces than the MDE. The valve shafts or shaft supports could be damaged causing accidental blockage of the sluice valves and thus blocking release of water from the reservoir.

The above findings are supported by the high demand-capacity ratios as discussed below. The moment DCR for pavilion roof beam is 1.72 and the shear DCR for the pavilion floor beam reaches 1.8, an indication that the floor beams could fail in shear. The pavilion columns exhibit demand-capacity ratios as high as 2.7 in shear and 6.21 in moment. Again such high DCR's, especially in shear, suggest that the pavilion will probably collapse.

The masonry tower will be subjected to tensile and shear stresses well beyond its capacities. The maximum tensile and shear demand-capacity ratios for the MDE are 9 (Table5-2) and 2.1 (Table 5-4), respectively. Major tensile cracks will develop at the contact with the abutment and could potentially separate the tower from its abutment support. Shear stresses are also quite high. While the out-of-plane shear stresses affect mostly the abutment contact regions, the in-plane shear stresses cover about 75% of the wall surfaces and could produce significant diagonal (stepped) cracks. Although the tower may not collapse, the extensive tensile and shear cracks are likely to turn the masonry tower into a disjointed structure with diminished lateral load resistance capabilities.

The estimated abutment stresses indicate that the 8-foot-diameter outlet shaft behind the tower would survive the MDE and MCE shaking, provided that the outlet is inspected to ensure that the brick liner is in good condition and that the gate operating steel gear has not corroded. A deteriorated brick liner could suffer damage in a major earthquake and the resulting earthquake debris could potentially block the outlet works at the tunnel entrance.

7. RECOMMENDATIONS FOR RETROFIT

Based on the results of this study no further structural analysis or material testing is recommended. This is because computed seismic demands are very high and the structure fails in brittle modes. Consequently, no nonlinear behavior is permitted and the materials should possess strengths as high as the computed demands. In fact, the periods of vibration of the masonry tower fall in the ascending portion of the earthquake response spectra, an indication that seismic forces increase with the nonlinear behavior.

However, depending on the operational needs and potential impacts on the release of water from the reservoir, additional efforts should be focused on retrofitting the structure to assure it will remain functional in the event of a major earthquake. In this preliminary stage, the following options are presented:

1) The pavilion structure offers no structural function other than perhaps sheltering the valve operators and facilitating operation of the stop timbers. Strengthening the pavilion structure appears to be an expensive undertaking. We recommend removing the pavilion to eliminate the possibly of the pavilion collapsing on top of the masonry tower, rather than bearing high expenses to fix it. If a platform structure is still desired, a light steel frame structure may be designed as the replacement. It should be noted that the absence of the pavilion will not change the seismic stress conditions of the masonry tower in any significant way. With the pavilion removed, one of the following retrofit options may be considered and evaluated for the masonry tower.

2) Do not fix the masonry tower. Instead remove the sluice gates entirely or only the valve shafts but fix the valves in open position to prevent accidental blockage of the sluice valves. In this case, the outflow from the reservoir can be controlled by the sluice gate at the entrance to Tunnel #2. However, the brick liner and gate operating steel gear in the outlet shaft should be inspected and if necessary repaired to ensure that the 36" sluice valve will remain operational and that the tunnel is not blocked at the entrance by the earthquake debris. To preclude such blockage, the 30" lower inlet pipe could be connected to the tunnel. It should also be noted that the mid-level inlet may still be blocked by debris, but the lower inlet made of a 30" extra strong pipe should remain open.

3) Strengthen the masonry tower to stabilize and maintain its structural integrity. One way to accomplish this is to anchor the masonry tower to the foundation and abutment rock using external anchors. The anchors should be designed to minimize cracking but more importantly to hold the masonry together and connected to the foundation and abutment. In addition to strengthening the tower, the outlet works in the back of the tower should be inspected and repaired as discussed above. This and other retrofit concepts need to be developed and evaluated on the basis of constructability and cost.

8. REFERENCES

URS Corporation (2004a) “Site-Specific Earthquake Ground Motions, Task D Technical Memorandum, Dynamic Stability of Chabot Dam,” April 12, 2004.

URS Corporation (2004b) “Probabilistic Ground Motion Analysis, Task K Technical Memorandum, Dynamic Stability of Chabot Dam,” June 14, 2004.

USACE (2003) “ US Army Corps of Engineers’ Engineer Manual EM 1110-2-2400: Structural Design and Evaluation of Outlet Works,” June 2, 2003.

FEMA 356 (2000) “Pre-standard and Commentary for the Seismic Rehabilitation of Buildings.”

James S.-H. Kuo (1982) “Fluid-Structure Interactions: Added Mass Computations for Incompressible Fluid,” Report No. UCB/EERC-82/09, Earthquake Engineering Research Center, August 1982.

ATTACHMENT - I

MATERIAL PROPERTIES AND CONDITION ASSESSMENT

for

INTAKE TOWER FOR TUNNEL #2
CHABOT RESERVOIR

for
URS Corporation

May 26, 2004
Revised December 22, 2004

Prepared by

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TABLE OF CONTENTS

SYNOPSIS

This memorandum establishes material properties for the intake tower at tunnel #2 at the Chabot Reservoir.

The intake tower is on the west shore of Lake Chabot. It is a stone and brick tower capped with a pavilion. The pavilion roof slab is reinforced concrete and is supported on reinforced concrete perimeter beams. These beams in turn are supported on 18 concrete columns. The columns and slotted reinforced concrete floor, rest on about four and one half feet of concrete above the masonry tower. The tower plan dimension is roughly 20’-0” square (the tower is trapezoidal). The height of the tower from the pavilion floor to the top of its footing is roughly 45’-0”.

The following material property values are based on default values found in FEMA 356 and based on a visual assessment of the structure. The rusting and spalling found at the roof structure will worsen rapidly with time and should be repaired.

SUMMARY OF MATERIAL PROPERTIES

Material (density)	Location	Property	Lower Bound (psi)	Expected Strength (psi)
Concrete (150 pcf)	Roof beams and slab	Compressive Strength	1875	2812
		Reinforcing Tensile Strength (& Yield)	41,250 (24,750)	51,560 (30,938)
		E_c	2,850,000	2,850,000
	Columns, floor slab, and beams	Compressive Strength	2500	3750
		Reinforcing Tensile Strength (& Yield)	55,000 (33,000)	68,750 (41,250)
		E_c	2,850,000	2,850,000
Brick (120 pcf)	Throughout	Compressive Strength	900	1170
		Tensile Strength	20	26
		Shear Strength	27	35
		Elastic Modulus	643,500	643,500
		Shear Modulus	257,400	257,400
Stone (160 pcf)	Throughout	Compressive Strength	1800	2340
		Tensile Strength	20	26
		Shear Strength	54	70
		Elastic Modulus	1,287,000	1,287,000
		Shear Modulus	514,800	514,800

STRUCTURAL DESCRIPTION OF CHABOT DAM INTAKE TOWER OF TUNNEL #2

The structure of the intake tower of Tunnel #2 at the Chabot Reservoir is embedded in the rocky shore and masonry retaining walls and revetments along the west shore of Lake Chabot.

The primarily stone tower is trapezoidal in plan with the narrow face on the lake side. Originally it rose about 40'-6" above its lowest intake pipe as two separate segments with an 8'-0" gap for water intake. On the shore there is dressed stone embedded into the hill side about 17'-0" high and 21'-11" horizontally (north-south). This dressed stone begins about 4'-6" above the top of the original tower and extends around the waste tunnel. The tower dimensions at the top are 17'-0" from face of dressed stone to the east. The east face is 20'-8" including the 8'-0" water-gap.

An 8'-0" inside diameter well is on the hillside behind the tower. The well has 13" thick brick walls. This well was capped and fitted with a slide gate in 1938. The top of the concrete cap is approximately 12'-9" above the top of the dressed stone.

In 1923 to 1924 a concrete pavilion was added above the existing tower. The floor of the pavilion was set at the top of the dressed stone and incorporated the dressed stone in supporting six of the roof columns. The floor is slotted for access to gate shafts and log ways. Concrete beams run parallel to these shafts and pick up the concrete slab. Sets of round concrete columns rise 10'-0" above the floor. There are four sets of three columns in each corner and pairs of columns on the north, south and east face. A flat concrete slab frames to perimeter beams running over the columns to form the roof. There is a concrete parapet above the roof. The roof is 23'-0" on each side.

The materials in the tower are as follows:

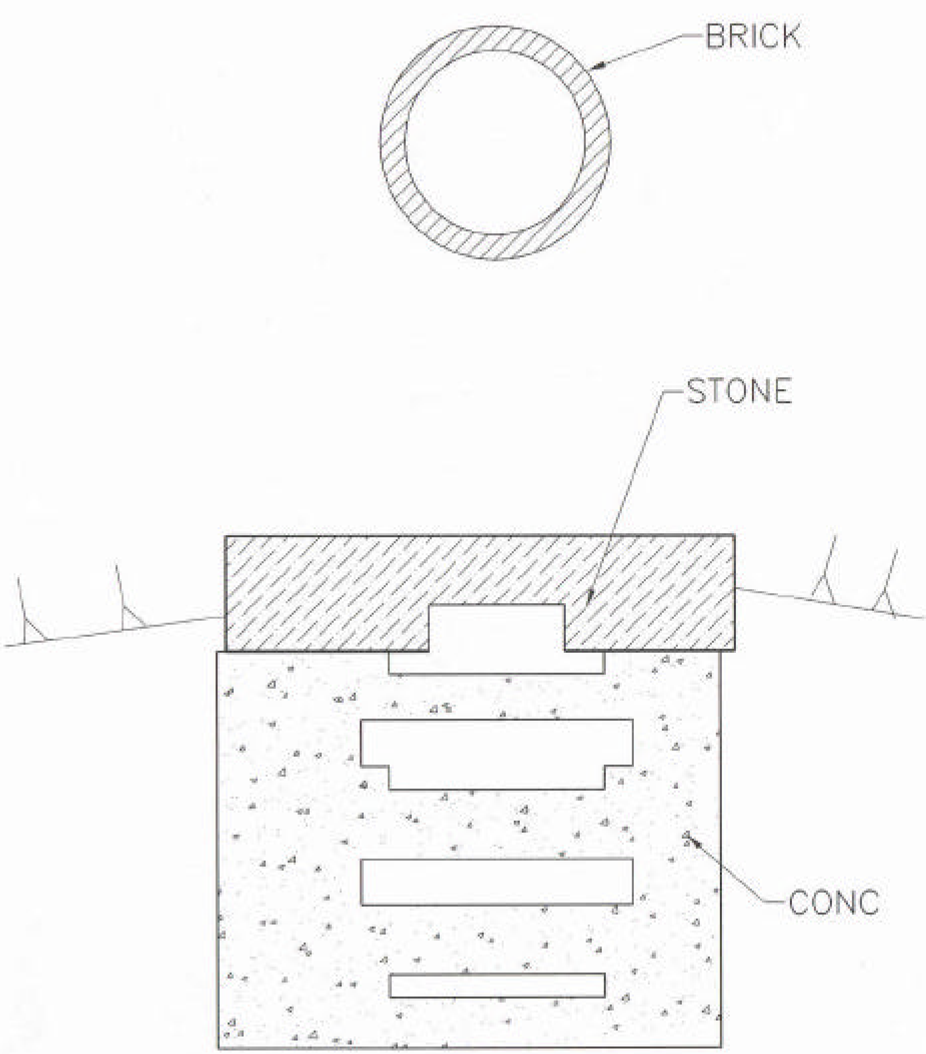
- 4'-6" of concrete from the floor slab to the top of brick.
- 18" of brick to the top of dressed stone.
- 3'-3" of dressed stone to the top of stone masonry which extends to the base of of the tower.
- Stone masonry retaining begins about 9" below the bottom of dressed stone on the north face and continues to fan out in a step wise manner.
- Along the water inlet brick is used to form the slots for the valve shafts and eastern log ways.

Please refer to attached sketches for approximate material layout of tower.

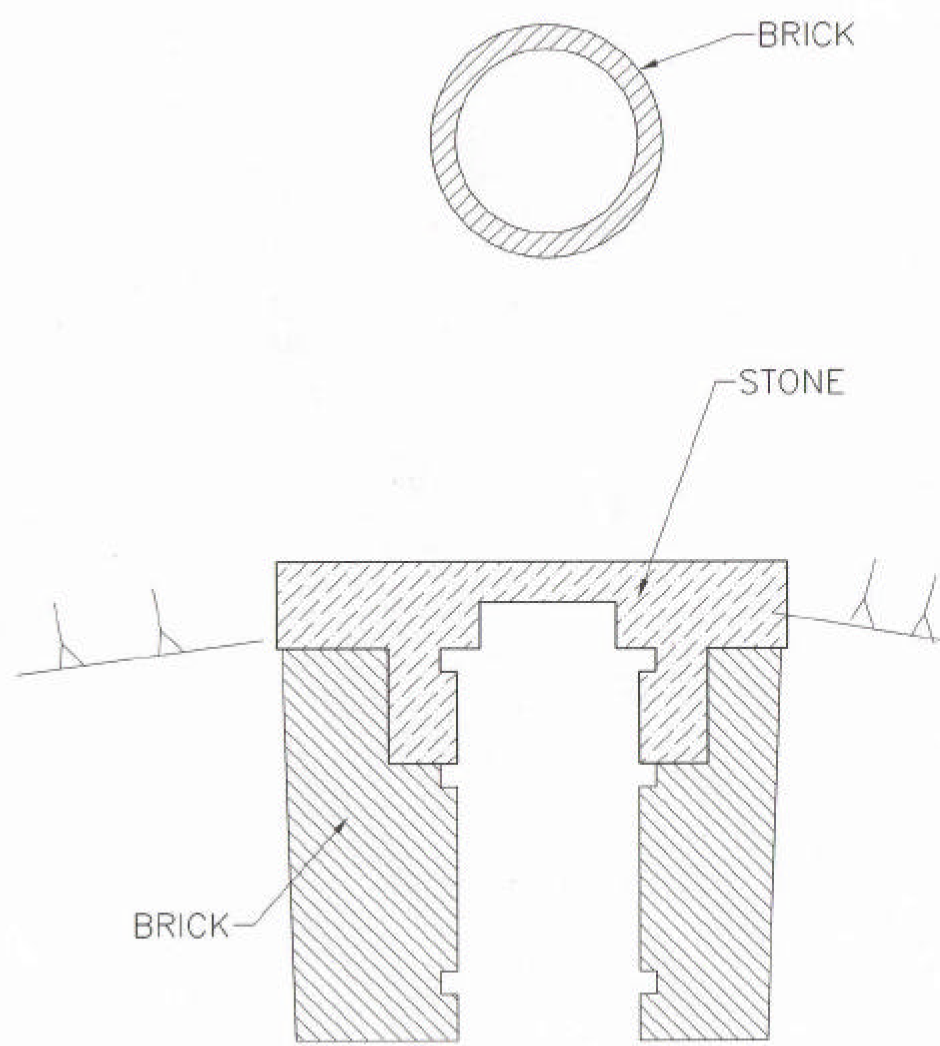
The information used in this report is based on the drawings listed on the following table; five photographs taken in 1924 (3 reproduced here), and two field visits on 5/7/04 on land and 5/18/04 by boat.

SOURCES OF INFORMATION

DWG	DATE	DESCRIPTION
5803-G-1	1969	Not Relevant to This Project
5803-G-2	1969	Not Relevant to This Project
5803-G-3	1969	Not Relevant to This Project
5803-G-4	1969	Not Relevant to This Project
E1107	1922	Plan and section of HEADWORKS TOWER NO 2 and Gate tower – prior to concrete addition – no valve or cover on manhole (indicated as a well).
1474R(i)	1922	Site Plans
1474R(ii)	1922	Site Plans
D1101	1923	Shaft Extension
D1103	1923	Reinforcing Bar Bends
E1102	1923	Elevation Of Structure At Intake Tower #2 W/ Dims And Reinf.
E1103	1923	Plans At Floor And Ceiling Of Structure @ Intake Tower #2 W/ Sections
E1104	1923	Elevations Of Structure @ Intake Tower #2
709G	1938	Concrete Plug & Slide Gate @ Chabot Tunnel #2
1342G	1940	Plan & Profile Of Tunnel #2
9480-G-1	1991	Section/Elevation Chabot Outlet Tower #2
9480-G-2	1991	Floor Plan @ Outlet Tower #2
9480-G-3 to G-5	1991	Tower Screen Details



EL 241.6 TO 237.1

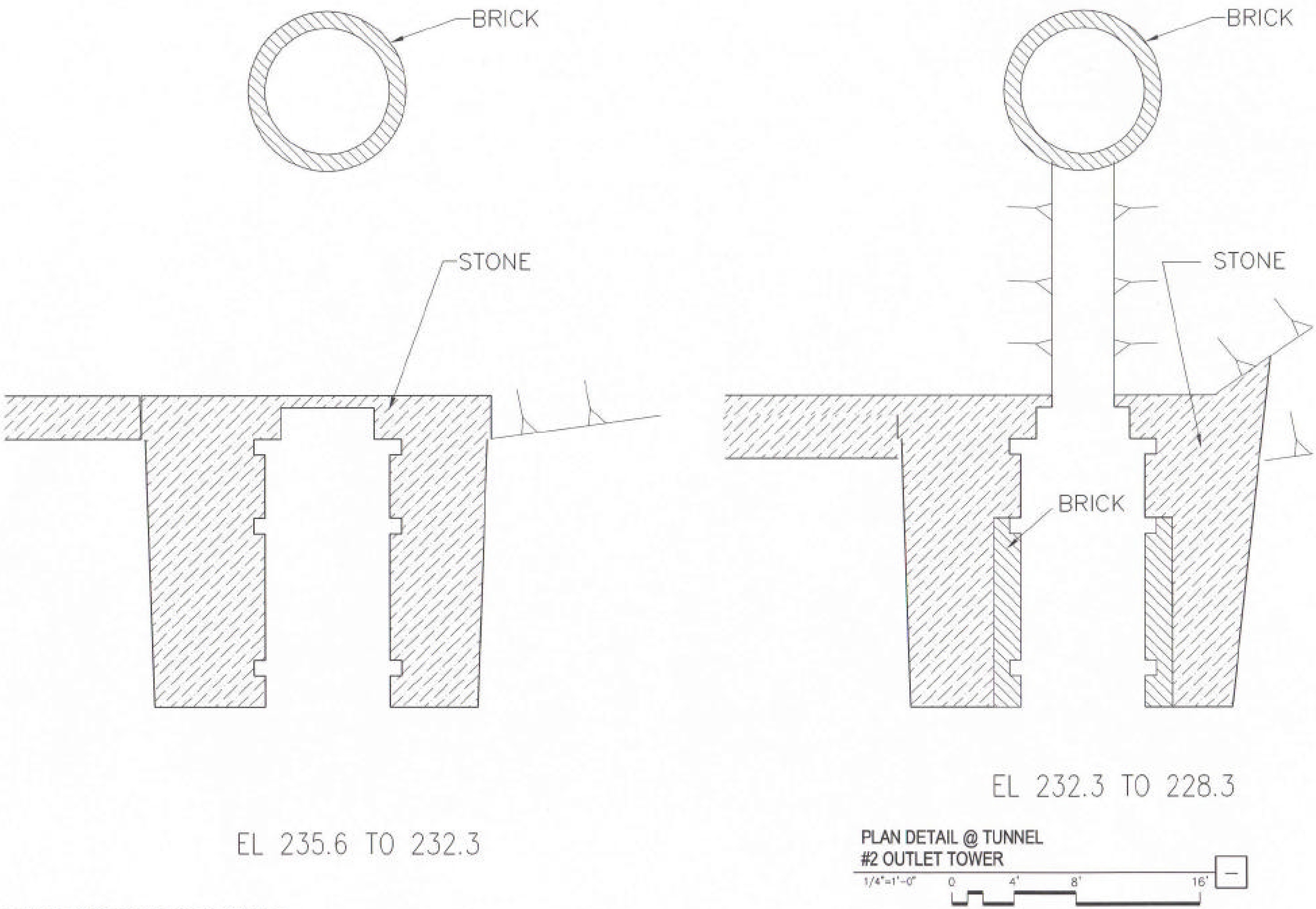


EL 237.1 TO 235.6

PLAN DETAIL @ TUNNEL
#2 OUTLET TOWER

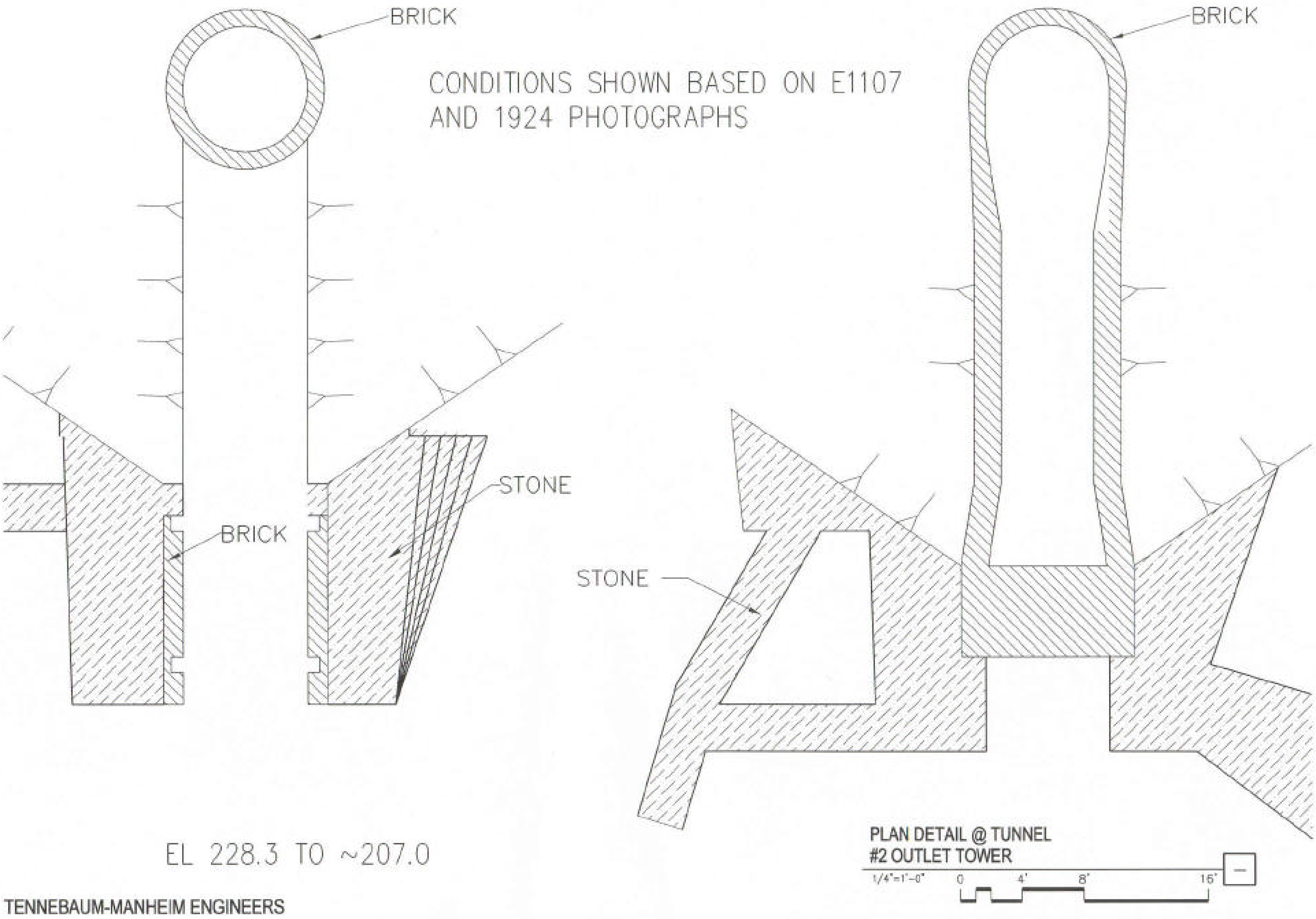
1/4"=1'-0" 0 4' 8' 16'

TENNEBAUM-MANHEIM ENGINEERS



PLAN DETAIL @ TUNNEL
#2 OUTLET TOWER

TENNEBAUM-MANHEIM ENGINEERS



TENNEBAUM-MANHEIM ENGINEERS

CONDITION ASSESSMENT

Overall the structure appears in good condition. It should be noted that the shaft for the 20” sluice gate appears to be dislodged. This condition is barely discernible on the right hand side of photo on page 22.

Concrete

The concrete was assessed in accordance with FEMA 356-6.3.3.2.1.

The pavilion roof shows signs of spalling and rust jacking (see photos) at both slab and at west side beam. The remainder of the concrete appeared to be in good condition except for some rock pockets below the floor. These showed evidence of lack of vibration. Further these pockets indicated the use of river gravel as aggregate. Finally, the rock pockets demonstrated to the good quality of the cement paste.

Based on these assessments we propose to assign a knowledge factor of 0.75 to the pavilion roof structure and 1.0 below.

It should be noted that the deterioration at the roof should be repaired as soon as possible as rusting and spalling damage tends to accelerate.

Masonry

The masonry was assessed in accordance with FEMA 356-7.3.3.1. Under water masonry was not examined.

In general, all the masonry appears to be in very good condition. The masonry below the high water line is plastered. Joints and beds in stone masonry are 3/8” qualifying as ashlar.

A brick was missing on the north face of the tower indicating that mortar coverage in the collar joint was at least 85% (see photo).

MATERIAL PROPERTIES

MASONRY

FEMA 356 does not address stone masonry. Traditionally (and in current codes empirically) allowable compressive stress and Young's Modulus for stone (ashlar) masonry have been taken as twice that of brick (See attached copies of code values). The Young's Modulus is also the theoretically affected by the number of joints per foot. Without knowledge of the relative stiffness of mortar vs. brick and stone, this factor cannot be evaluated. However, this factor is usually less than 5%. As indicated, all the masonry was in good condition.

	Brick		Stone	
	Default & Lower Bound (psi)	Expected Strength (psi)	Lower Bound (psi) *	Expected Strength (psi)
Compressive Strength	900	1170	1800	2340
Tensile Strength	20	26	20	26**
Shear Strength	27	35	54	70
Elastic Modulus	643,500	643,500	1,287,000	1,287,000
Shear Modulus	257,400	257,400	514,800	514,800

<u>Brick Density:</u>
120 pcf

<u>Stone Density:</u>
160 pcf

* See discussion ** Based on mortar only

CONCRETE

All concrete density = 150pcf

	Default Value (psi)	K	Lower Bound Strength (psi)	Expected Strength (psi)
Compressive Strength of concrete at pavilion roof slab and beams	2500	.75	1875	2812
Reinforcing Tensile Strength (& yield strength) at pavilion roof slab and beams	55,000 (33,000)	.75	41,250 (24,750)	51,560 (30,938)
Compressive Concrete Strength Remainder	2500	1.0	2500	3750
Reinforcing Tensile Strength (& yield strength) Remainder	55,000 (33,000)	1.0	55,000 (33,000)	68,750 (41,250)
Young's Modulus E_c	2,850,000	-	2,850,000	2,850,000

For appropriate stiffness see Table 6.5

If the shearing stress occurs without diagonal tension, the working shearing unit stress may be taken at 4/10 of the working compressive unit stress, but with detached blocks and slabs of stone shear is almost always accompanied by diagonal tension.

Tension, whether direct or flexural, should not occur in stone masonry, and working tensile unit stress should be taken as zero in the computations of designing. In the analysis of existing structures, where the mortar is found to be strong and adhesive, a tensile unit stress of 15 lb. per sq. in. may be allowed for masonry laid in portland cement mortar, 10 lb. per sq. in. for that in natural cement mortar and 5 lb. per sq. in. for that in lime mortar.

The resistance of masonry joints against tension is often wholly or partially destroyed by erection stresses, by shrinkage of the mortar in setting, and by expansion and contraction of the mass under changes of temperature. The mortar in the vertical joints, as a rule, wholly loses its adhesion from these causes. The mortar of the bed joints, however, sets under pressure and hence is more or less available to transmit tension. Since the shrinkage of mortar is less below ground, due to ground moisture and a uniform temperature, and since expansion and contraction are also less, masonry below ground is stronger in tension than that in air.

When it is necessary to build masonry to take tension, either direct or flexural, special care should be given to the bond. In the design of concrete footings a tensile unit stress not to exceed 8% of the safe compressive unit stresses may be allowed.

Allowed Working Compressive Stresses in Pounds per Square Inch, for Masonry, According to Building Laws

Kind of masonry	New York 1925	Philadelphia 1927	Baltimore 1924	Washington 1925	Buffalo 1926	Detroit 1926	Kansas City 1927	New Orleans 1927	Denver 1926	San Francisco 1926
Ashlar (portland cement mortar):										
Granite	600	.....	1000	800	.....	.....	600	600	800	389
Limestone	600	.....	600	500	.....	.....	600	600	500	.....
Sandstone	300	.....	400	500	.....	.....	300	300	400	.....
Rubble masonry:										
Portland cement mortar	140	139	125	.....	125	139	170	140	140	.....
Natural cement mortar	110	.....	.....	.....	.....	.....	.....	.....	100	.....
Lime-cement mortar	100	111	100	.....	97	97	120	105	100	.....
Lime mortar	.....	70	60	.....	70	70	70	70	70	.....
Brickwork:										
Portland cement mortar	250	208	150	170	250	208	225	175–450	170	208
Natural cement mortar	210	.....	.....	.....	.....	.....	175	.....	130	.....
Lime-cement mortar	160	167	.....	130	195	153	175	130–340	.....	139
Lime mortar	110	111	110	.....	125	97	120	90–225	90	97
Plain concrete: (1 : 2 : 4; portland cement)	500	208	500	400	348	111	500	By test	400	278

In design of structures to be built in cities, the unit stresses permitted by the building codes must be considered.

17. Unit Weights and Other Constants

Unit Weights of Masonry are slightly less than those of the materials of which it is composed. Average values are given in the accompanying table; the third column also gives the approximate moduli of elasticity and the fourth the coefficients of expansion for one degree Fahrenheit (based on using cement or cement-lime mortar).

Physical Properties of Masonry

Kind of masonry	Weight, lb. per cu. ft.	Modulus E, lb. per sq. in.	Coefficient e of expansion
Ashlar: granite, syenite, gneiss	165	4 000 000	0.000 0035
Limestone, marble	160	4 000 000	0.000 0035
Sandstone	140	4 000 000	0.000 0035
Mortar rubble: Granite, syenite, gneiss	155	2 000 000	0.000 0035
Limestone	150	2 000 000	0.000 0035
Sandstone	130	2 000 000	0.000 0035
Dry rubble: Granite, syenite, gneiss	130	....	....
Limestone	125	....	....
Sandstone	110	....	....
Brick: Pressed, thin joints	140	2 000 000	0.000 0030
Common, 3/8-in. joints	120	2 000 000	0.000 0030
Soft, 3/8-in. joints	100	....	....
Concrete: Broken stone, 1 : 2 : 4	145	2 500 000	0.000 0060
Broken stone, 1 : 3 : 6	145	2 000 000	0.000 0060
Cinder	110	....	....
Cyclopean: Masonry with maximum volume of stone	155	....	....

Slabs or detached block stone have values of E much higher than those in the table. Approximate values are 7 000 000 for granite, syenite and gneiss, and the harder limestones; 8 000 000 for hard marble; 5 500 000 for soft limestone; 2 800 000 for sandstone. Coefficients of expansion are 0.000 0040 for the granitic rocks, 0.000 0037 for limestone and 0.000 0050 for sandstone.

Values in the last two columns are used in investigating the temperature stresses which may come upon masonry arches (Art. 40) and for computing their deformations.

Weights of Other Materials which may bring pressure upon masonry walls and arches are given in the next table, in pounds per cubic foot.

Weights of Miscellaneous Materials

Sand, dry clean	90	Cinders, bituminous, dry compact	45
Sand, wet	115	Ashes, anthracite, dry compact	30
Gravel, clean	100	Paving in place:	
Broken stone	100	Asphalt top and binder	107
Clay, dry, compact	100	Asphalt block	145
Clay, plastic	100	Granite block	155
Sand, gravel, and clay, mixed:		Wooden block	50
Dry, compact	100	Brick	140
Wet	115	Macadam	105
Mud	110	Water, fresh	62.5
Rock, rotten, soft compact	110	Water, salt	64
Rock, hard, loose	100	Snow, fresh	8

The weight of snow to be used in designing is generally assigned by specification, as is also the lateral wind pressure, the latter being usually 30 lb. per sq. ft. of vertical surface.

The Slope of Repose of a bank of loose earth, in its natural state, is a factor which governs the lateral pressure which the earth may bring to exert against

Merrian, T. and Wiggin, T.H., Civil Engineer's Handbook, 5th Edition, John Wiley and Sons Inc, New York, 1942.

TABLE 21-M—ALLOWABLE COMPRESSIVE STRESSES FOR EMPIRICAL DESIGN OF MASONRY

CONSTRUCTION: COMPRESSIVE STRENGTH OF UNIT, GROSS AREA × 6.89 for kPa	ALLOWABLE COMPRESSIVE STRESSES[1] GROSS CROSS-SECTIONAL AREA (psi) × 6.89 for kPa	
	Type M or S Mortar	Type N Mortar
Solid masonry of brick and other solid units of clay or shale; sand-lime or concrete brick:		
8,000 plus, psi	350	300
4,500 psi	225	200
2,500 psi	160	140
1,500 psi	115	100
Grouted masonry, of clay or shale; sand-lime or concrete:		
4,500 plus, psi	275	200
2,500 psi	215	140
1,500 psi	175	100
Solid masonry of solid concrete masonry units:		
3,000 plus, psi	225	200
2,000 psi	160	140
1,200 psi	115	100
Masonry of hollow load-bearing units:		
2,000 plus, psi	140	120
1,500 psi	115	100
1,000 psi	75	70
700 psi	60	55
Hollow walls (cavity or masonry bonded)[2] solid units:		
2,500 plus, psi	160	140
1,500 psi	115	100
Hollow units	75	70
Stone ashlar masonry:		
Granite	720	640
Limestone or marble	450	400
Sandstone or cast stone	360	320
Rubble stone masonry Coarse, rough or random	120	100
Unburned clay masonry	30	—

[1]Linear interpolation may be used for determining allowable stresses for masonry units having compressive strengths which are intermediate between those given in the table.

[2]Where floor and roof loads are carried upon one wythe, the gross cross-sectional area is that of the wythe under load. If both wythes are loaded, the gross cross-sectional area is that of the wall minus the area of the cavity between the wythes.

TABLE 21-N—ALLOWABLE SHEAR ON BOLTS FOR EMPIRICALLY DESIGNED MASONRY EXCEPT UNBURNED CLAY UNITS

DIAMETER BOLT (inches)	EMBEDMENT[1] (inches)	SOLID MASONRY (shear in pounds)	GROUTED MASONRY (shear in pounds)
× 25.4 for mm		× 4.45 for N	
$^1/_2$	4	350	550
$^5/_8$	4	500	750
$^3/_4$	5	750	1,100
$^7/_8$	6	1,000	1,500
1	7	1,250	1,850[2]
$1^1/_8$	8	1,500	2,250[2]

[1]An additional 2 inches of embedment shall be provided for anchor bolts located in the top of columns for buildings located in Seismic Zones 2, 3 and 4.

[2]Permitted only with not less than 2,500 pounds per square inch (17.24 MPa) units.

7.3.2.10 Default Properties

Use of default material properties to determine component strengths shall be permitted with the linear analysis procedures in Chapter 3.

Default lower-bound values for masonry compressive strength, elastic modulus in compression, flexural tensile strength, and masonry shear strength shall be based on Table 7-1. Default expected strength values for masonry compressive strength, elastic modulus in compression, flexural tensile strength, and masonry shear strength shall be determined by multiplying lower-bound values by an appropriate factor taken from Table 7-2.

Default lower-bound and expected strength yield stress values for reinforcing bars shall be determined in accordance with Section 6.3.2.5.

Table 7-1 Default Lower-Bound Masonry Properties

	Masonry Condition[1]		
Property	**Good**	**Fair**	**Poor**
Compressive Strength (f'_m)	900 psi	600 psi	300 psi
Elastic Modulus in Compression	$550f'_m$	$550f'_m$	$550f'_m$
Flexural Tensile Strength[2]	20 psi	10 psi	0
Shear Strength[3]			
Masonry with a running bond lay-up	27 psi	20 psi	13 psi
Fully grouted masonry with a lay-up other than running bond	27 psi	20 psi	13 psi
Partially grouted or ungrouted masonry with a lay-up other than running bond	11 psi	8 psi	5 psi

1. Masonry condition shall be classified as good, fair, or poor as defined in this standard.

Table 7-2 Factors to Translate Lower-Bound Masonry Properties to Expected Strength Masonry Properties[1]

Property	Factor
Compressive Strength (f_{me})	1.3
Elastic Modulus in Compression[2]	–
Flexural Tensile Strength	1.3
Shear Strength	1.3

1. See Chapter 6 for properties of reinforcing steel.
2. The expected elastic modulus in compression shall be taken as $550f_{me}$, where f_{me} is the expected masonry compressive strength.

Table 6-1 Default Lower-Bound Tensile and Yield Properties of Reinforcing Bars for Various Periods[1]

		Structural[2]	Intermediate[2]	Hard[2]			
	Grade	33	40	50	60	70	75
Year	Minimum Yield (psi)	33,000	40,000	50,000	60,000	70,000	75,000
	Minimum Tensile (psi)	55,000	70,000	80,000	90,000	95,000	100,000
1911-1959		x	x	x			
1959-1966		x	x	x	x		x
1966-1972			x	x	x		
1972-1974			x	x	x		
1974-1987			x	x	x	x	
1987-present			x	x	x	x	x

Notes:

1. An entry of "x" indicates the grade was available in those years.
2. The terms structural, intermediate, and hard became obsolete in 1968.

Table 6-3 ***Default Lower-Bound Compressive Strength of Structural Concrete (psi)***

Time Frame	Footings	Beams	Slabs	Columns	Walls
1900–1919	1000–2500	2000–3000	1500–3000	1500–3000	1000–2500
1920–1949	1500–3000	2000–3000	2000–3000	2000–4000	2000–3000
1950–1969	2500–3000	3000–4000	3000–4000	3000–6000	2500–4000
1970–Present	3000–4000	3000–5000	3000–5000	3000–10000	3000–5000

Table 6-4 ***Factors to Translate Lower Bound Material Properties to Expected Strength Material Properties***

Material Property	Factor
Concrete Compressive Strength	1.50
Reinforcing Steel Tensile & Yield Strength	1.25
Connector Steel Yield Strength	1.50

6.3 Material Properties and Condition Assessment

6.3.1 General

Mechanical properties for concrete materials and components shall be based on available construction documents and as-built conditions for the particular structure. Where such information fails to provide adequate information to quantify material properties or document the condition of the structure, such information shall be supplemented by materials tests and assessments of existing conditions as required in Section 2.2.6.

Material properties of existing concrete components shall be determined in accordance with Section 6.3.2. A condition assessment shall be conducted in accordance with Section 6.3.3. The extent of materials testing and condition assessment performed shall be used to determine the knowledge factor as specified in Section 6.3.4.

Use of default material properties shall be permitted in accordance with Section 6.3.2.5.

C6.3.1 General

This section identifies properties requiring consideration and provides guidelines for determining the properties of buildings. Also described is the need for a thorough condition assessment and utilization of knowledge gained in analyzing component and system behavior. Personnel involved in material property quantification and condition assessment shall be experienced in the proper implementation of testing practices and the interpretation of results.

Documentation of properties and grades of material used in component/connection construction is invaluable and may be effectively used to reduce the amount of in-place testing required. The design professional is encouraged to research and acquire all available records from original construction.

6.3.2 Properties of In-Place Materials and Components

6.3.2.1 Material Properties

6.3.2.1.1 General

The following component and connection material properties shall be obtained for the as-built structure:

1. Concrete compressive strength.
2. Yield and ultimate strength of conventional and prestressing reinforcing steel and metal connection hardware.

When materials testing is required by Section 2.2.6, the test methods to quantify material properties shall comply with the requirements of Section 6.3.2.3. The frequency of sampling, including the minimum number of tests for property determination shall comply with the requirements of Section 6.3.2.4.

Table 6-5 ***Effective Stiffness Values***

Component	Flexural Rigidity	Shear Rigidity	Axial Rigidity
Beams—nonprestressed	$0.5E_cI_g$	$0.4E_cA_w$	—
Beams—prestressed	E_cI_g	$0.4E_cA_w$	—
Columns with compression due to design gravity loads $\geq 0.5\ A_g f'_c$	$0.7E_cI_g$	$0.4E_cA_w$	E_cA_g
Columns with compression due to design gravity loads $\leq 0.3\ A_g f'_c$ or with tension	$0.5E_cI_g$	$0.4E_cA_w$	E_sA_s
Walls—uncracked (on inspection)	$0.8E_cI_g$	$0.4E_cA_w$	E_cA_g
Walls—cracked	$0.5E_cI_g$	$0.4E_cA_w$	E_cA_g
Flat Slabs—nonprestressed	See Section 6.5.4.2	$0.4E_cA_g$	—
Flat Slabs—prestressed	See Section 6.5.4.2	$0.4E_cA_g$	—

Note: It shall be permitted to take I_g for T-beams as twice the value of I_g of the web alone. Otherwise, I_g shall be based on the effective width as defined in Section 6.4.1.3. For columns with axial compression falling between the limits provided, linear interpolation shall be permitted. Alternatively, the more conservative effective stiffnesses shall be used.

Alternatively, the use of effective stiffness values in Table 6-5 shall be permitted.

6.4.1.2.2 Nonlinear Procedures

Where design actions are determined using the nonlinear procedures of Chapter 3, component load-deformation response shall be represented by nonlinear load-deformation relations. Linear relations shall be permitted where nonlinear response will not occur in the component. The nonlinear load-deformation relation shall be based on experimental evidence or taken from quantities specified in Sections 6.5 through 6.13. For the Nonlinear Static Procedure (NSP), use of the generalized load-deformation relation shown in Figure 6-1 or other curves defining behavior under monotonically increasing deformation shall be permitted. For the Nonlinear Dynamic Procedure (NDP), load-deformation relations shall define behavior under monotonically increasing lateral deformation and under multiple reversed deformation cycles as specified in Section 6.4.2.1.

The generalized load-deformation relation shown in Figure 6-1 shall be described by linear response from A (unloaded component) to an effective yield B, then a linear response at reduced stiffness from point B to C, then sudden reduction in lateral load resistance to point D, then response at reduced resistance to E, and final loss of resistance thereafter. The slope from point A to B shall be determined according to Section 6.4.1.2.1. The slope from point B to C, ignoring effects of gravity loads acting through lateral displacements, shall be taken between zero and 10% of the initial slope unless an alternate slope is justified by experiment or analysis. Point C shall have an ordinate equal to the strength of the component and an abscissa equal to the deformation at which significant strength degradation begins. Representation of the load-deformation relation by points A, B, and C only (rather than all points A–E), shall be permitted if the calculated response does not exceed point C. Numerical values for the points identified in Figure 6-1 shall be as specified in Sections 6.5 through 6.13. Other load-deformation relations shall be permitted if justified by experimental evidence or analysis.

PHOTOGRAPHS

GENERAL VIEW FROM EAST

EAST FACE TOWER

EAST FACE OF TOWER

EAST FACE PAVILION

SOUTH SIDE OF GATE
CONCRETE OVER BRICK OVER DRESSED STONE

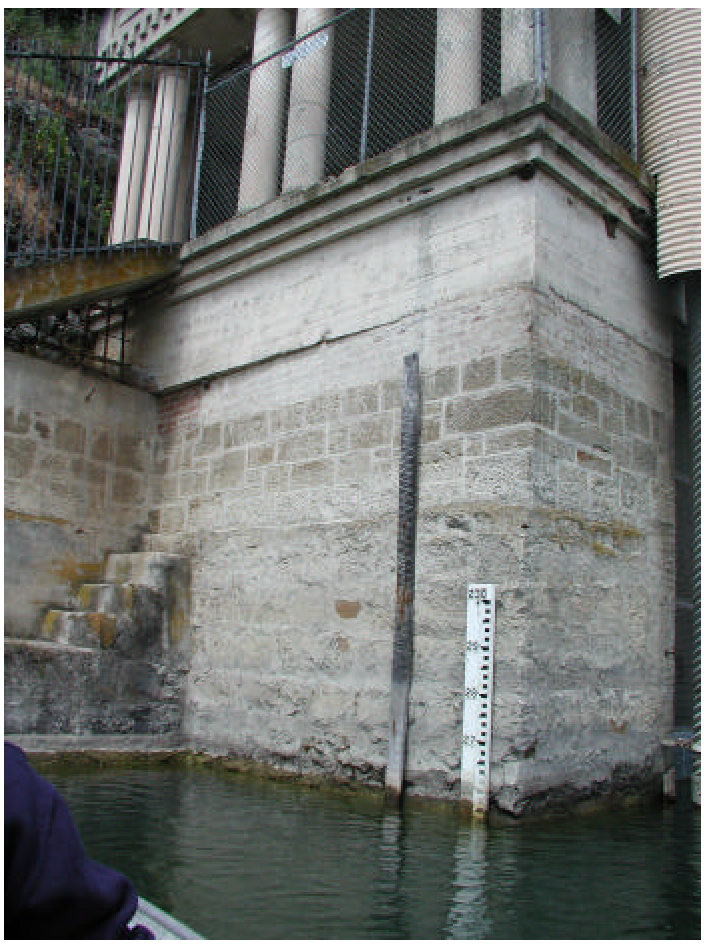
NORTH END OF GATE
DRESSED STONE OVER BRICK

NORTH END OF GATE – EAST FACE
NOTE NARROWER BRICK

VIEW INSIDE TOWER

NOTE: MATERIAL FROM LOGWAY TO SLOT IS DRESSED STONE
FROM SLOT TO CAMERA MATERIAL IS BRICK
ON FAR RIGHT SKEWED ROD IS DISLODGED SLUICE GATE SHAFT

VIEW FROM SOUTH



SOUTH FACE
CONCRETE FLOOR AND PERIMETER OVER BRICK
OVER DRESSED STONE OVER STONE

SOUTH FACE
CONCRETE, BRICK, DRESSED STONE, STONE

SOUTH FACE
CONCRETE, BRICK, DRESSED STONE
NOTE SWALLOWS NEST

TOWER FROM NORTH-EAST

TOWER FROM NORTH-EAST

TOWER FROM NORTH

NORTH FACE OF TOWER

NORTH FACE OF PAVILION

VIEW DOWN NORTH FACE

PAVILION ROOF

WEST SIDE BRIDGE TO ROOF

CLOSE-UP AT WEST SIDE - BRIDGE TO ROOF

SPALLING AND RUSTING AT ROOF

SPALLING AND RUSTING AT ROOF

SPALLING AND RUSTING AT ROOF

SPALLING AND RUSTING AT ROOF

SPALLING AND RUSTING AT ROOF

SPALLING AND RUSTING AT ROOF

COLLAR JOINT AT REMOVED BRICK

VIEW INSIDE WELL

VIEW INSIDE WELL

PUBLISHED REFERENCES

American Society of Civil Engineers, Prestandard and Commentary for the Seismic Rehabilitation of Buildings (FEMA 356), Federal Emergency Management Agency, Washington D.C., 2000

International Conference of Building Officials and California Building Standards Commission, 2001 California Building Code, California Code of Regulations, Title 24, Part 2 (Volume 2), ICBO, Whittier CA, 2002.

Merriman, T. and Wiggin, T.H., Civil Engineer's Handbook, 5th Edition, John Wiley and Sons Inc., New York, 1942.

ATTACHMENT - II

Final Report
Simplified Baseline Analysis

East Bay Municipal Utility District
Intake Tower for Tunnel #2
Chabot Reservoir, San Leandro, CA

Prepared for:
URS Corporation

January 14,2005

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SIMPLIFIED BASELINE ANALYSIS
Intake Tower for Tunnel #2 Chabot Reservoir
January 14, 2005

1.0 INTRODUCTION

OLMM Consulting Engineers is pleased to submit this report summarizing the findings and recommendations of a seismic review of the Intake Tower for Tunnel #2 at the Chabot Reservoir. This work was performed under a contract to the URS Corporation and coordinated with the work of Quest Structures in support of the Seismic Performance Evaluation of the Chabot Tower.

The scope of this seismic review consisted of (1) review of previous seismic calculations of the Intake Tower by the East Bay Municipal Utility District (EBMUD), (2) review of the information provided by Tennebaum-Manheim Engineers (TME) in their report titled "Material Properties And Condition Assessment" dated May 26, 2004, (3) seismic evaluation of the Intake Tower based on the 2001 California Building Code (CBC), (4) calculation of section properties of both the reinforced concrete and masonry members which make up the Intake Tower including demand to capacity ratios; and, (5) preparation of a brief report to summarize the findings from the current baseline analysis. For the purpose of this study our approach was to utilize existing available reports and data about the facility and observations from a site visit along with our professional engineering judgment in order to both determine the forces on the Intake Tower and to calculate the capacities of the different structural members. Inspections, material testing and geologic/soil explorations were not included within the scope of this study.

This report and associated work was conducted under the review of Dr. Sunil Gupta, Registered Structural Engineer.

2.0 FACILITIES DESCRIPTION

The Intake Tower is located at the west shore of Lake Chabot in San Leandro, California. It consists of a brick and stone masonry structure partially submerged under water with a one story reinforced concrete Pavilion on top. Photographs 1 through 4 give an idea of how the Intake Tower looked back in 1924 before the lake was filled. The Pavilion sits on top of a 4'-6" thick layer of concrete which in turn sits on top of the Tower. Both the Intake Tower and a portion of the Pavilion are partially embedded into the surrounding shoreline which consists of rock.

2.1 Pavilion

Based on a review of available drawings, the Pavilion was added to the Intake Tower some time between 1923 and 1924. A sample of the original drawings showing the Pavilion can be seen in Figures 2 through 7. The Pavilion is approximately 10'-0" tall from top of floor slab (or top of Tower) to top of roof. The roof extends over an area of roughly 23'-0"x 23'-0". The structural framing of the roof consists of a 7" thick, two way, reinforced concrete slab supported by 2'-9" wide x 2'-0" deep reinforced concrete beams along the perimeter. A 3'-0" high parapet sits on top of the roof beams. Eighteen 1'-3" diameter reinforced concrete columns support the perimeter beams. These columns are hollow and have a wall thickness of 2". The Pavilion floor, which supports the columns, consists of a 4'-6" thick unreinforced concrete slab bearing directly on top of the Intake Tower. There are several penetrations on the floor to permit access to gate shafts and log ways. Since access to these penetrations is not continuously needed, 3" precast panels are used to cover them. The precast panels are supported by reinforced concrete beams.

2.2 Intake Tower

The Intake Tower has a height of approximately 45'-0" from top of footing to top of Pavilion floor. It consists of two separate walls which are embedded into the surrounding rock with an 8'-0" gap in between them for water movement. At the top, the plan dimensions of each wall are approximately 17'- 0" in length and 6'- 4" thick. The thickness of the walls increases towards the bottom as they embed into the surrounding rock. Unfortunately, neither the existing drawings nor any available reports give dimensions indicating how the thicknesses of the walls change along the height. Therefore, the narrower plan dimensions at the top of the walls were used in all the calculations. Part of the reason why dimensions of the lower portions of the Intake Tower are missing is due to the fact that the structure is partially under water, making access to these lower areas difficult.

The walls themselves consist of different layers stacked on top of each other, each layer built from a different material. There are four distinct layers identified in the report by TME and some of the existing drawings. None of the layers has any reinforcement or anchors into the surrounding rock. The layers which make up the intake Tower walls, can be seen in Figure 1 and in Photograph 7, are as follows:

- The first layer is a 4'- 6" thick section of concrete which forms the Pavilion floor.
- The second layer is a 1'- 6" thick brick zone.
- The third layer consists of a 3'- 3" thick section of dressed stone.
- Finally, the lower 35'- 9" section of the Intake Tower is made of stone masonry.

The brick, dressed stone and the stone masonry are all laid in mortar. The thicknesses of each layer provided above were obtained from the TME report.

3.0 <u>ASSESSMENT OF EXISTING CONDITIONS</u>

3.1 Pavilion

Our assessment of the condition of the Pavilion is based on the report by TME and a site visit that took place on May 7, 2004. A visual inspection of the structure showed cracking and spalling of concrete of the Pavilion roof slab and roof beams due to corrosion of reinforcement. At this point in time the corrosion and concrete damage does not seem to have adversely affected the capacities of the members, because there are no perceptible excessive deflections. However, as in any case where reinforcement has began to corrode some type of remedial measure should take place to prevent further damage to the reinforcement and to the concrete. Photograph 8 gives an idea of the roof corrosion problem. The columns and floor of the Pavilion do not show visible corrosion of reinforcement or concrete damage.

The original drawings do not provide any information on the material properties of the concrete or the reinforcement. There was no testing done of any type. All of our calculations for the capacities of the reinforced concrete members are based on the properties provided in the report by TME which used FEMA 356-6.3.3.2.1 as its main source of information. In addition, the report details reductions applied to the material properties to account for corrosion.

3.2 Intake Tower

Our assessment of the condition of the Intake Tower is also based on the report by TME and the previously mentioned site visit. Based purely on visual inspection, the stone, bricks and grout seem to be in fairly good condition. As far as it could be seen there were no cracks in the bricks and stones and there were no areas missing grout. No testing of the brick, stone or grout was performed and FEMA 356-7.3.3.1 was used as the main source of information for the material properties. Photograph 7 shows the concrete, brick and a portion of the stone layer which makes up the Intake Tower. Reaching conclusions as to the condition of the materials that make up the Tower proved more challenging, than for the Pavilion, due to the fact that a large portion of the structure is under water, which limited how much of the Tower could be visually inspected. Therefore, it should be noted that the material properties provided in the report are based on the portions of the walls visible above the water line.

4.0 REVIEW OF PREVIOUS SEISMIC CALCULATIONS

In May of 1991 EBMUD performed a seismic review of the Intake Tower. The analysis was based on the 1988 Unified Building Code (UBC) and it concentrated on the evaluation of the Pavilion. A very brief lateral capacity check of the Intake Tower was also done.

4.1 Pavilion

The calculations by EBMUD explain that the Pavilion does not qualify under any defined lateral structural system in the 1988 UBC, but, in the interest of completing the analysis, a system that best fit the given parameters was chosen. A Special Moment Resisting Space Frame (SMRSF), with an Rw value of 3.0, was eventually used because under the code at the time it was the only system that could be used for concrete construction in a zone 4 area. Furthermore, the base shear for the structure was computed for two different importance factors, I = 1.0 and 1.5. This was done, in part, to compare the behavior of the structure at two different force levels.

It was concluded that the Pavilion would not collapse if an importance factor of I = 1.0 was used in the calculation of the base shear, but the structure would suffer severe damage and would no longer be safe to use. However, if an importance factor of I = 1.5 were to be used, the structure would suffer serious damage leading to a possible collapse.

4.2 Intake Tower

In the case of the Intake Tower the same base shear coefficients as the ones used for the Pavilion were used in the EBMUD analysis. While the calculations for the Tower walls were far more simplified than for the Pavilion, the results were more conclusive. The shear stress in the stone masonry for importance factors of I =1.0 and 1.5 exceeded the allowable shear stresses of the material. Therefore, it was concluded the Intake Tower would be severely damaged under both levels of seismic forces calculated using importance factors of I = 1.0 and 1.5.

5.0 SEISMIC ANALYSIS AND FINDINGS

In our analysis, the first item which needed to be established, much like in the original EBMUD report, was the type of structural system to be used for the Intake Tower. There is no structural system within the 2001 CBC in which this Intake Tower can be categorized. But, because of low ductility and archaic materials of construction, an R value greater than 2.0 did not seem reasonable. As a comparison, the code allows an R value of 2.2 for Cantilevered Column Building Systems and the Tower could be interpreted as cantilevering from its foundation. The Pavilion was analyzed as a separate structure from the Tower and assumed fixed at its lower level. Splitting the two structures is appropriate when the greater stiffness of the Tower is taken into account due to both the size of the walls and their embedment into surrounding rock. Both structures were analyzed for an importance factor of 1.5 per 2001 CBC. An importance factor of 1.5 was deemed appropriate because the Tower is used to empty the reservoir should a breach in the dam occur in the event of a major earthquake.

The following parameters required to determine the base shear from the 2001 CBC were provided by the URS Corporation:

- Fault Type A
- Soil Type S_B
- Near Source Distance = 0.5km

Based on the given information, the following constants and base shear values were obtained using 2001 CBC:

- Na = 1.2
- Nv = 2.0

- Ca = 0.6
- Cv = 0.8
- V = 1.125 x (Weight of Structure) (EQ. 30-5, 2001 CBC)
- Vmin (Other Non-Building Structure) = 0.96 x (Weight of Structure) (EQ. 34-3, 2001CBC)

The original EBMUD report used a base shear of V = 0.54 x (Weight of Structure).

5.1 Pavilion

An analysis of the Pavilion was performed by creating a computer model of the structure using the SAP 2000 computer program. Analyses were performed for both gravity and seismic forces. Material properties provided in the TME report for the concrete and reinforcing were included in the model. As a general assumption all the beam-to-column joints and column-to-floor slab joints were modeled as rigid. See pages #3-30 of the attached calculations for the SAP 2000 model.

The next steps in the analysis involved calculating the flexural, axial and shear capacities of different members based on the material properties available, dimensions and quantity of reinforcement shown in the original drawings (Figures 2 through 7) for the Pavilion. The Moment-Axial Force interaction diagrams for the columns were calculated using the computer program "PCA-COLUMN". These capacities were used to estimate Demand to Capacity Ratios (DCR) in order to obtain an understanding of how the structure might behave during a seismic event. The code bases capacities on the underlying assumption that proper detailing of the members, as delineated in the code, has been incorporated to develop full capacities. Therefore, based on our engineering judgment, some of the member capacities have been reduced to account for insufficient or missing shear reinforcement, inadequate development lengths and deficient confinement of compression elements. We have assigned the Pavilion roof beams greater capacity reduction values than for the columns since the beams have no shear reinforcement and the columns have ties at 18" O.C. A summary of the capacity reductions used is as follows:

Member Type	Type Of Force	Capacity Reduction
Roof Beam	Moment	50%
Floor Beam	Moment	50%
Column	Moment	33%
Column	Axial Force	33%

Once the capacities were calculated they were compared with the demands calculated. DCR values are summarized below:

Member Type	Type Of Force	DCR
Roof Beam	Moment @ Supports	3.96
	Moment @ Midspan	2.82

	Shear Force	0.71
Column	Moment + Axial Force	5.85
	Shear Force	1.23
Floor Beam	Moment @ Supports	0.67
	Moment @ Midspan	0.41
	Shear Force	0.42
Pavilion Roof Stem Wall	Shear Force	0.92

A review of the DCR values shows that the Pavilion roof beams will fail in bending. Since there are no ties in the beams it is likely the longitudinal reinforcement will buckle. The results also indicate that the columns will fail due to bending and axial compression and shear, since the demand on these columns is almost six times their capacity. There are not enough ties in the columns to provide proper confinement of the concrete, which would lead to a likely collapse of the structure.

5.2 Intake Tower

The lateral force applied to the walls of the Intake Tower was based on the base shear resulting from the mass of the walls themselves. Equation 30-15 from the 2001 CBC was used to distribute the base shear to the different layers of stone and brick which make up the Tower walls. In addition, the lateral force from the Pavilion was applied at the top of the Tower. There is no reinforcement or anchors of any type for the masonry walls. All the shear and tension in these members is resisted by the brick, stone and grout alone.

The shear and axial forces along with moments on the walls were used to calculate shear, tensional and compressional stresses in order to compare them with allowable stress values. Part of the TME report includes allowable stress values, based on FEMA 356, for the different materials which make up the Intake Tower. However, as part of our work we searched through other sources for further information on reasonable allowable stresses we could use in our analysis. As can be expected different sources provided significantly different possible capacities. The final allowable stresses used came from averaging the values we found in a textbook titled “Reinforced Masonry Engineering Handbook” by J.E. Amrhein (See page #59 of calculations) with those provided in the TME report.

It is not clear from the information available how much each wall that makes up the Intake Tower is embedded into the surrounding rock at its base and back side. Therefore, in order to deal with this issue the seismic analysis includes two separate cases. Case I involves the conservative assumption that the walls are connected to the surrounding rock only at their base, which leads to a wall height of 45ft to be used in the calculations. While in Case II it is assumed that the lowest 1/3 section of the Stone Masonry layer is embedded into the surrounding rock, producing reduced wall heights of 33’-1” as shown in Page #61 of the attached calculations. Since there are no drawings with dimensions indicating the embedment of the walls into the rock, Photographs 1 through 4 were used to estimate the embedment. The base shear coefficient of 1.125, shown in section 5.0, remains the same in both cases, but the reduction in wall height for Case II resulted in a drop of 25% in the base shear for Case II compared to Case I. The tables below show DCR values for the different layers which make up the Intake Tower for both Cases I and II.

CASE I

Layer No *	Height (ft)	Material Type	Type of Stress	DCR
1	4'- 6"	Concrete	Compressive	0.02
			Tensile	0.13
			Shear	0.15
2	1'- 6"	Brick	Compressive	0.08
			Tensile	3.07
			Shear	0.81
3	3'- 3"	Dressed Stone	Compressive	0.08
			Tensile	7.91
			Shear	0.70
4	35' – 9"	Stone Masonry	Compressive	0.73
			Tensile	86.81
			Shear	1.48

* The layers are organized beginning from the top of the Intake Tower down to its base.

CASE II

Layer No *	Height (ft)	Material Type	Type of Stress	DCR
1	4'- 6"	Concrete	Compressive	0.02
			Tensile	0.12
			Shear	0.15
2	1'- 6"	Brick	Compressive	0.08
			Tensile	2.98
			Shear	0.79
3	3'- 3"	Dressed Stone	Compressive	0.07
			Tensile	7.65
			Shear	0.67
4	35' – 9"	Stone Masonry	Compressive	0.44
			Tensile	50.97
			Shear	1.14

* The layers are organized beginning from the top of the Intake Tower down to its base.

After reviewing the DCR values for the different layers in both Cases I and II, some trends become apparent. Since the tensile capacity of the materials is very low, three of the four layers which make up the Intake Tower fail in tension due to the tensile stresses produced by moments in the walls. The lowest layer, made up of stone masonry, exceeds its allowable tensile stress by more than 80 times for Case I and by more than 50 times for Case II. In addition, the lowest layer also fails in shear. A total of about 90% of the Intake Tower is overstressed either due to flexural or shear forces. Severe damage across the walls can be expected due to bending and shear. It is unreasonable to expect an unreinforced masonry structure to sustain the flexural and shear stresses this tower will see during a seismic event. See Pages #60 and 61 of the calculations for a summary of the Tower forces and stresses.

6.0 CONCLUSIONS AND RECOMMENDATIONS

Our analysis indicates that both the Intake Tower and the Pavilion will be severely overstressed under 2001 CBC forces. The Pavilion lacks the reinforcement and proper detailing required by the 2001 CBC for the forces which will be produced during an earthquake. The Intake Tower is also not capable of resisting the flexural and shear demands due to the fact it has no reinforcement at all. Based on these analyses and results for the 2001 CBC, it is our professional opinion that both the Intake Tower and the Pavilion can likely sustain severe damage during a major earthquake with potential for collapse.

While the materials could be tested to determine the true allowable stresses, in our professional opinion it may not provide any benefit. The members which make the Pavilion and Intake Tower walls are overstressed to a point where testing of the materials would not improve allowable stresses enough to make a significant difference. Furthermore, strengthening of the members seems unrealistic due to the condition, location and size of the members. The Intake Tower is partially under water, embedded into surrounding rock, built from archaic construction materials and quite extensive in size. Attempting to somehow strengthen it could prove to be an expensive enterprise. The Pavilion presents its own difficulties due to the inadequate reinforcement and detailing of the members. It would take a considerable amount of reinforcement to bring the structure up to 2001 CBC standards, including the replacement and rehabilitation of reinforcement already corroding.

7.0 LIMITATIONS

Our services were performed in accordance with general accepted standards of professional practice for the locality, intended use of the project, and at the time such services were rendered. No other warranty or representation, either expressed or implied, is included in this report. Specifically, the findings and recommendations presented herein were based on our limited calculations, review of the information made available to us and no testing was performed.

8.0 DOCUMENTS REVIEWED

The following documents relevant to this project were reviewed as part of our study:

1. Tennebaum-Manehim Engineers report dated May 26, 2004: “Material Properties and Condition Assessment for Intake Tower for Tunnel #2 Chabot Reservoir”.

2. East Bay Municipal Utility District report dated April 25, 1991" "Seismic Evaluation of Chabot Reservoir - Tunnel No. 2, Inlet Structure".

3. "Reinforced Masonry Engineering Handbook. Brick and Other Structural Clay Units." By J.M. Amrhein. Copyright © 1972 by Masonry Institute of America, Pg. 155.(See Page #59 of calculations.)

4. Drawings E1102, E1103 and E1103 by East Bay Water Company, Oakland, CA dated October 19, 1923: "Plans of Structure at Intake of Tunnel No. 2, Lower San Leandro Project".

5. Drawings D1101 and D1103 by East Bay Water Company, Oakland, CA dated November 20, 1923: "Plans of Structure at Intake of Tunnel No. 2, Lower San Leandro Project".

6. Drawing 9480-G-1 by East Bay Municipal Utility District, Oakland, CA dated May 1, 1991: "Chabot Reservoir Outlet Tower Screen Replacement (Tunnel #2 Inlet Structure), Section/Elevation".

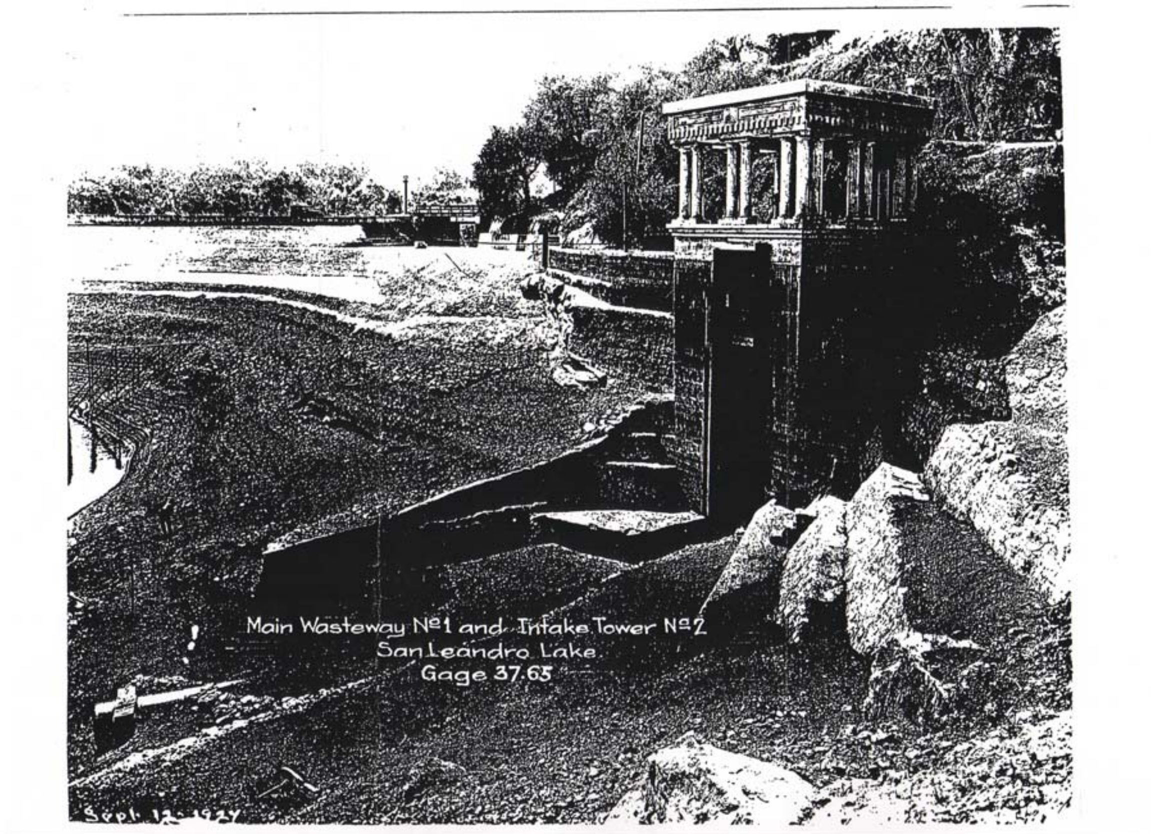
7. Drawing 9480-G-2 by East Bay Municipal Utility District, Oakland, CA dated May 1, 1991: "Chabot Reservoir Outlet Tower Screen Replacement (Tunnel #2 Inlet Structure), Floor Plan and plan Beneath Floor".

8. Five pictures of Intake Tower Taken in 1924.

PHOTOGRAPHS

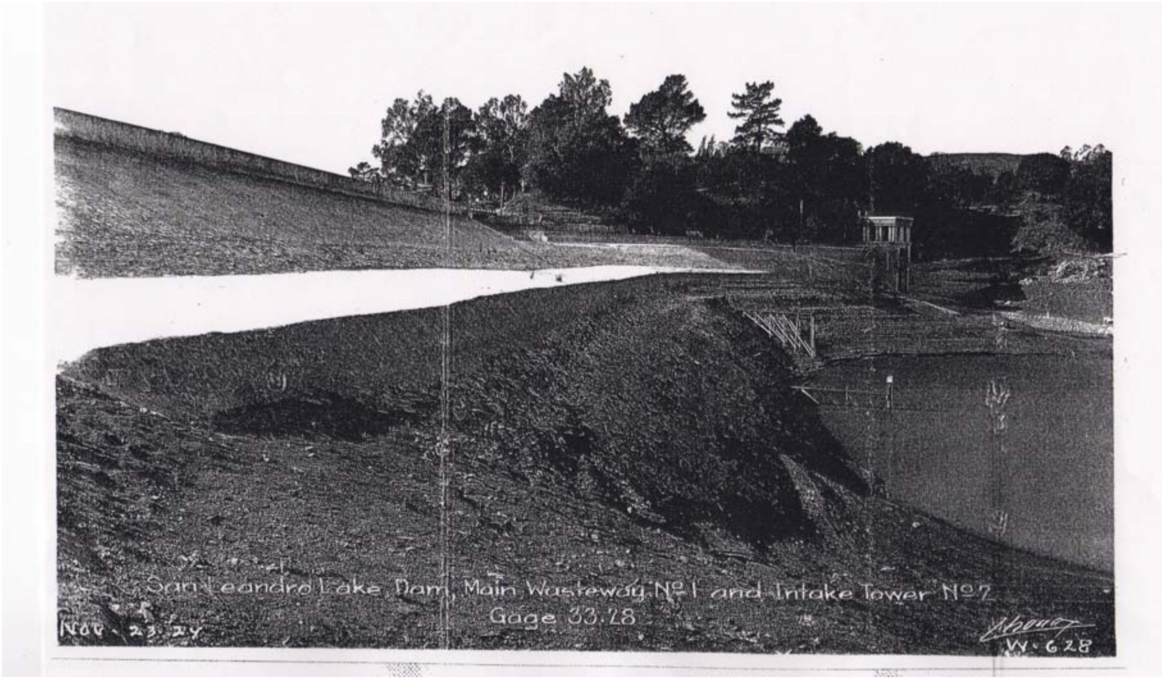
Intake Tower #2
Chabot Reservoir
San Leandro, CA. 1924

Photograph 1.



Intake Tower #2
Chabot Reservoir
San Leandro, CA. 1924

Photograph 2.



Intake Tower #2
Chabot Reservoir
San Leandro, CA. 1924

Photograph 3.

Intake Tower #2
Chabot Reservoir
San Leandro, CA. 1924

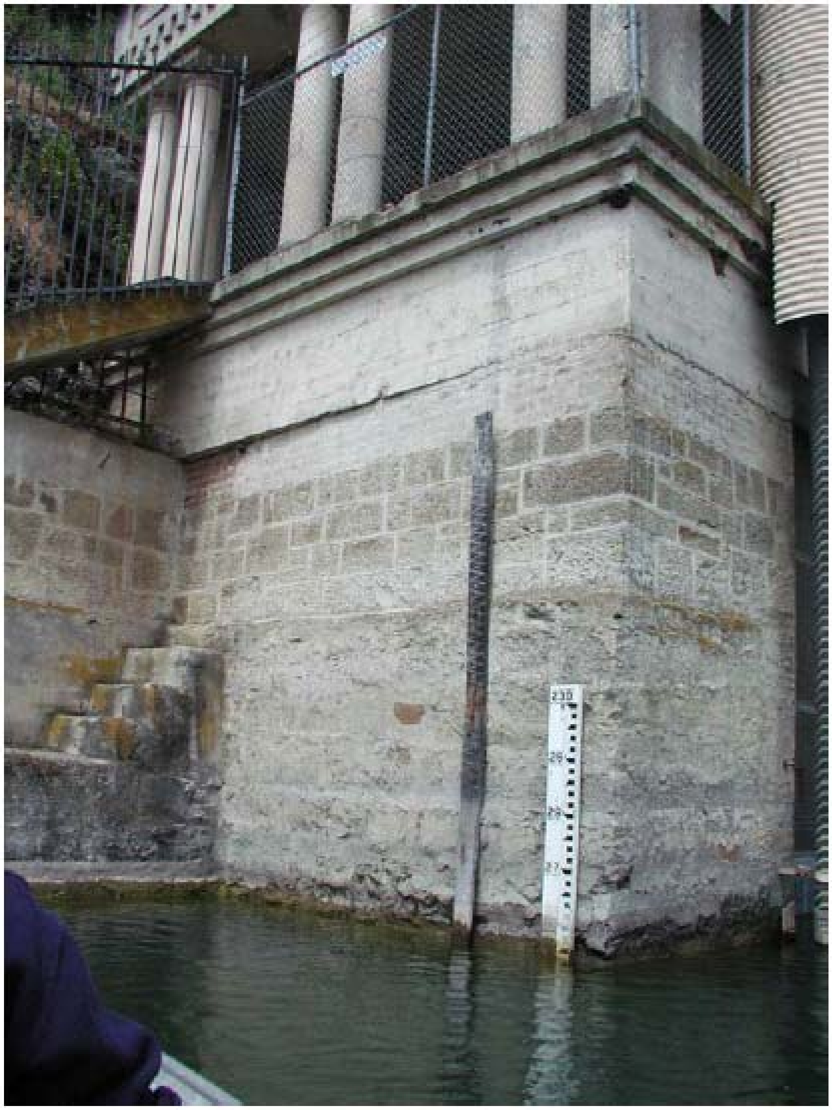
Photograph 4.

East Face of Intake Tower

Photograph 5.

South View of Intake Tower

Photograph 6.



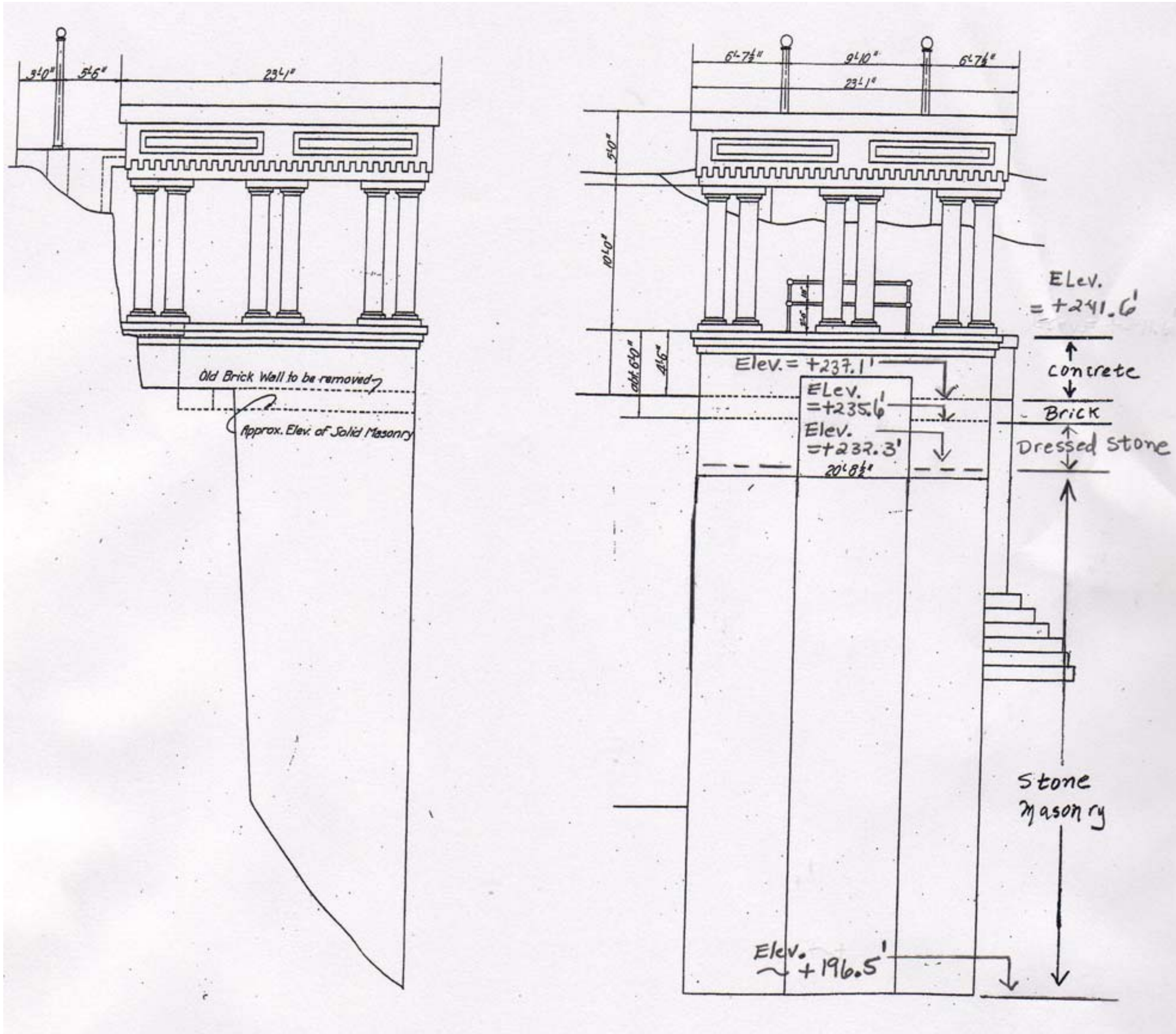
South face of Intake Tower
Layers of Concrete, Brick And
Dressed Stove

Photograph 7.

Concrete Spalling at Pavilion Roof

Photograph 8.

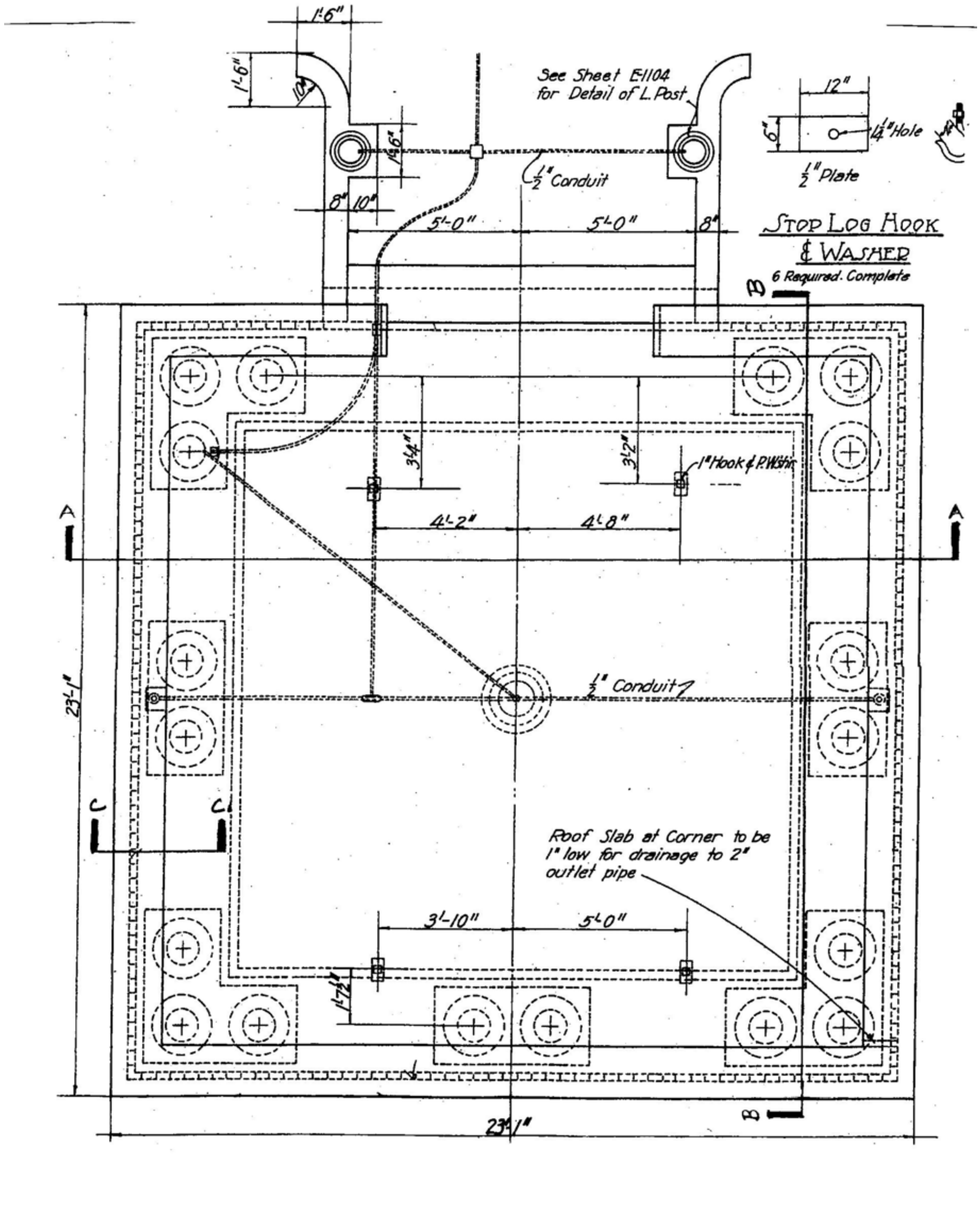
FIGURES



South Elevation **East Elevation**

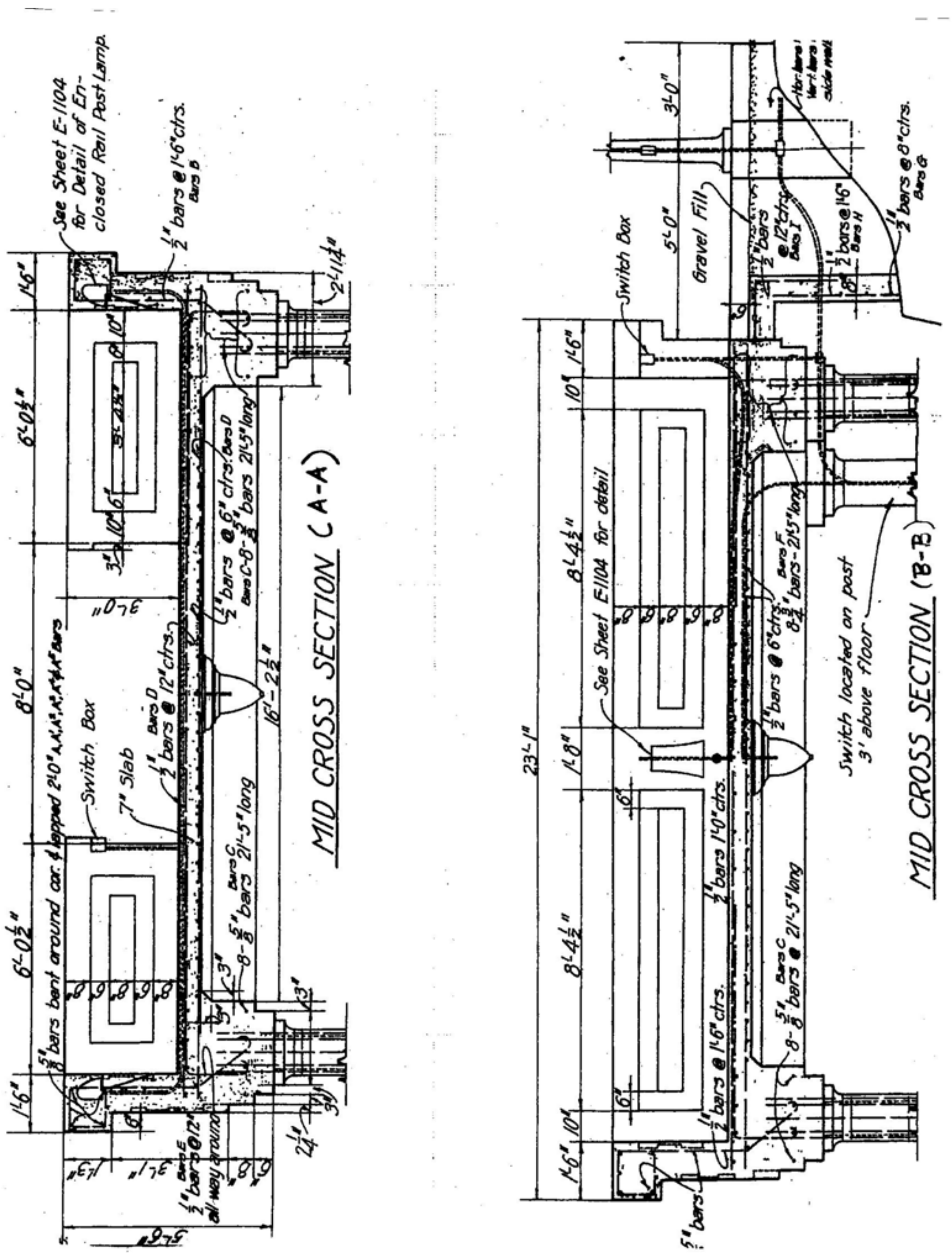
South and East Elevations
Of Intake Tower

Figure 1.



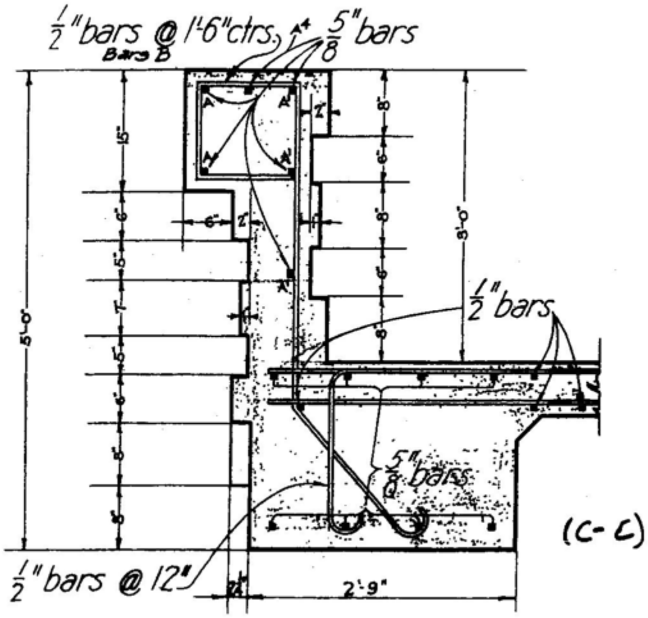
Pavilion Roof
Framing Plan

Figure 2.



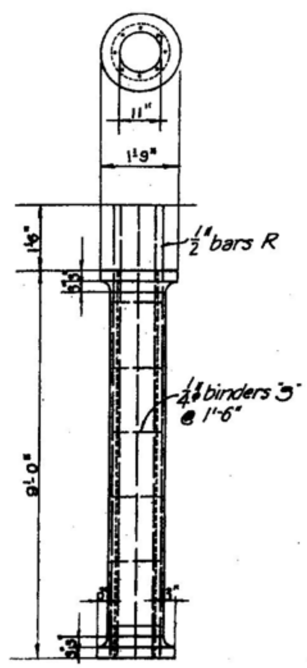
Cross-Sections
Of Pavilion Roof

Figure 3.



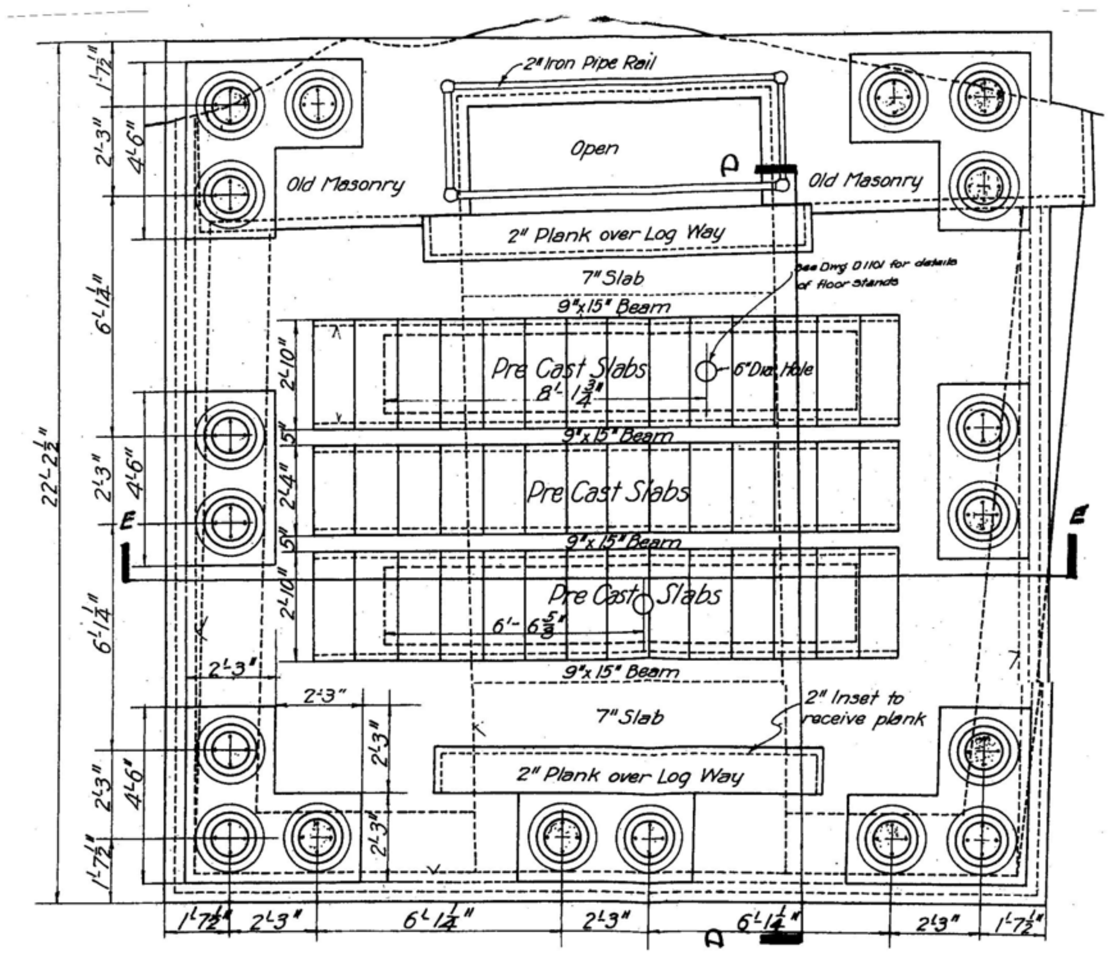
Cross-Section of
Pavilion Roof Beam

Figure 4.



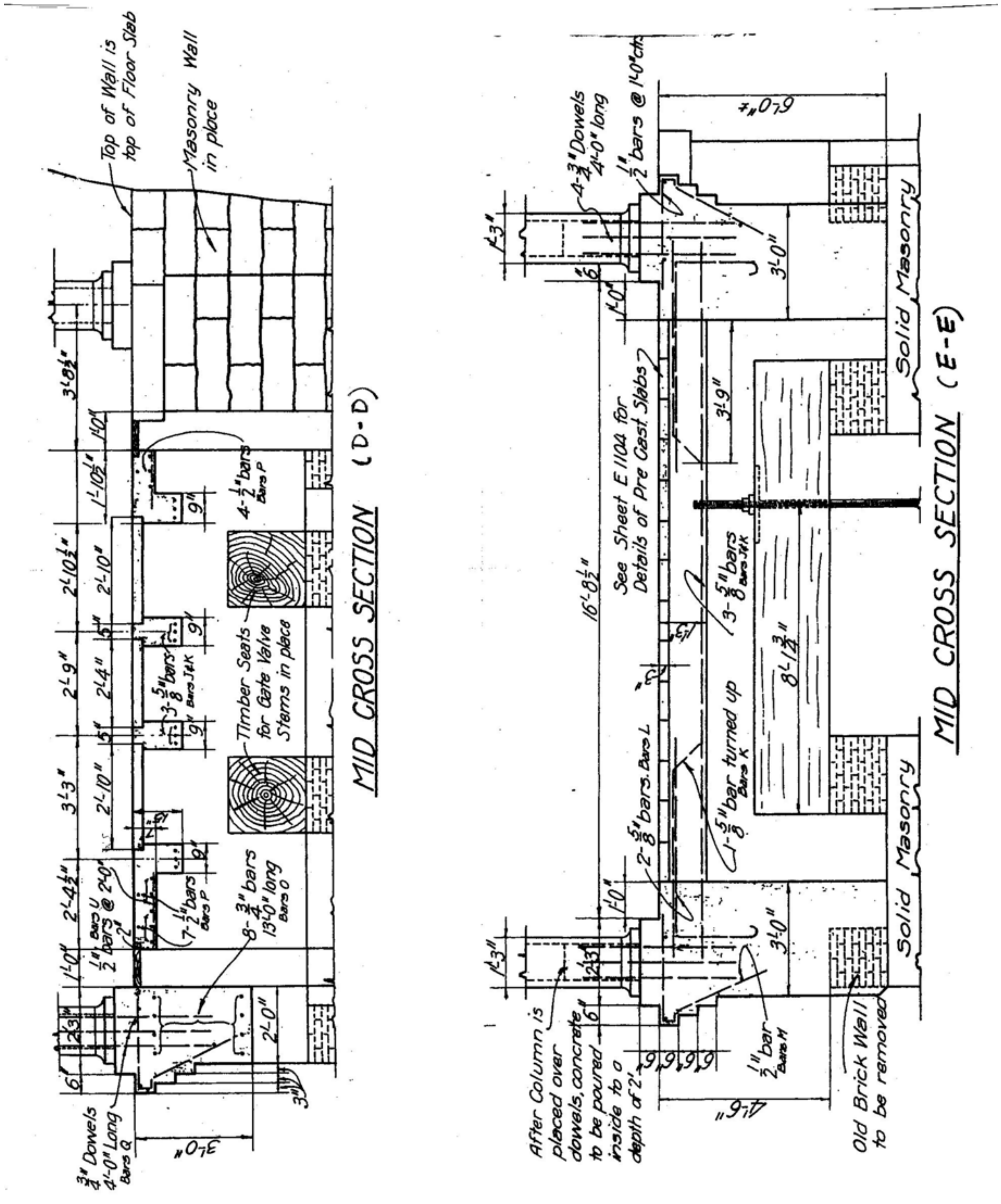
Elevation of
Pavilion Column

Figure 5.



Pavilion Floor
Framing Plan

Figure 6.



Cross-Sections of
Pavilion Floor

Figure 7.

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APPENDIX – A

Quantitative Analysis and Evaluation Calculations

Intake Tower for Tunnel #2
Chabot Reservoir
San Leandro, CA

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Table of Contents

January. 14, 2005

SUMMARY

The calculations are divided in to two sections. Section "A" is for the Pavilion and section "B" is for the Intake Tower.

Section "A":

The calculations for the Pavilion include the gravity forces (Page #1) and Base Shear calculation (page #2) for the structure. These forces were then entered into a 3-dimensional model of the Pavilion in SAP 2000 (Pages 3-30). The SAP 2000 output included in the calculations contains: diagrams showing members forces, geometry of the structure, load combinations, reactions and member forces. A summary of the controlling forces for all members can be found in Page #44. The controlling forces were then inserted into spread sheets to determine the Demand-to-Capacity Ratios (DCR) for the different members (Pages #45-54). A DCR value greater than 1.0 represents a member with forces greater than its capacities and requiring some strengthening procedure to avoid damage during a seismic event. While a DCR value of less than one represents a member that requires no strengthening since the forces are less than its capacity.

The flexural and shear capacities of the roof beams in the Pavilion were calculated in the spreadsheets shown in Pages #45-50. The capacities of the beams were determined based on the equations provided in the 2001 CBC. The capacities of the columns were calculated using the program PCA COLUMN. The output produced by the program is shown on Pages #31-44. There are two sets of capacities calculated for the columns since our scope of work required us to provide Quest Structures with column capacities with the appropriate reduction factors (φ) per 2001 CBC and with unreduced capacities. The column capacities from PCA COLUMN were then entered into a spreadsheet and provided to Quest Structures (Pages 51-52). Finally, the DCR values for the columns are shown in Page #53.

Section "B":

The calculations for the Intake Tower include the weight of the Tower (Pages #55-56) and the base shear (Pages #57-58). The base shear calculations show the two cases studied for the Tower. In Case I the walls that form the Tower are assumed to be connected to the surrounding rock only at the bottom of each wall. This is the more conservative of the two cases since the walls are assumed to behave as standing cantilevers. Case II takes into account the fact that a section of each wall embeds into the surrounding rock along its height. This condition was included in Case II by decreasing the height of the walls in the seismic calculations. The wall forces were then used to calculate the stresses in the materials and compared to the allowable stresses (Pages #59-61).

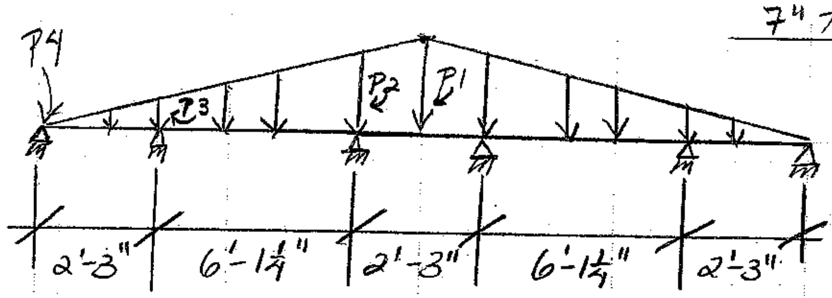
A. PAVILION

OLMM Consulting Engineers
• San Francisco (415) 882-9449
• Oakland (510) 433-0828

JOB: Chabot Tower
SHEET NO. 1 OF
CALCULATED BY: F.C DATE: 6/8/04
CHECKED BY: DATE:
SCALE:

Roof Beam Loads:

7" Thick 2-Way slab



Parapet → $W_{DL} = 0.15\,K/ft^3\,(1.5ft)(1.25ft) + 0.15K/ft^3\,(1ft)(3ft - 1.25ft)$
$= 0.544\,K/ft$ $\quad (23 + 1/12)ft - 2.75ft - ((2.25+6)/12)(2)$

$P1_{DL} = 0.544K/ft + (18.96\,ft/2)(7/12\,ft)(0.15K/ft^3)$ $\quad = 18.96ft$

See Figure 4

$P1_{DL} = 0.544\,K/ft + 0.83K/ft = 1.374K/ft$
$P1_{LL} = (18.96\,ft/2)(20/1{,}000\,ksf) = 0.19K/ft$

$P2_{DL} = 0.544\,K/ft + 0.83K/ft\,(8.35ft/9.48ft) = 1.275K/ft$
$P2_{LL} = 0.19K/ft\,(8.35ft/9.48ft) = 0.167K/ft$

$P3_{DL} = 0.544\,K/ft + 0.83K/ft\,(2.25ft/9.48ft) = 0.741K/ft$
$P3_{LL} = 0.19K/ft\,(2.25\,ft/9.48ft) = 0.045\,K/ft$

$P4_{DL} = 0.544\,K/ft$
$P4_{LL} = 0$

OLMM
CONSULTING
ENGINEERS

San Francisco
(415)882-9449
Oakland
(510)433-0828

PROJECT	Chabot Tower/Pavilion
JOB NO.	0404
ENGINEER	FC
CHECKED BY	OLMM

FILENAME C:\FC-Jobs\0404-Chabot Tower\Excel\[Ubc97Vb-Pavilion.xls]UBC97X

UBC 97 STATIC SEISMIC LOAD CALCULATION

STRENGTH DESIGN

Date= 6/8/2004

PAVILION

IMPORTANT FACTOR	1.5				
BUILDING TYPE					
BUILDING HEIGHT(hn)(Ft)	10	FT			
PLAN IRREGULARITIES	YES				
VERTICAL IRREGULARITIES	YES		Given Na?	No	
FAULT TYPE	A		Given Nv?	No	
NEAR SOUCE DISTANCE(km)	0.5	Km	*If "Yes", Input Data*		
Z	0.4		Na(Given)	1.5	
Ct	0.02		Nv(Given)	2	
S	SB				
R	2		Ωo		
PERIOD T_B (fomula 30-10)	0.11	s	W =	131	kips
Meet 1629.4.2. Requirement?	No	("Yes" or "No")			

Na(Code)	1.5	Na(Used)	1.5	
Nv(Code)	2	Nv(Used)	2	
Ca			0.6	
Cv			0.8	
$T_A = Ct(hn)^{3/4}$			0.11	s
USED T for strength design			0.11	s

1.5

$V = \boxed{\dfrac{Cv*I*W}{RT}}$ strength design 5.4545 W

drift check 5.4545 W

$V_{max} = \boxed{\dfrac{2.5*Ca*I*W}{R}}$ 1.125 W

$V_{min} = \boxed{\dfrac{0.8*Z*Nv*I*W}{R}}$ 0.48 W

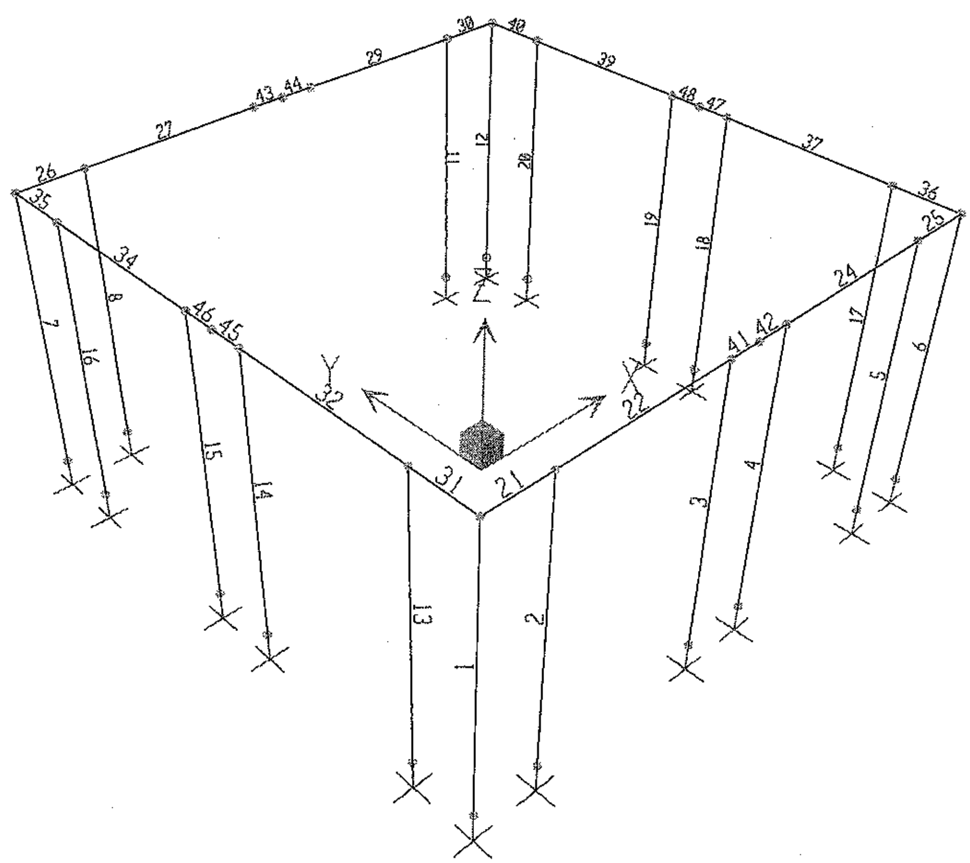
$V_{min} = \boxed{0.11*Ca*I*W}$ 0.099 W (Omit for drift checking)

FOR STRENGTH DESIGN

V= 1.125 **W =** **147.4** kips

FOR DRIFT CHECKING

V= 1.125 **W =** **147.4** kips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Z

SAP2000

4

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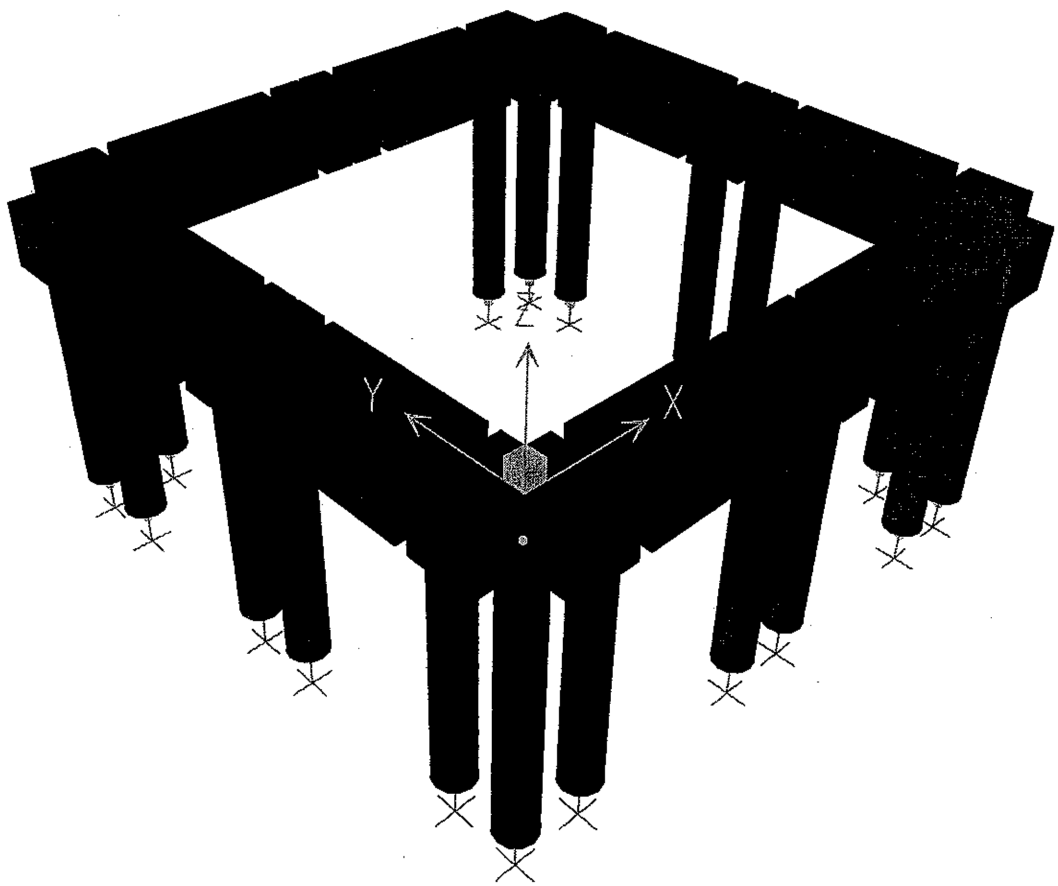

SAP2000 v7.44 - File:Pavilion - 3-D View - Kip-in Units

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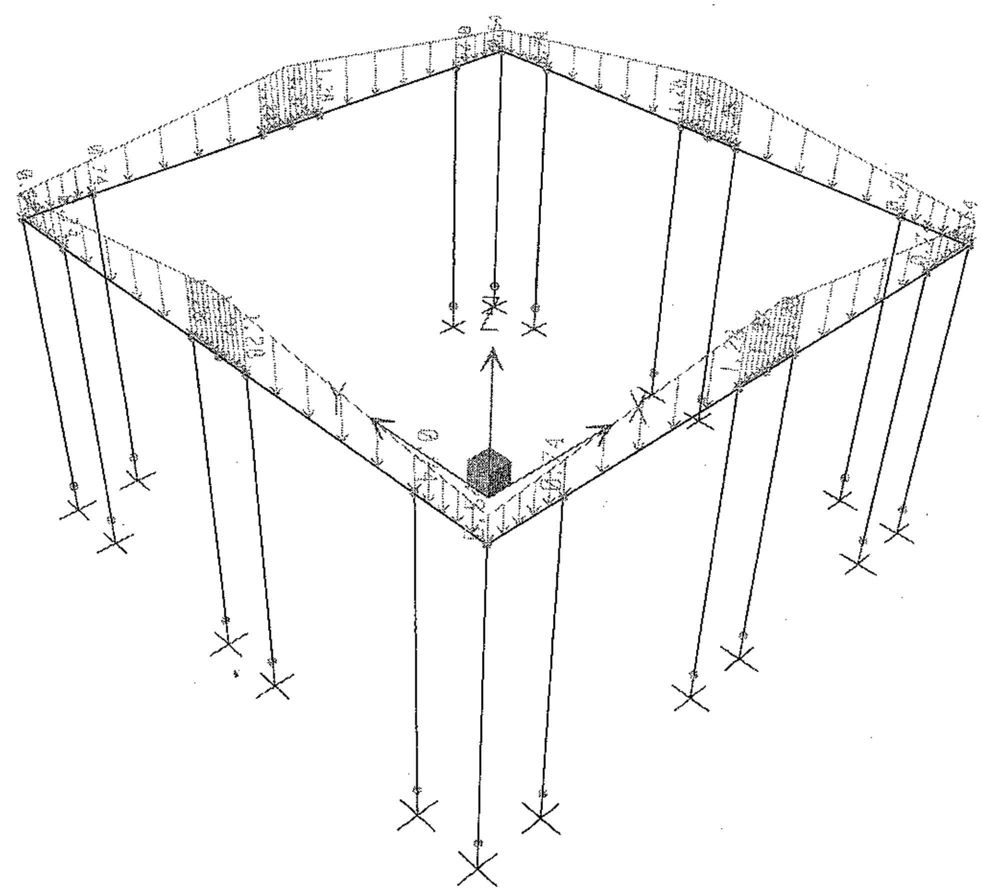

SAP2000 v7.44 - File:Pavilion - Frame Span Loads (DL) - Kip-ft Units

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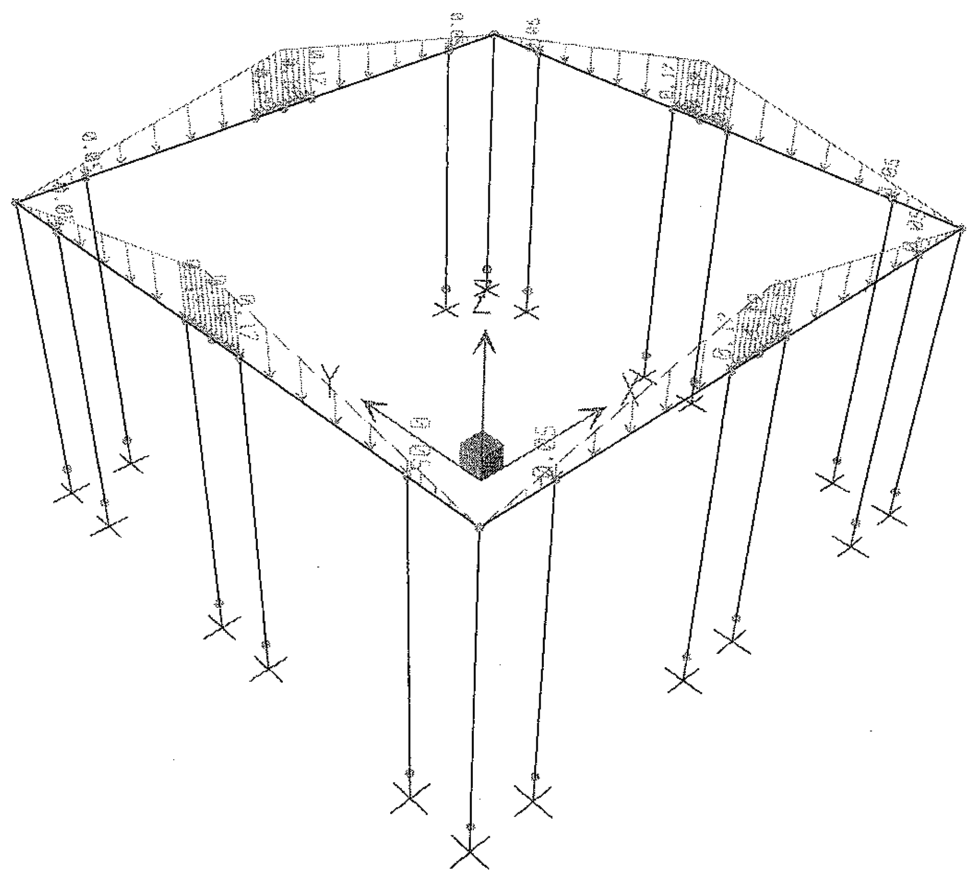

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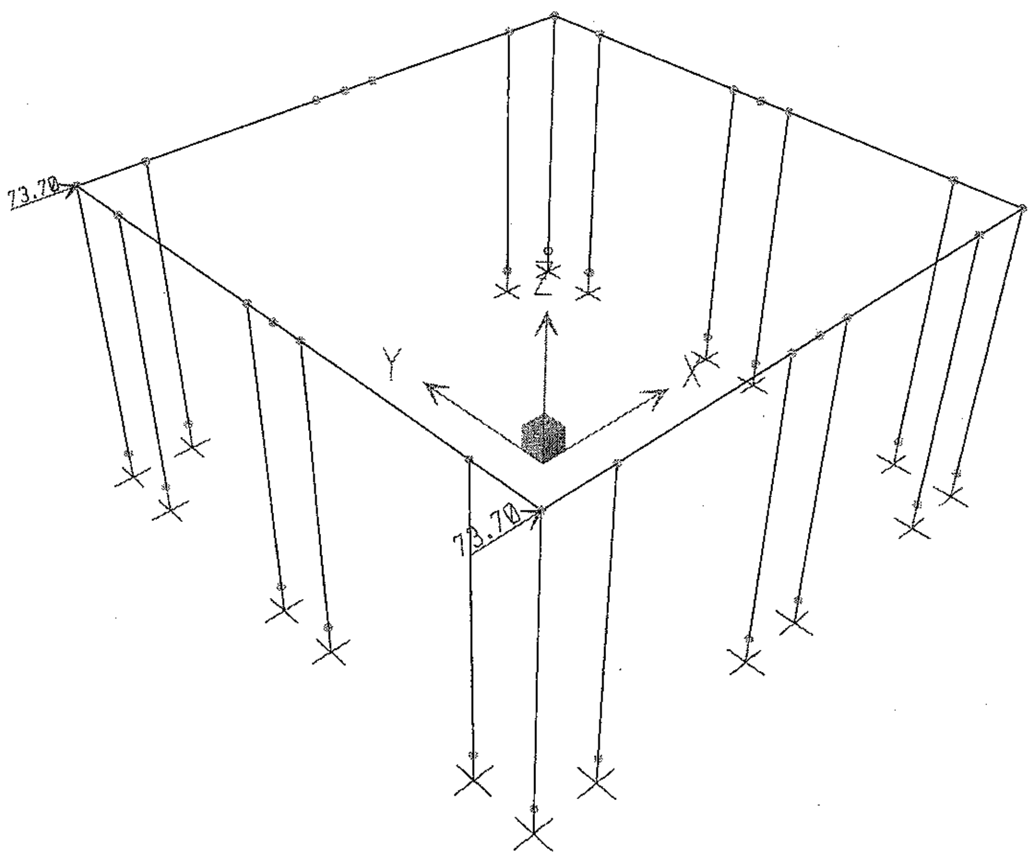
73.70
73.70
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SAP2000

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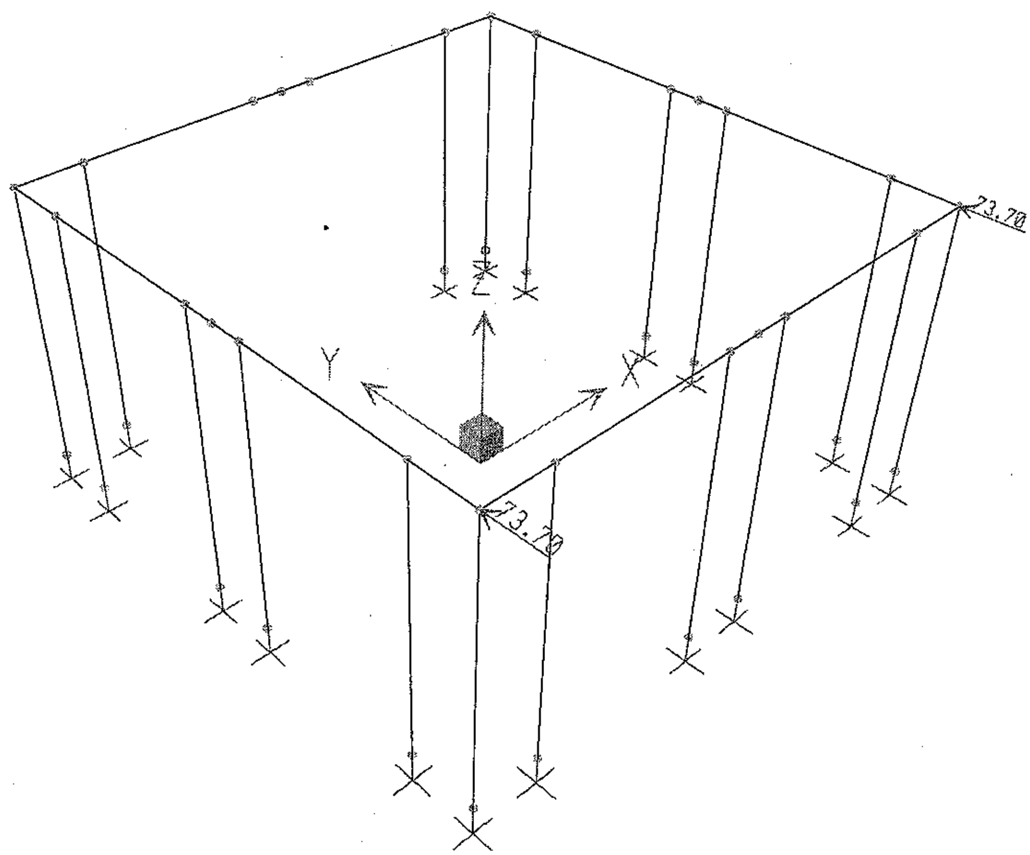

SAP2000 v7.44 - File:Pavilion - Joint Loads (EQY) - Kip-ft Units

FACTORED LOAD CASES & COMBINATIONS:

LOAD CASES CONSIDERED:

Dead Load	(DL)
Live Load	(LL)
Earthquake X-Direction	(EQX)
Earthquake Y-Direction	(EQY)

LOAD COMBINATIONS CONSIDERED:

The load combinations considered are based on the 2001 CBC load combinations for strength design presented in section 1612.2.1 and section 1909.2.1 for concrete.

For load combinations containing earthquake loads, the load factor for Dead Load includes the load effect from the vertical component of the earthquake ground motion calculated as 0.5CaID = 0.5*0.60*1.5*DL = 0.45DL (See section 1630.1.1).

In addition, orthogonal effects are considered in these load combinations, using 100% of the seismic load in one direction with 30% of the seismic load in the perpendicular direction (See section 1633.1).

LC #	Load Case	2001 CBC EQ #/Pg #
1	1.4DL+1.7LL	9-1/Pg #2-113
2	(1.2+0.45)DL+0.5LL+EQX+0.3EQY	12-5/Pg #2-4
3	(1.2+0.45)DL+0.5LL+EQX-0.3EQY	12-5/Pg #2-4
4	(1.2+0.45)DL+0.5LL-EQX+0.3EQY	12-5/Pg #2-4
5	(1.2+0.45)DL+0.5LL-EQX-0.3EQY	12-5/Pg #2-4
6	(1.2+0.45)DL+0.5LL+0.3EQX+EQY	12-5/Pg #2-4
7	(1.2+0.45)DL+0.5LL+0.3EQX-EQY	12-5/Pg #2-4
8	(1.2+0.45)DL+0.5LL-0.3EQX+EQY	12-5/Pg #2-4
9	(1.2+0.45)DL+0.5LL-0.3EQX-EQY	12-5/Pg #2-4
10	(0.9-0.45)DL+EQX+0.3EQY	12-6/Pg #2-4
11	(0.9-0.45)DL+EQX-0.3EQY	12-6/Pg #2-4
12	(0.9-0.45)DL-EQX+0.3EQY	12-6/Pg #2-4
13	(0.9-0.45)DL-EQX-0.3EQY	12-6/Pg #2-4
14	(0.9-0.45)DL+0.3EQX+EQY	12-6/Pg #2-4
15	(0.9-0.45)DL+0.3EQX-EQY	12-6/Pg #2-4
16	(0.9-0.45)DL-0.3EQX+EQY	12-6/Pg #2-4
17	(0.9-0.45)DL-0.3EQX-EQY	12-6/Pg #2-4

L O A D   C O M B I N A T I O N   M U L T I P L I E R S

COMBO	TYPE	CASE	FACTOR	TYPE	TITLE
COMB1	ADD				1.4DL+1.7LL
		DL	1.4000	STATIC(DEAD)	
		LL	1.7000	STATIC(LIVE)	
COMB2	ADD				1.65DL+0.5LL+EQX+0.3EQY
		DL	1.6500	STATIC(DEAD)	
		LL	0.5000	STATIC(LIVE)	
		EQX	1.0000	STATIC(QUAKE)	
		EQY	0.3000	STATIC(QUAKE)	
COMB3	ADD				1.65DL+0.5LL+EQX-0.3EQY
		DL	1.6500	STATIC(DEAD)	
		LL	0.5000	STATIC(LIVE)	
		EQX	1.0000	STATIC(QUAKE)	
		EQY	-0.3000	STATIC(QUAKE)	
COMB4	ADD				1.65DL+0.5LL-EQX+0.3EQY
		DL	1.6500	STATIC(DEAD)	
		LL	0.5000	STATIC(LIVE)	
		EQX	-1.0000	STATIC(QUAKE)	
		EQY	0.3000	STATIC(QUAKE)	
COMB5	ADD				1.65DL+0.5LL-EQX-0.3EQY
		DL	1.6500	STATIC(DEAD)	
		LL	0.5000	STATIC(LIVE)	
		EQX	-1.0000	STATIC(QUAKE)	
		EQY	-0.3000	STATIC(QUAKE)	
COMB6	ADD				1.65DL+0.5LL+0.3EQX+EQY
		DL	1.6500	STATIC(DEAD)	
		LL	0.5000	STATIC(LIVE)	
		EQX	0.3000	STATIC(QUAKE)	
		EQY	1.0000	STATIC(QUAKE)	

$0.5 C_a I\, DL = 0.5(0.6)(1.5)(DL)$
$= 0.45 DL$

$= 1.2DL + 0.45DL = 1.65DL$

COMB7	ADD				1.65DL+0.5LL+0.3EQX-EQY
		DL	1.6500	STATIC(DEAD)	
		LL	0.5000	STATIC(LIVE)	
		EQX	0.3000	STATIC(QUAKE)	
		EQY	-1.0000	STATIC(QUAKE)	
COMB8	ADD				1.65DL+0.5LL-0.3EQX+EQY
		DL	1.6500	STATIC(DEAD)	
		LL	0.5000	STATIC(LIVE)	
		EQX	-0.3000	STATIC(QUAKE)	
		EQY	1.0000	STATIC(QUAKE)	
COMB9	ADD				1.65DL+0.5LL-0.3EQX-EQY
		DL	1.6500	STATIC(DEAD)	
		LL	0.5000	STATIC(LIVE)	
		EQX	-0.3000	STATIC(QUAKE)	
		EQY	-1.0000	STATIC(QUAKE)	
COMB10	ADD				0.45DL+EQX+0.3EQY
		DL	0.4500	STATIC(DEAD)	
		EQX	1.0000	STATIC(QUAKE)	
		EQY	0.3000	STATIC(QUAKE)	
COMB11	ADD				0.45DL+EQX-0.3EQY
		DL	0.4500	STATIC(DEAD)	
		EQX	1.0000	STATIC(QUAKE)	
		EQY	-0.3000	STATIC(QUAKE)	
COMB12	ADD				0.45DL-EQX+0.3EQY
		DL	0.4500	STATIC(DEAD)	
		EQX	-1.0000	STATIC(QUAKE)	
		EQY	0.3000	STATIC(QUAKE)	
COMB13	ADD				0.45DL-EQX-0.3EQY
		DL	0.4500	STATIC(DEAD)	
		EQX	-1.0000	STATIC(QUAKE)	
		EQY	-0.3000	STATIC(QUAKE)	
COMB14	ADD				0.45DL+0.3EQX+EQY
		DL	0.4500	STATIC(DEAD)	
		EQX	0.3000	STATIC(QUAKE)	
		EQY	1.0000	STATIC(QUAKE)	

COMB15	ADD			0.45DL+0.3EQX-EQY
		DL	0.4500	STATIC(DEAD)
		EQX	0.3000	STATIC(QUAKE)
		EQY	-1.0000	STATIC(QUAKE)
COMB16	ADD			0.45DL-0.3EQX+EQY
		DL	0.4500	STATIC(DEAD)
		EQX	-0.3000	STATIC(QUAKE)
		EQY	1.0000	STATIC(QUAKE)
COMB17	ADD			0.45DL-0.3EQX-EQY
		DL	0.4500	STATIC(DEAD)
		EQX	-0.3000	STATIC(QUAKE)
		EQY	-1.0000	STATIC(QUAKE)

SAP2000 v7.44 File: PAVILION Kip-in Units PAGE 2
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F R A M E   E L E M E N T   F O R C E S

FRAME	LOAD	LOC	P	V2	V3	T	M2	M3
1	Minima		-31.36	-8.01	-8.34	-5.34	-522.86	-500.09
			COMB5	COMB4	COMB9	COMB6	COMB9	COMB4
1	Maxima	Max. Tension @ Column →	21.34	7.81	8.18	5.09	516.98	492.48
			COMB10	COMB11	COMB14	COMB17	COMB14	COMB11
2	Minima		-23.88	-8.55	-7.58	-6.49	-489.29	-520.61
			COMB7	COMB4	COMB9	COMB6	COMB9	COMB4
2	Maxima		8.74	8.38	7.43	6.34	483.78	514.02
			EQY	COMB11	COMB14	COMB17	COMB14	COMB11
3	Minima		-20.50	-8.94	-6.48	-5.65	-437.50	-540.04
			COMB2	COMB12	COMB9	COMB2	COMB9	COMB3
3	Maxima		-5.952E-01	8.96	6.37	5.64	432.58	538.37
			LL	COMB3	EQY	COMB13	EQY	COMB12
4	Minima		-20.22	-8.94	-6.48	-5.65	-437.45	-537.14
			COMB4	COMB5	COMB7	COMB11	COMB7	COMB10
4	Maxima		2.68	8.92	6.37	5.66	432.58	538.81

		EQX	COMB10	EQY	COMB4	EQY	COMB5
5	Minima	-23.83	-8.26	-7.58	-6.37	-489.30	-506.94
		COMB9	COMB13	COMB7	COMB15	COMB7	COMB13
5	Maxima	8.74	8.43	7.44	6.51	483.79	513.53
		EQY	COMB2	COMB16	COMB8	COMB16	COMB2
6	Minima	-31.05	-7.67	-8.34	-5.12	-522.92	-483.47
		COMB3	COMB13	COMB7	COMB15	COMB7	COMB13
6	Maxima	21.03	7.86	8.19	5.37	517.04	491.08
		COMB12	COMB2	COMB16	COMB8	COMB16	COMB2
7	Minima	-28.51	-10.24	-8.41	-10.30	-522.73	-628.83
		COMB4	COMB5	COMB17	COMB10	COMB17	COMB5
7	Maxima	20.54	9.53	8.60	10.61	530.26	601.46
		COMB11	EQX	COMB6	COMB5	COMB6	EQX
8	Minima	$\boxed{-46.24}$	$\boxed{-10.99}$	-7.95	-7.21	-502.48	-657.55
		$\boxed{COMB2}$	$\boxed{COMB5}$	COMB17	COMB10	COMB17	~~COMB5~~
8	Maxima	8.35	10.14	8.13	7.43	509.97	$\boxed{660.82}$
		COMB13	EQX	COMB6	COMB5	COMB6	$\boxed{COMB5}$
11	Minima	-45.77	-9.88	-7.95	-7.35	-502.47	-655.17
		COMB4	COMB12	COMB15	COMB3	COMB15	COMB3
11	Maxima	13.33	10.88	8.13	7.13	509.96	650.93
		EQX	COMB3	COMB8	COMB12	COMB8	COMB3
12	Minima	-28.17	-9.28	-8.41	-10.50	-522.68	-592.12
		COMB2	COMB12	COMB15	COMB3	COMB15	COMB3
12	Maxima	20.21	10.10	8.60	10.20	530.21	620.17
		COMB13	COMB3	COMB8	COMB12	COMB8	COMB3
13	Minima	-23.29	-7.42	-8.90	-3.88	-544.03	-477.78
		COMB4	COMB4	COMB9	COMB2	COMB9	COMB4
13	Maxima	8.73	7.22	8.74	3.56	538.03	470.67
		EQX	EQX	COMB14	COMB13	COMB14	EQX
14	Minima	-20.69	-7.00	-9.37	-8.08	-564.65	-469.66
		COMB6	COMB5	COMB17	COMB3	COMB6	COMB5
14	Maxima	-2.716E-01	6.72	9.39	7.96	563.05	458.48
		COMB17	EQX	COMB6	COMB12	COMB17	EQX

Max Compression @ Column

Max. Shear @ Column.

Max. Moment @ Column.

15	Minima	-18.02	-7.28	-9.37	-10.82	-562.52	-485.69
		COMB7	COMB5	COMB9	COMB11	COMB14	COMB5
15	Maxima	8.412E-01	6.94	9.36	10.85	563.57	472.41
		EQY	EQX	COMB14	COMB4	COMB9	EQX
16	Minima	-22.75	-9.14	-8.85	-12.70	-539.85	-578.52
		COMB5	COMB5	COMB17	COMB11	COMB17	COMB5
16	Maxima	9.51	8.57	9.03	13.03	547.16	556.45
		EQX	EQX	COMB6	COMB4	COMB6	EQX
17	Minima	-23.15	-7.07	-8.90	-3.68	-544.08	-461.02
		COMB2	COMB13	COMB7	COMB11	COMB7	COMB13
17	Maxima	7.70	7.29	8.74	3.99	538.08	469.40
		COMB13	COMB2	COMB16	COMB4	COMB16	COMB2
18	Minima	-20.68	-6.56	-9.37	-7.98	-564.66	-450.07
		COMB8	COMB12	COMB15	COMB10	COMB8	COMB12
18	Maxima	5.961E-01	6.89	9.39	8.11	563.06	462.23
		EQX	COMB3	COMB8	COMB5	COMB15	COMB3
19	Minima	-18.02	-6.77	-9.37	-10.83	-562.52	-463.51
		COMB9	COMB12	COMB7	COMB2	COMB16	COMB12
19	Maxima	8.412E-01	7.17	9.36	10.79	563.56	478.29
		EQY	COMB3	COMB16	COMB13	COMB7	COMB3
20	Minima	-22.59	-8.34	-8.85	-12.92	-539.81	-544.49
		COMB3	COMB12	COMB15	COMB2	COMB15	COMB12
20	Maxima	8.09	9.02	9.03	12.58	547.12	570.38
		COMB12	COMB3	COMB8	COMB13	COMB8	COMB3
21	Minima	-55.18	-30.13	-14.54	-754.50	-425.92	-1256.08
		COMB3	COMB4	COMB9	COMB16	COMB9	COMB12
21	Maxima	54.33	30.55	14.28	775.89	405.33	1283.15
		COMB12	COMB3	COMB14	COMB7	COMB14	COMB3
22	Minima	-46.90	-26.83	-6.97	-352.48	-488.93	-1081.59
		COMB3	COMB5	COMB9	COMB16	COMB8	COMB13
22	Maxima	45.88	29.82	6.84	362.46	484.78	1089.97
		COMB12	COMB2	COMB14	COMB7	COMB15	COMB2
24	Minima	-29.04	-29.55	-6.84	-362.45	-487.64	-1064.97
		COMB3	COMB4	COMB16	COMB9	COMB6	COMB11

24	Maxima	28.03	26.56	6.97	352.48	483.49	1073.34
		COMB12	COMB3	COMB7	COMB14	COMB17	COMB4
25	Minima	-20.73	-30.11	-14.28	-775.82	-424.59	-1234.25
		COMB3	COMB5	COMB16	COMB9	COMB7	COMB10
25	Maxima	19.88	29.70	14.54	754.43	404.00	1261.31
		COMB12	COMB2	COMB7	COMB14	COMB16	COMB5
26	Minima	-48.75	-29.70	-9.35	-481.30	-683.29	-1540.82
		COMB2	COMB5	COMB15	COMB6	COMB11	COMB5
26	Maxima	46.96	34.50	9.54	466.28	700.76	1322.49
		COMB13	COMB2	COMB8	COMB17	COMB4	EQX
27	Minima	-39.35	-36.39	-5.61	-50.54	-497.64	-1521.33
		COMB2	COMB4	COMB2	COMB10	COMB10	COMB4
27	Maxima	36.55	13.51	5.61	50.54	509.82	1177.34
		COMB13	EQX	COMB5	COMB5	COMB5	EQX
29	Minima	-39.35	-12.42	-5.61	-50.54	-493.29	-1510.54
		COMB2	COMB12	COMB2	COMB2	COMB12	COMB2
29	Maxima	36.55	36.39	5.61	50.54	505.46	1105.97
		COMB13	COMB3	COMB13	COMB13	COMB3	COMB13
30	Minima	-28.57	-34.03	-9.54	-466.22	-678.91	-1522.84
		COMB2	COMB4	COMB6	COMB15	COMB13	COMB3
30	Maxima	26.78	29.22	9.35	481.23	696.38	1279.53
		COMB13	COMB3	COMB17	COMB8	COMB2	COMB12
31	Minima	-52.81	-30.94	-12.59	-804.70	-400.14	-1246.05
		COMB8	COMB7	COMB10	COMB12	COMB14	COMB7
31	Maxima	52.40	29.45	13.24	847.73	420.97	1212.17
		COMB15	COMB8	COMB5	COMB3	COMB9	COMB16
32	Minima	-44.36	-27.70	-5.44	-402.59	-547.33	-1092.27
		COMB8	COMB9	COMB10	COMB12	COMB4	COMB9
32	Maxima	43.79	29.87	5.87	463.79	519.68	1073.31
		COMB15	COMB6	COMB5	COMB3	COMB11	COMB14
34	Minima	-26.28	-27.08	-9.75	-308.55	-497.87	-895.36
		COMB8	COMB9	COMB4	EQX	COMB4	COMB16
34	Maxima	25.71	24.55	9.45	423.19	467.67	897.04
		COMB15	COMB6	COMB11	COMB5	COMB11	COMB7

Max. Shear @ Beam.

Max. Moment @ Beam.

35	Minima	-17.69	-25.09	-18.81	-780.16	-693.41	-983.60
		COMB8	COMB9	COMB4	EQX	COMB11	COMB6
35	Maxima	17.31	27.73	17.84	941.33	711.18	953.03
		COMB15	COMB6	COMB11	COMB5	COMB4	COMB17
36	Minima	-52.82	-30.90	-13.00	-834.00	-419.61	-1245.86
		COMB6	COMB9	COMB3	COMB5	COMB7	COMB9
36	Maxima	52.40	29.42	12.35	790.96	398.77	1211.98
		COMB17	COMB6	COMB12	COMB10	COMB16	COMB14
37	Minima	-44.36	-27.70	-5.76	-457.06	-510.19	-1091.51
		COMB6	COMB7	COMB3	COMB5	COMB13	COMB7
37	Maxima	43.79	29.86	5.33	395.86	537.84	1072.55
		COMB17	COMB8	COMB12	COMB10	COMB2	COMB16
39	Minima	-26.28	-27.09	-9.34	-419.07	-458.22	-894.50
		COMB6	COMB7	COMB13	COMB3	COMB13	COMB14
39	Maxima	25.72	24.56	9.64	297.50	488.41	896.19
		COMB17	COMB8	COMB2	COMB12	COMB2	COMB9
40	Minima	-17.70	-25.05	-17.61	-930.72	-706.69	-983.42
		COMB6	COMB7	COMB13	COMB3	COMB2	COMB8
40	Maxima	17.32	27.69	18.58	754.21	688.92	952.85
		COMB17	COMB8	COMB2	COMB12	COMB13	COMB15
41	Minima	-37.93	-19.88	-1.64	-79.60	-489.13	-300.69
		COMB3	COMB5	COMB12	COMB12	COMB8	COMB5
41	Maxima	36.94	15.79	1.64	79.60	484.98	216.04
		COMB12	COMB2	COMB3	COMB11	COMB15	COMB10
42	Minima	-37.93	-15.79	-1.64	-79.60	-487.84	-302.87
		COMB3	COMB13	COMB4	COMB4	COMB6	COMB3
42	Maxima	36.94	19.88	1.64	79.60	483.70	218.22
		COMB12	COMB2	COMB11	COMB11	COMB17	COMB12
43	Minima	-39.35	-17.60	-5.61	-50.54	-86.71	-82.41
		COMB2	COMB4	COMB2	COMB10	COMB10	EQY
43	Maxima	36.55	13.51	5.61	50.54	98.88	915.97
		COMB13	COMB3	COMB13	COMB5	COMB5	COMB3
44	Minima	-39.35	-13.51	-5.61	-50.54	-82.35	-176.98

		COMB2	COMB12	COMB2	COMB10	COMB12	EQX
44	Maxima	36.55	17.60	5.61	50.54	94.53	905.18
		COMB13	COMB3	COMB13	COMB13	COMB3	COMB5
45	Minima	-35.28	-19.93	-2.88	-57.42	-555.21	-278.17
		COMB8	COMB9	COMB16	EQY	COMB4	COMB9
45	Maxima	34.73	15.72	2.97	142.78	527.69	204.41
		COMB15	COMB14	COMB7	COMB7	COMB11	COMB14
46	Minima	-35.28	-15.84	-2.88	-57.42	-521.11	-300.17
		COMB8	COMB9	COMB16	EQY	COMB4	COMB8
46	Maxima	34.73	19.74	2.97	142.78	492.27	229.79
		COMB15	COMB6	COMB7	COMB7	COMB11	COMB15
47	Minima	-35.29	-19.93	-2.97	-142.40	-518.23	-277.81
		COMB6	COMB7	COMB9	COMB9	COMB13	COMB7
47	Maxima	34.73	15.72	2.88	57.42	545.75	204.05
		COMB17	COMB16	COMB14	EQY	COMB2	COMB16
48	Minima	-35.29	-15.84	-2.97	-142.40	-482.83	-299.79
		COMB6	COMB7	COMB9	COMB9	COMB13	COMB6
48	Maxima	34.73	19.74	2.88	57.42	511.67	229.41
		COMB17	COMB8	COMB14	EQY	COMB2	COMB17

SAP2000

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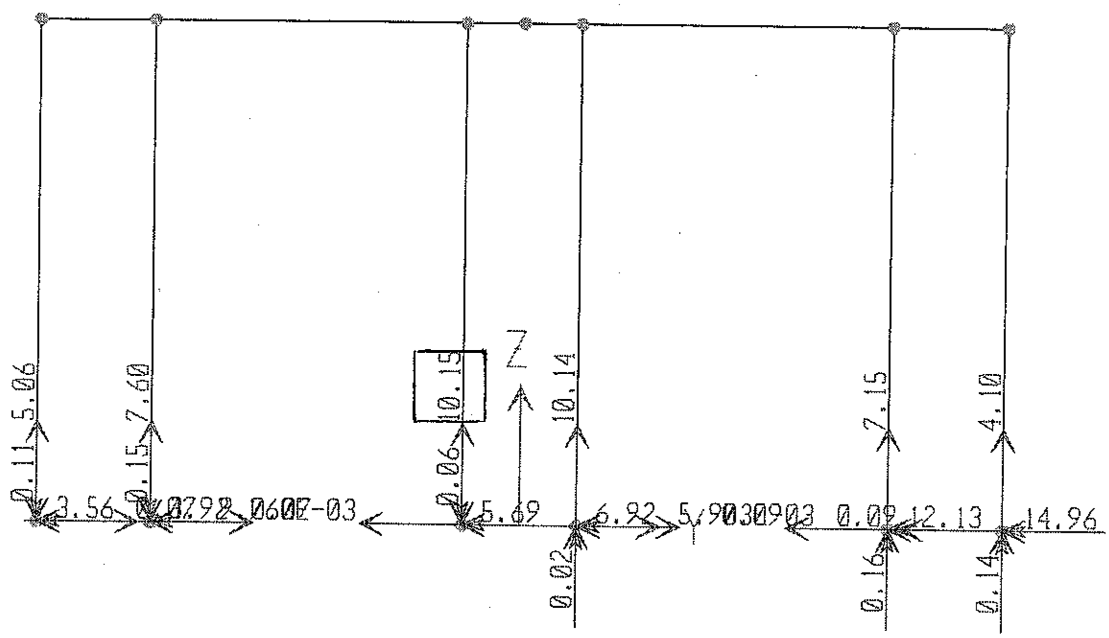
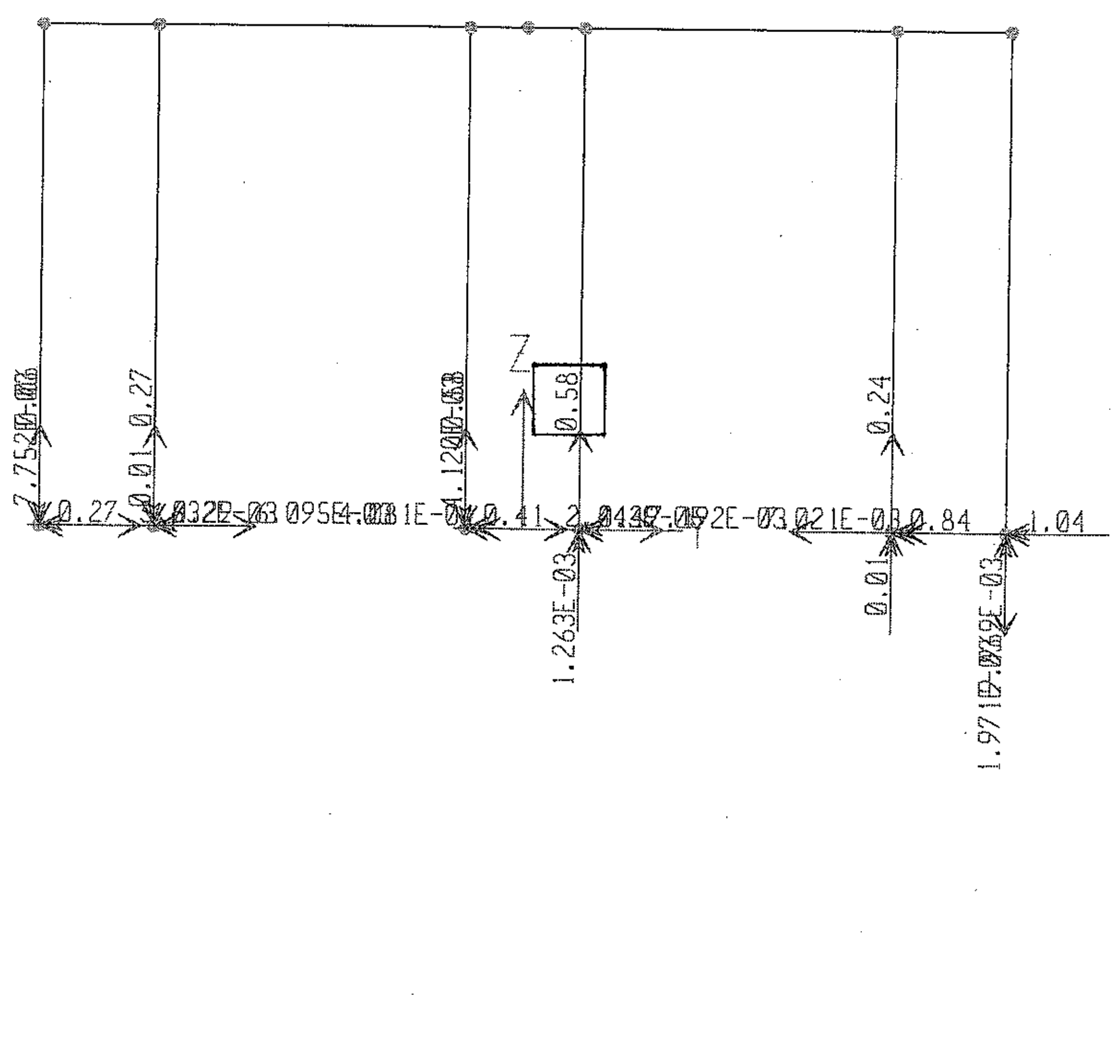


SAP2000 v7.44 - File:Pavilion - Restraint Reactions (DL) - Kip-in Units



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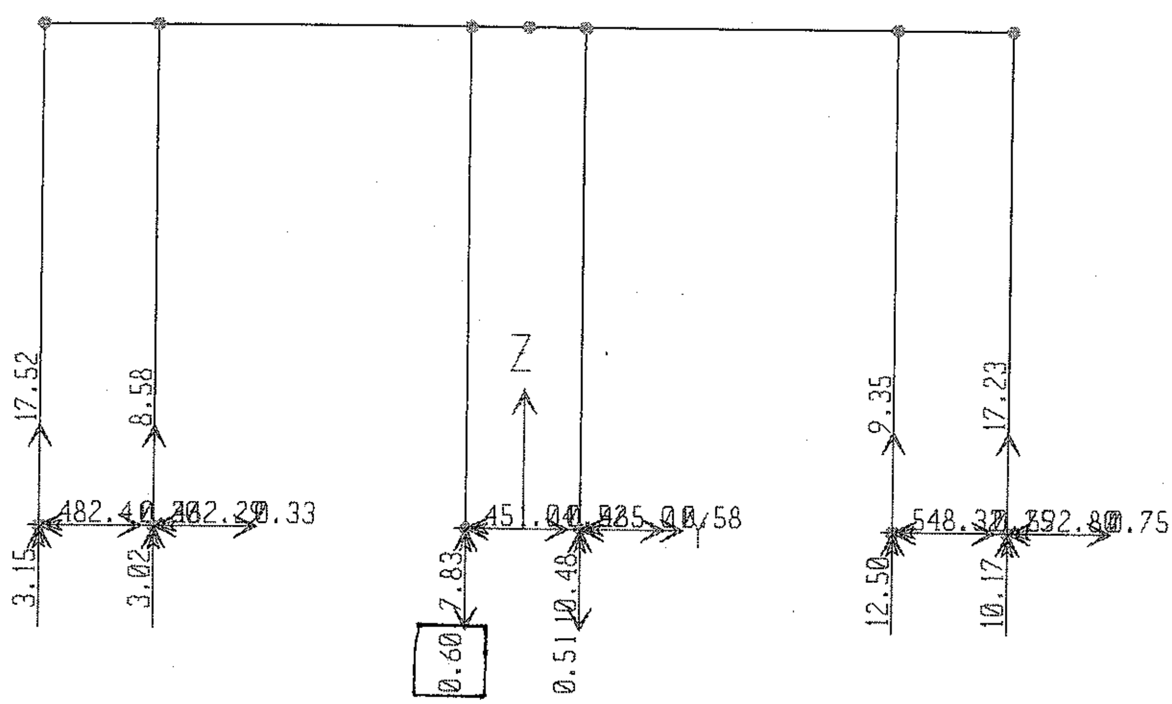

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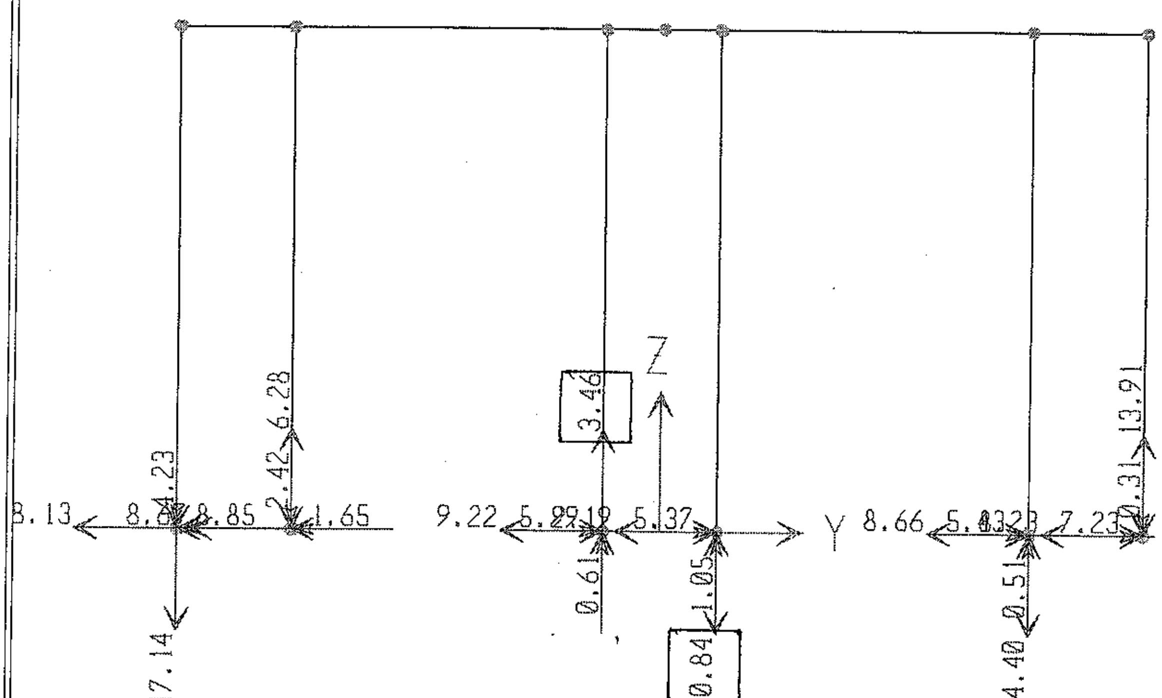

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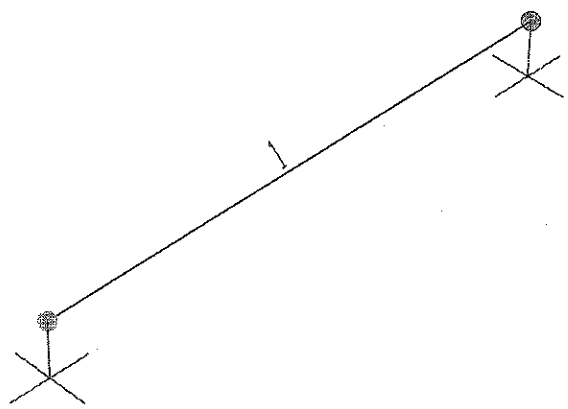

SAP2000 v7.44 - File:PavilionFloorBeam - 3-D View - Kip-in Units

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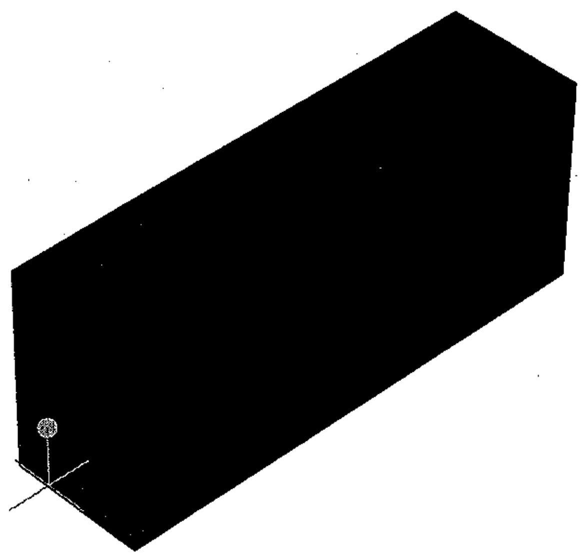
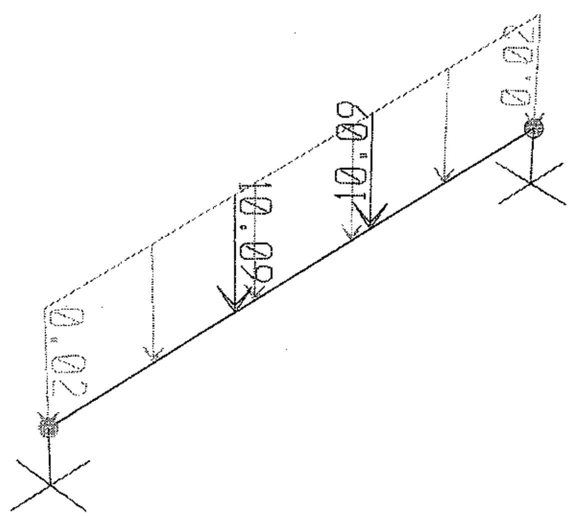


SAP2000 v7.44 - File:PavilionFloorBeam - 3-D View - Kip-in Units

SAP2000

6/11/04 8:37:51



SAP2000 v7.44 - File:PavilionFloorBeam - Frame Span Loads (DL) - Kip-ft Units

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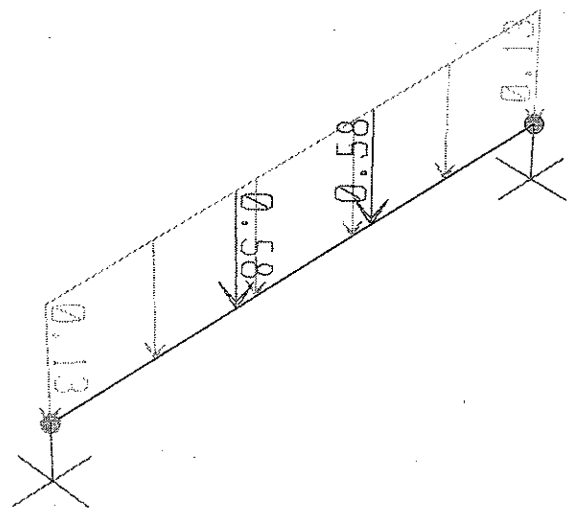
SAP2000

6/11/04 8:37:59



SAP2000 v7.44 - File:PavilionFloorBeam - Frame Span Loads (LL) - Kip-ft Units

25

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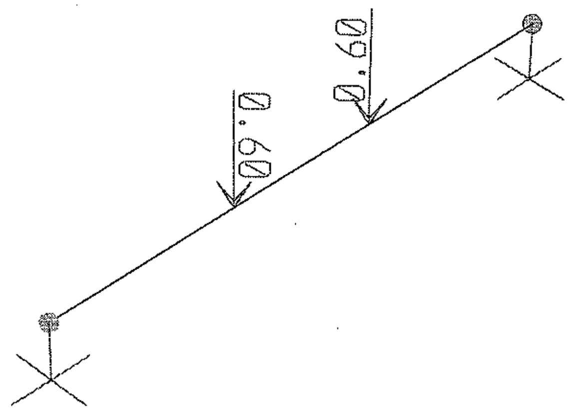

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SAP2000

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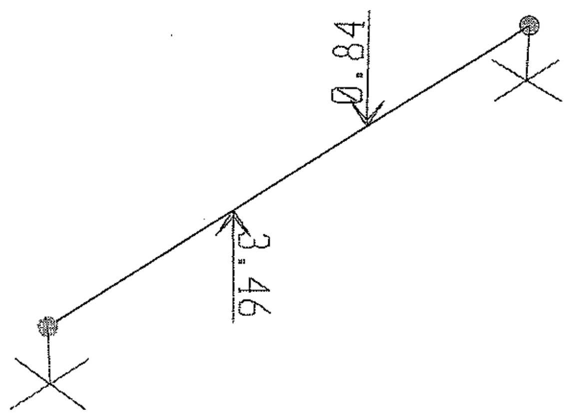


SAP2000 v7.44 - File:PavilionFloorBeam - Frame Span Loads (EQY) - Kip-ft Units

L O A D   C O M B I N A T I O N   M U L T I P L I E R S

COMBO	TYPE	CASE	FACTOR	TYPE	TITLE
COMB1	ADD				1.4DL+1.7LL
		DL	1.4000	STATIC(DEAD)	
		LL	1.7000	STATIC(LIVE)	
COMB2	ADD				1.65DL+0.5LL+EQX+0.3EQY
		DL	1.6500	STATIC(DEAD)	
		LL	0.5000	STATIC(LIVE)	
		EQX	1.0000	STATIC(QUAKE)	
		EQY	0.3000	STATIC(QUAKE)	
COMB3	ADD				1.65DL+0.5LL+EQX-0.3EQY
		DL	1.6500	STATIC(DEAD)	
		LL	0.5000	STATIC(LIVE)	
		EQX	1.0000	STATIC(QUAKE)	
		EQY	-0.3000	STATIC(QUAKE)	
COMB4	ADD				1.65DL+0.5LL-EQX+0.3EQY
		DL	1.6500	STATIC(DEAD)	
		LL	0.5000	STATIC(LIVE)	
		EQX	-1.0000	STATIC(QUAKE)	
		EQY	0.3000	STATIC(QUAKE)	
COMB5	ADD				1.65DL+0.5LL-EQX-0.3EQY
		DL	1.6500	STATIC(DEAD)	
		LL	0.5000	STATIC(LIVE)	
		EQX	-1.0000	STATIC(QUAKE)	
		EQY	-0.3000	STATIC(QUAKE)	
COMB6	ADD				1.65DL+0.5LL+0.3EQX+EQY
		DL	1.6500	STATIC(DEAD)	
		LL	0.5000	STATIC(LIVE)	
		EQX	0.3000	STATIC(QUAKE)	
		EQY	1.0000	STATIC(QUAKE)	

27

COMB7	ADD				1.65DL+0.5LL+0.3EQX-EQY
		DL	1.6500	STATIC(DEAD)	
		LL	0.5000	STATIC(LIVE)	
		EQX	0.3000	STATIC(QUAKE)	
		EQY	-1.0000	STATIC(QUAKE)	
COMB8	ADD				1.65DL+0.5LL-0.3EQX+EQY
		DL	1.6500	STATIC(DEAD)	
		LL	0.5000	STATIC(LIVE)	
		EQX	-0.3000	STATIC(QUAKE)	
		EQY	1.0000	STATIC(QUAKE)	
COMB9	ADD				1.65DL+0.5LL-0.3EQX-EQY
		DL	1.6500	STATIC(DEAD)	
		LL	0.5000	STATIC(LIVE)	
		EQX	-0.3000	STATIC(QUAKE)	
		EQY	-1.0000	STATIC(QUAKE)	
COMB10	ADD				0.45DL+EQX+0.3EQY
		DL	0.4500	STATIC(DEAD)	
		EQX	1.0000	STATIC(QUAKE)	
		EQY	0.3000	STATIC(QUAKE)	
COMB11	ADD				0.45DL+EQX-0.3EQY
		DL	0.4500	STATIC(DEAD)	
		EQX	1.0000	STATIC(QUAKE)	
		EQY	-0.3000	STATIC(QUAKE)	
COMB12	ADD				0.45DL-EQX+0.3EQY
		DL	0.4500	STATIC(DEAD)	
		EQX	-1.0000	STATIC(QUAKE)	
		EQY	0.3000	STATIC(QUAKE)	
COMB13	ADD				0.45 DL-EQX-0.3EQY
		DL	0.4500	STATIC(DEAD)	
		EQX	-1.0000	STATIC(QUAKE)	
		EQY	-0.3000	STATIC(QUAKE)	
COMB14	ADD				0.45DL+0.3EQX+EQY
		DL	0.4500	STATIC(DEAD)	
		EQX	0.3000	STATIC(QUAKE)	
		EQY	1.0000	STATIC(QUAKE)	

COMB15	ADD				0.45DL+0.3EQX-EQY
		DL	0.4500	STATIC(DEAD)	
		EQX	0.3000	STATIC(QUAKE)	
		EQY	-1.0000	STATIC(QUAKE)	
COMB16	ADD				0.45DL-0.3EQX+EQY
		DL	0.4500	STATIC(DEAD)	
		EQX	-0.3000	STATIC(QUAKE)	
		EQY	1.0000	STATIC(QUAKE)	
COMB17	ADD				0.45DL-0.3EQX-EQY
		DL	0.4500	STATIC(DEAD)	
		EQX	-0.3000	STATIC(QUAKE)	
		EQY	-1.0000	STATIC(QUAKE)	

SAP2000 v7.44 File: PAVILIONFLOORBEAM Kip-in Units PAGE 2
6/11/04 8:39:45

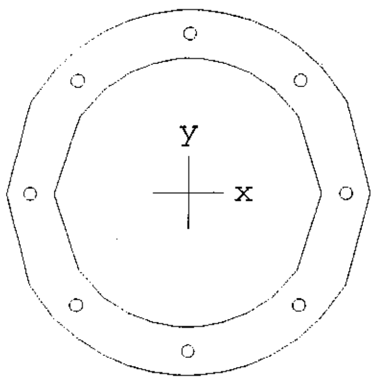
F R A M E E L E M E N T F O R C E S

max Beam shear.

max. Beam Moment

FRAME	LOAD	LOC	P	V2	V3	T	M2	M3
1	Minima		0.00	$\boxed{-25.54}$	0.00	0.00	0.00	$\boxed{-517.91}$
			COMB17	$\boxed{\text{COMB7}}$	COMB3	COMB17	COMB7	$\boxed{\text{COMB7}}$
1	Maxima		0.00	24.01	0.00	0.00	0.00	279.81
			COMB17	COMB3	COMB7	COMB17	COMB7	COMB7

PCACOL V2.30



15.0 x 15.0 inch

f'c = 2.5 ksi

fy = 33.0 ksi

Confinement: Tied

clr cover = NA

spacing = 4.41 in

8 bars at 2.46%

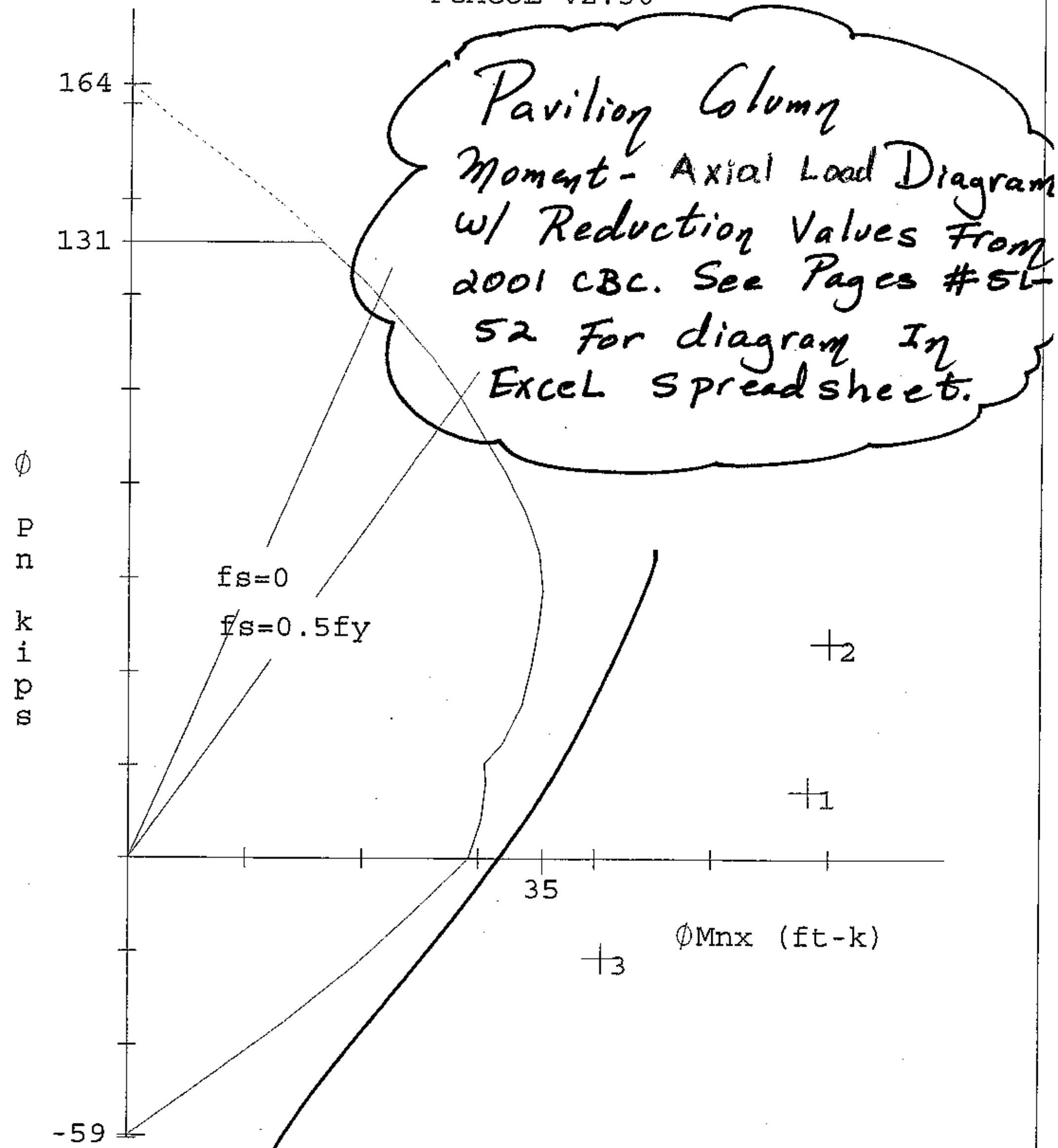
As = 2 in^2

Ix = 1751 in^4

Iy = 1685 in^4

Xo = 0.00 in

Yo = 0.00 in



Licensed To: OLMM, Oakland, CA

File name: C:\FC-JOBS\0404-C~1\PCACOL\PAVCOL.COL

Project: Chabot Tower

Column Id: Pavilion Column

Engineer: Francisco Castillo

Date: 06/04/04 Time: 10:05:44

Code: ACI 318-89

Units: in-lb

Material Properties:

Ec = 2850 ksi eu = 0.003 in/in

fc = 2.13 ksi Es = 29000 ksi

Beta1 = 0.85

Stress Profile: Block

$\boxed{\text{phi(c) = 0.70, phi(b) = 0.90}}$

X-axis slenderness is considered; k(b) = 1.00 k(s) = 2.00

```
0000000     00000     00000     00000     00000    00
00     00  00    00  00    00  00    00  00    00  00
00     00  00    00  00    00  00    00  00    00  00
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00          00000    00    00   00000     00000    00000   (TM)
```

===
Computer program for the Strength Design of Reinforced Concrete Sections
===

General Information:
====================

File Name: C:\FC-JOBS\0404-C~1\PCACOL\PAVCOL.COL
Project: Chabot Tower
Column: Pavilion Column
Engineer: Francisco Castillo
Code: ACI 318-89
Units: US in-lbs
Date: 06/04/04 Time: 10:05:44

Run Option: Investigation
Run Axis: X-axis
Slender column
Column Type: Structural

Material Properties:
====================

f'c = 2.5 ksi
Ec = 2850 ksi
fc = 2.125 ksi
eu = 0.003 in/in
Stress Profile: Block

fy = 33 ksi
Es = 29000 ksi
erup = 0 in/in

Beta1 = 0.85

Geometry:
=========

Exterior Points

Point No.	X-Loc (in)	Y-Loc (in)	Point No.	X-Loc (in)	Y-Loc (in)	Point No.	X-Loc (in)	Y-Loc (in)
1	7.5	0.0	2	6.5	3.7	3	5.5	5.1
4	4.5	6.0	5	3.5	6.6	6	2.5	7.1
7	1.5	7.3	8	0.5	7.5	9	0.0	7.5
10	-0.5	7.5	11	-1.5	7.3	12	-2.5	7.1
13	-3.5	6.6	14	-4.5	6.0	15	-5.5	5.1
16	-6.5	3.7	17	-7.5	0.0	18	-6.5	-3.7
19	-5.5	-5.1	20	-4.5	-6.0	21	-3.5	-6.6
22	-2.5	-7.1	23	-1.5	-7.3	24	-0.5	-7.5
25	0.0	-7.5	26	0.5	-7.5	27	1.5	-7.3
28	2.5	-7.1	29	3.5	-6.6	30	4.5	-6.0
31	5.5	-5.1	32	6.5	-3.7			

Interior Points

Point No.	X-Loc (in)	Y-Loc (in)	Point No.	X-Loc (in)	Y-Loc (in)	Point No.	X-Loc (in)	Y-Loc (in)
1	5.5	0.0	2	4.5	3.2	3	3.5	4.2
4	2.5	4.9	5	1.5	5.3	6	0.5	5.5
7	0.0	5.5	8	-0.5	5.5	9	-1.5	5.3
10	-2.5	4.9	11	-3.5	4.2	12	-4.5	3.2
13	-5.5	0.0	14	-4.5	-3.2	15	-3.5	-4.2
16	-2.5	-4.9	17	-1.5	-5.3	18	-0.5	-5.5
19	0.0	-5.5	20	0.5	-5.5	21	1.5	-5.3
22	2.5	-4.9	23	3.5	-4.2	24	4.5	-3.2

Gross section area, Ag = 81.2 in^2
Ix = 1751.43 in^4
Iy = 1684.63 in^4
Xo = 0 in
Yo = 1.99661e-007 in

Reinforcement:
==============

Rebar Database: ASTM

Continued from previous page...

Size	Diam	Area	Size	Diam	Area	Size	Diam	Area
3	0.38	0.11	4	0.50	0.20	5	0.63	0.31
6	0.75	0.44	7	0.88	0.60	8	1.00	0.79
9	1.13	1.00	10	1.27	1.27	11	1.41	1.56
14	1.69	2.25	18	2.26	4.00			

Confinement: Tied; phi(c) = 0.7, phi(b) = 0.9, a = 0.8
#3 ties with #10 bars, #3 with larger bars.

Pattern: Irregular

Total steel area, As = 2.00 in^2 at 2.46%

Area (in^2)	X-Loc (in)	Y-Loc (in)	Area (in^2)	X-Loc (in)	Y-Loc (in)	Area (in^2)	X-Loc (in)	Y-Loc (in)
0.25	6.5	0.0	0.25	4.6	4.6	0.25	0.0	6.5
0.25	-4.6	4.6	0.25	-6.5	0.0	0.25	-4.6	-4.6
0.25	0.0	-6.5	0.25	4.6	-4.6			

35

Slenderness:

X-axis: Unbraced against sidesway -- Not hinged at either end.

Columns:

Col.	Axis	Height (ft)	Width (in)	Depth (in)	I (in^4)	f'c (ksi)	Ec (ksi)
Design	X	10	0	0	1751.43	2.5	2850
Above		(NO COLUMN SPECIFIED...)					
Below		(NO COLUMN SPECIFIED...)					

Beams:

X-Beams Location	Length (ft)	Width (in)	Depth (in)	I (in^4)	f'c (ksi)	Ec (ksi)
Above Left	6.1	33	24	38016	1.875	2625.13
Above Right	2.25	33	24	38016	1.875	2625.13
Below Left	(NO BEAM SPECIFIED...)					
Below Right	(NO BEAM SPECIFIED...)					

Effective Length Factors:

Axis	Psi(top)	Psi(bot)	k(Braced)	k(Sway)	klu/r
X	0.000	0.000	1.000	2.000	51.7

Moment Magnification Factors:

Beta(d) load case factors: Dead = 1.4, Live = 1.7
Strength reduction factor = 0.7
Sum of Pc = 18.00*Pc; Sum of Pu = 18.00*Pu

	Braced (X-axis)					Sway (X-axis)		
Load Comb	Pc (kip)	Betad	EI (k-in^2)	Cm	Delta	Pc (kip)	EI (k-in^2)	Delta
1 U1	1525	0.000	2.22e+006	1.000	1.014	381	2.22e+006	N/A
2 U1	1525	0.000	2.22e+006	1.000	1.045	381	2.22e+006	N/A
3 U1	1525	0.000	2.22e+006	1.000	1.000	381	2.22e+006	N/A

Load Combinations:

U1 = 1.000*Dead + 1.000*Live + 1.000*Lateral

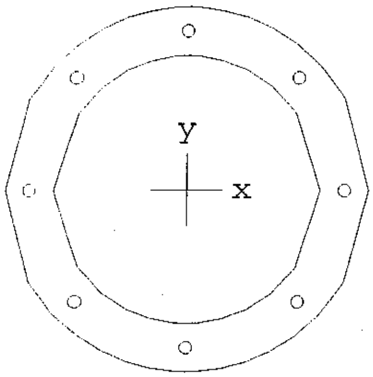
Service Loads:

No.	Load Case	Axial Load (kip)	Moments about X-axis @ Top (ft-k)	Moments about X-axis @ Bot (ft-k)	Moments about Y-axis @ Top (ft-k)	Moments about Y-axis @ Bot (ft-k)
1	Dead	0	0	0	0	0
	Live	0	0	0	0	0
	Latl	14.54	55.1	-54.8	0	0
2	Dead	0	0	0	0	0
	Live	0	0	0	0	0
	Latl	46.24	44.37	-49.67	0	0
3	Dead	0	0	0	0	0
	Live	0	0	0	0	0
	Latl	-21.34	36.15	-40.6	0	0

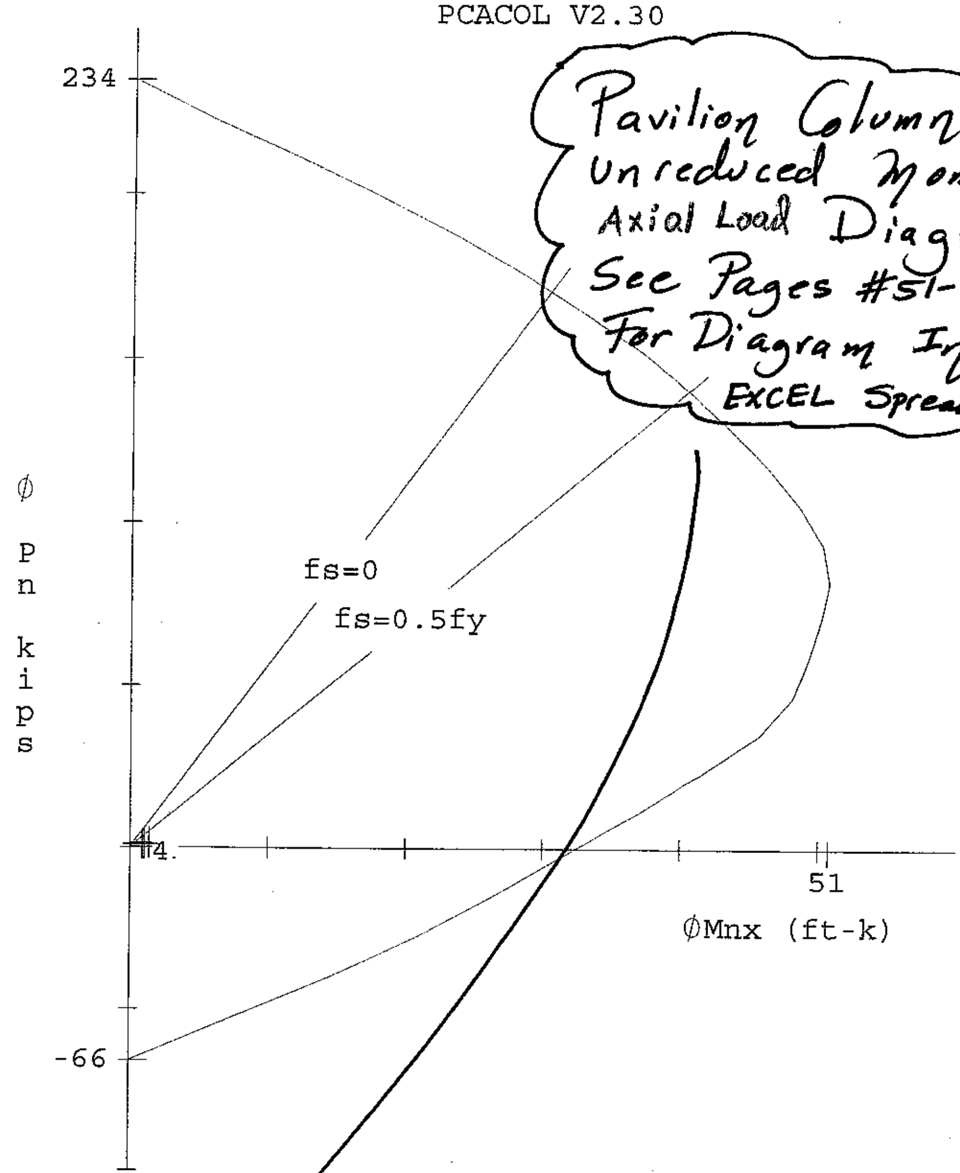
Pt.	Load Comb	Applied Loads P (kips)	Applied Loads Mx (ft-k)	Computed Strength P (kips)	Computed Strength Mx (ft-k)	Computed/ Applied Ray length
1	1 U1	15	58	7	30	0.516
2	2 U1	46	60	24	32	0.530
3	3 U1	-21	41	-12	24	0.590

Program completed as requested!

PCACOL V2.30



15.0 x 15.0 inch



Pavilion Column Unreduced Moment Axial Load Diagram. See Pages #51-52 For Diagram In EXCEL Spreadsheet

f'c = 2.5 ksi

fy = 33.0 ksi

Confinement: Other

clr cover = NA

spacing = 4.41 in

8 bars at 2.46%

As = 2 in^2

Ix = 1751 in^4

Iy = 1685 in^4

Xo = 0.00 in

Yo = 0.00 in

File name: C:\FC-JOBS\0404-C~1\PCACOL\PAVCOL.COL

Project: Chabot Tower

Column Id: Pavilion Column

Engineer: Francisco Castillo

Date: 06/04/04 Time: 10:05:44

Code: ACI 318-89

Units: in-lb

Material Properties:

Ec = 2850 ksi eu = 0.003 in/in

fc = 2.13 ksi Es = 29000 ksi

Beta1 = 0.85

Stress Profile: Block

phi(c) = 1.00, phi(b) = 1.00

X-axis slenderness is considered; k(b) = 1.00 k(s) = 2.00

```
OOOOOOO   OOOOO   OOOOO   OOOOO   OOOOO   OO
OO   OO  OO   OO OO   OO OO   OO OO   OO  OO
OO   OO  OO   OO OO   OO OO   OO OO   OO  OO
OO   OO  OO      OO   OO OO       OO   OO OO
OO   OO  OO      OOOOOOO OO       OO   OO OO
OOOOOOO  OO   OO OO   OO OO   OO  OO   OO OO
OO       OO   OO OO   OO OO   OO  OO   OO OO
OO        OOOOO  OO   OO  OOOOO    OOOOO  OOOOO  (TM)
```

==
Computer program for the Strength Design of Reinforced Concrete Sections
==

40

General Information:
====================

File Name: C:\FC-JOBS\0404-C~1\PCACOL\PAVCOL.COL
Project: Chabot Tower — Code: ACI 318-89
Column: Pavilion Column — Units: US in-lbs
Engineer: Francisco Castillo — Date: 06/04/04 Time: 10:05:44

Run Option: Investigation — Slender column
Run Axis: X-axis — Column Type: Structural

Material Properties:
====================

f'c = 2.5 ksi
Ec = 2850 ksi
fc = 2.125 ksi
eu = 0.003 in/in
Stress Profile: Block

fy = 33 ksi
Es = 29000 ksi
erup = 0 in/in

Beta1 = 0.85

Geometry:
=========

Exterior Points

Point No.	X-Loc (in)	Y-Loc (in)	Point No.	X-Loc (in)	Y-Loc (in)	Point No.	X-Loc (in)	Y-Loc (in)
1	7.5	0.0	2	6.5	3.7	3	5.5	5.1
4	4.5	6.0	5	3.5	6.6	6	2.5	7.1
7	1.5	7.3	8	0.5	7.5	9	0.0	7.5
10	-0.5	7.5	11	-1.5	7.3	12	-2.5	7.1
13	-3.5	6.6	14	-4.5	6.0	15	-5.5	5.1
16	-6.5	3.7	17	-7.5	0.0	18	-6.5	-3.7
19	-5.5	-5.1	20	-4.5	-6.0	21	-3.5	-6.6
22	-2.5	-7.1	23	-1.5	-7.3	24	-0.5	-7.5
25	0.0	-7.5	26	0.5	-7.5	27	1.5	-7.3
28	2.5	-7.1	29	3.5	-6.6	30	4.5	-6.0
31	5.5	-5.1	32	6.5	-3.7			

Interior Points

Point No.	X-Loc (in)	Y-Loc (in)	Point No.	X-Loc (in)	Y-Loc (in)	Point No.	X-Loc (in)	Y-Loc (in)
1	5.5	0.0	2	4.5	3.2	3	3.5	4.2
4	2.5	4.9	5	1.5	5.3	6	0.5	5.5
7	0.0	5.5	8	-0.5	5.5	9	-1.5	5.3
10	-2.5	4.9	11	-3.5	4.2	12	-4.5	3.2
13	-5.5	0.0	14	-4.5	-3.2	15	-3.5	-4.2
16	-2.5	-4.9	17	-1.5	-5.3	18	-0.5	-5.5
19	0.0	-5.5	20	0.5	-5.5	21	1.5	-5.3
22	2.5	-4.9	23	3.5	-4.2	24	4.5	-3.2

Gross section area, Ag = 81.2 in^2
Ix = 1751.43 in^4 — Xo = 0 in
Iy = 1684.63 in^4 — Yo = 1.99661e-007 in

Reinforcement:
==============

Rebar Database: ASTM

Continued from previous page...

Size	Diam	Area	Size	Diam	Area	Size	Diam	Area
3	0.38	0.11	4	0.50	0.20	5	0.63	0.31
6	0.75	0.44	7	0.88	0.60	8	1.00	0.79
9	1.13	1.00	10	1.27	1.27	11	1.41	1.56
14	1.69	2.25	18	2.26	4.00			

Confinement: User-defined; phi(c) = 1, phi(b) = 1, a = 1
#3 ties with #10 bars, #3 with larger bars.

Pattern: Irregular

Total steel area, As = 2.00 in^2 at 2.46%

Area (in^2)	X-Loc (in)	Y-Loc (in)	Area (in^2)	X-Loc (in)	Y-Loc (in)	Area (in^2)	X-Loc (in)	Y-Loc (in)
0.25	6.5	0.0	0.25	4.6	4.6	0.25	0.0	6.5
0.25	-4.6	4.6	0.25	-6.5	0.0	0.25	-4.6	-4.6
0.25	0.0	-6.5	0.25	4.6	-4.6			

Slenderness:
============

X-axis: Unbraced against sidesway -- Not hinged at either end.

Columns:

Col.	Axis	Height (ft)	Width (in)	Depth (in)	I (in^4)	f'c (ksi)	Ec (ksi)
Design	X	10	0	0	1751.43	2.5	2850
Above		(NO COLUMN SPECIFIED...)					
Below		(NO COLUMN SPECIFIED...)					

Beams:

X-Beams Location	Length (ft)	Width (in)	Depth (in)	I (in^4)	f'c (ksi)	Ec (ksi)
Above Left	6.1	33	24	38016	1.875	2625.13
Above Right	2.25	33	24	38016	1.875	2625.13
Below Left	(NO BEAM SPECIFIED...)					
Below Right	(NO BEAM SPECIFIED...)					

Effective Length Factors:

Axis	Psi(top)	Psi(bot)	k(Braced)	k(Sway)	klu/r
X	0.000	0.000	1.000	2.000	51.7

Moment Magnification Factors:

Beta(d) load case factors: Dead = 1.4, Live = 1.7
Strength reduction factor = 0.7
Sum of Pc = 18.00*Pc; Sum of Pu = 18.00*Pu

	Braced (X-axis)					Sway (X-axis)		
Load Comb	Pc (kip)	Betad	EI (k-in^2)	Cm	Delta	Pc (kip)	EI (k-in^2)	Delta
1 U1	762	1.000	1.11e+006	0.600	1.000	381	2.22e+006	N/A
U2				0.600	1.000			N/A
U3				0.600	1.000			N/A
U4				0.600	1.000			N/A

Load Combinations:

U1 = 1.400*Dead + 1.700*Live + 0.000*Lateral
U2 = 1.050*Dead + 1.275*Live + 1.275*Lateral
U3 = 1.050*Dead + 0.000*Live + 1.275*Lateral
U4 = 0.900*Dead + 0.000*Live + 1.300*Lateral

Service Loads:

No.	Load Case	Axial Load (kip)	Moments about X-axis @ Top (ft-k)	Moments about X-axis @ Bot (ft-k)	Moments about Y-axis @ Top (ft-k)	Moments about Y-axis @ Bot (ft-k)
1	Dead	1	1	0	0	0
	Live	0	0	0	0	0
	Latl	0	0	0	0	0

Pt.	Load Comb	Applied Loads P (kips)	Applied Loads Mx (ft-k)	Computed Strength P (kips)	Computed Strength Mx (ft-k)	Computed/ Applied Ray length
1	1 U1	1	1	46	48	33.575
2	U2	1	1	46	48	44.766
3	U3	1	1	46	48	44.766
4	U4	1	1	46	48	52.227

Program completed as requested!

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SHEET NO.		OF	
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CHECKED BY		JOB # :	0404

FILENAME C:\FC-Jobs\0404-Chabot Tower\Excel\[MemberForces.xls]Member Forces

Pavilion Member Forces

Member Type	Member Number(1)	Type Of Force	Load Combination(1)	Top Moment(k-ft)(1)	Bot. Moment(k-ft)(1)	Axial Force (k)(1)	Shear Force (k)(1)
Column	8	Max. Moment & Shear	5	55.1	-54.8	14.54	10.99
Column	8	Max. Comp. Force	2	44.37	-49.67	46.24	9.4
Column	1	Max. Tension Force	10	36.15	-40.6	-21.34	7.67

Member Type	Member Number(1)	Type Of Force	Load Combination(1)	Moment(k-ft)(1)	Shear Force (k)(1)
Roof Beam(Supports)	27	Max. Moment & Shear	4	126.78	36.39
Roof Beam(Mid-Span)	29	Max. Moment & Shear	4	84.25	9.42

Member Type	Member Number(1)	Type Of Force	Load Combination(1)	Moment(k-ft)(1)	Shear Force (k)(1)
Pavilion Floor Beam (Supports)	1	Max. Moment & Shear	7	43.16	25.54
Pavilion Floor Beam (Mid-Span)	1	Max. Moment & Shear	7	29.06	0

Member Type	Type Of Force	Shear Force (k)(2)
Pavilion Roof Stem Wall	Max. Shear	73.7

Notes:
1) For member number, load combination, and forces see SAP 2000 output.
2) For member forces see attached hand calculations.

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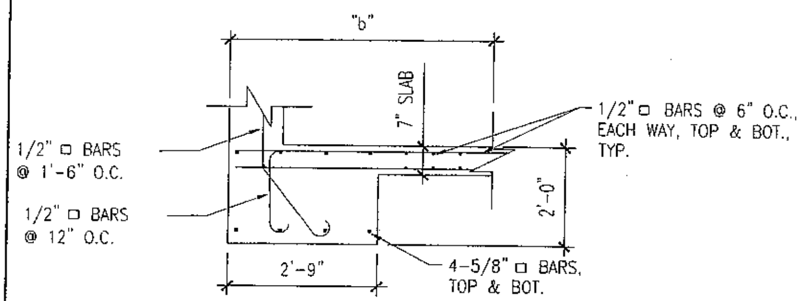
FILENAME C:\FC-Jobs\0404-Chabot Tower\Excel\[ROOFBEAM.xls]Roof Beam

Pavilion Concrete Beam/Roof Beam



Material Properties (Lower Bound):

$f'c$ = 1875 psi
fy = 24750 psi

(Note: Material properties come from the report by Tennebaum-Manehim Engineers dated May 26, 2004.)

Bending Capacity:

bt =	33	in
bb =	33	in
dt =	22.5	in
db =	21	in
Ast =	1.56	in^2

Asb = 1.56 in^2

(Note: The width of the slab that can be used as a T beam was determined as the minimum value of (1/4 x span length of the beam, beam web width + 8 x the slab thickness and beam web width + 1/2 x the clear distance to the next beam web). Since 1/4 x span length of the beam = 1/4 x (6x12 +1 1/4)" = 18.3" we had to use the beam web thickness in our calculations. The value we obtained from the code was less than the beam web thickness and therefore, we can not assume part of the slab works with the beam to create a "T Beam" section.

At Supports: **(The beams have no ties of any type. Therefore, the full bending capacity of the member should not be used since the concrete will spall off and the longitudinal reinforcement will buckle before the member can reach it's full capacity. Furthermore, it is not clear what the development length of the the square bars that form the longitudinal reinforcement actualy is. Because of all these reasons the bending capacity of the members will be reduced by 50%.)**

φMn = (φ*Ast*fy*(dt-((Ast*fy)/(1.7*f'c*bt))))*0.50 = 385 k-in = 32 k-ft
Mu = 126.78 k-ft
φ = 0.9 Demand-To-Capacity Ratio(DCR) = Mu/φMn = 3.96

At Midspan: **(The reduction in capacity used at the supports also applies at midspan.)**

φMn = (φ*Asb*fy*(db-((Asb*fy)/(1.7*f'c*bb))))*0.5 = 358 k-in = 30 k-ft
Mu = 84.25 k-ft
φ = 0.9 Demand-To-Capacity Ratio(DCR) = Mu/φMn = 2.82

Shear Capacity:

(The beam has no ties or shear reinf. of any type. Therefore, only the concrete strength will be used in calculating the shear strength in the members.)

φVn = φVc = φ*(2*(f'c)^1/2*bb*db) = 51013 lbs = 51.01 kips
Vu = 36.39 k
φ = 0.85 Demand-To-Capacity Ratio(DCR) = Vu/φVn = 0.71

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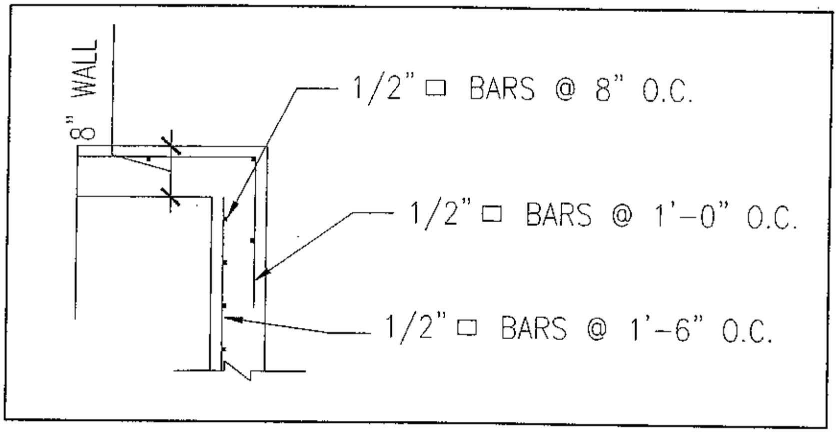
FILENAME *C:\FC-Jobs\0404-Chabot Tower\Excel\[RoofStemWall.xls]Stem Wall*

Pavilion Roof Stem Wall



Material Properties (Lower Bound):

f'c =	**1875**	**psi**
fy =	**24750**	**psi**

(Note: Material properties come from the report by Tennebaum-Manehim Engineers dated May 26, 2004.)

Shear Capacity:

(The wall has no shear reinf. into the roof slab. Therefore, only the concrete strength will be used in calculating the shear strength of the member.)

b = **8** **in** **L =** **136** **in**

φVn = φVc = φ*2*(f'c)^1/2*b*L = 80090 **lbs =** 80.09 **kips**

φ = 0.85 **Vu =** **73.7** **kips**

Demand-To-Capacity Ratio(DCR) = Vu/φVn = 0.92

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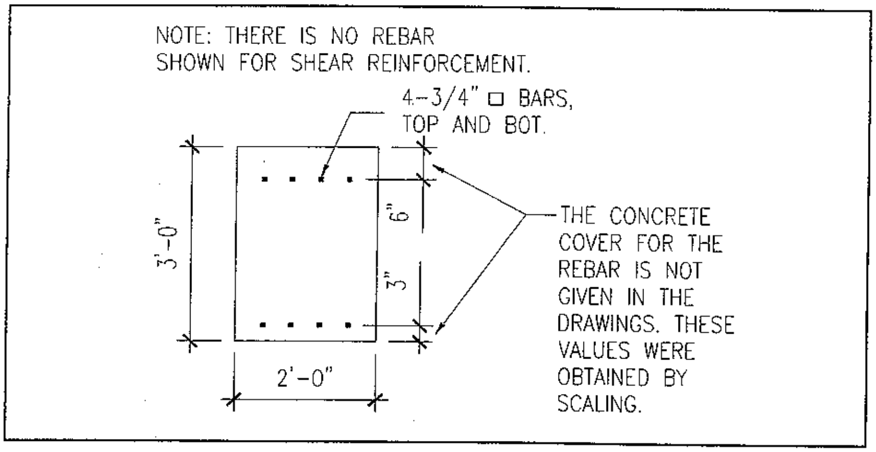
FILENAME C:\FC-Jobs\0404-Chabot Tower\Excel\[WallConnectorBeam.xls]Beam Connectors

Pavilion Concrete Beam/Beam Connecting Walls



Material Properties (Lower Bound):

f'c = 2500 psi
fy = 33000 psi

(Note: Material properties come from the report by Tennebaum-Manehim Engineers dated May 26, 2004.)

Bending Capacity:

b =	24	in	dt =	30	in	db =	33	in
Ast =	1.77	in^2	Asb =	1.77	in^2			

At Supports: (The beams have no ties of any type. Therefore, the full bending capacity of the member should not be used since the concrete will spall off and the longitudinal reinforcement will buckle before the member can reach it's full capacity. Furthermore, it is not clear what the development length of the the square bars that form the longitudinal reinforcement actualy is. Because of all these reasons the bending capacity of the members will be reduced by 50%.)

φMn = (φ*Ast*fy*(dt-((Ast*fy)/(1.7*f'c*b))))*0.5 = 773 **k-in =** 64 **k-ft**
Mu = 43.16 **k-ft**
φ = 0.9 Demand-To-Capacity Ratio(DCR) = Mu/φMn = 0.67

At Midspan:

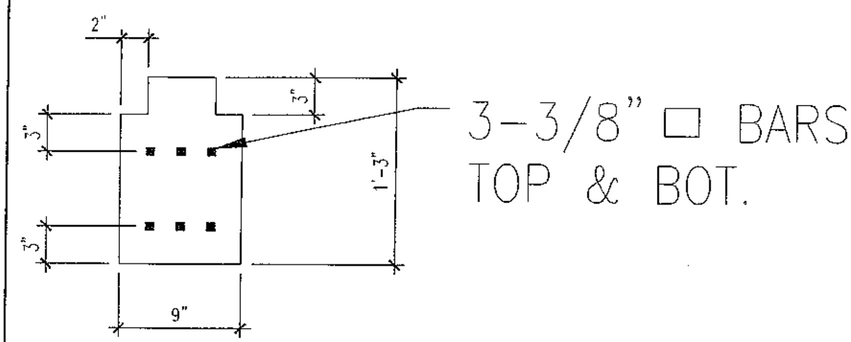
φMn = (φ*Asb*fy*(db-((Asb*fy)/(1.7*f'c*b))))*0.5 = 852 **k-in =** 71 **k-ft**
Mu = 29.06 **k-ft**
φ = 0.9 Demand-To-Capacity Ratio(DCR) = Mu/φMn = 0.41

Shear Capacity:

(The beam has no ties or shear reinf. of any type. Therefore, only the concrete strength will be used in calculating the shear strength in the members.)

φVn = φVc = (φ*2*(f'c)^1/2*b*d) = 61200 **lbs =** 61.2 **kips**
Vu = 25.54 **k**
φ = 0.85 Demand-To-Capacity Ratio(DCR) = Vu/φVn = 0.42

Pavilion Concrete Beam/Interior Rectangular Floor Beam



Material Properties (Lower Bound):

f'c = 2500 psi
fy = 33000 psi

(Note: Material properties come from the report by Tennebaum-Manehim Engineers dated May 26, 2004.)

Bending Capacity:

At Supports:

b = 9 in **dt = 9 in** **Ast = 1.17 in^2**

φMn = φ*Ast*fy*(dt-((Ast*fy)/(1.7*f'c*b))) = 278 **k-in =** 23 **k-ft**
φ = 0.9

At Midspan:

Ast = 1.17 in^2 **a = (Ast*fy-0.85*f'c*3*5)/(0.85*f'c*9)+3 =** 3.35 **in (Depth of conc. block)**
db = 12 in
Centroid of concrete block Xc = 1.79 **in**

fMn = (φ*Asb*fy*(db-Xc)) = 355 **k-in =** 30 **k-ft**
φ = 0.9

Shear Capacity:

Aconc. = 123 in^2

φVn = φVc = (φ*2*(f'c)^1/2*Aconc.) = 10455 **lbs =** 10.46 **kips**
φ = 0.85

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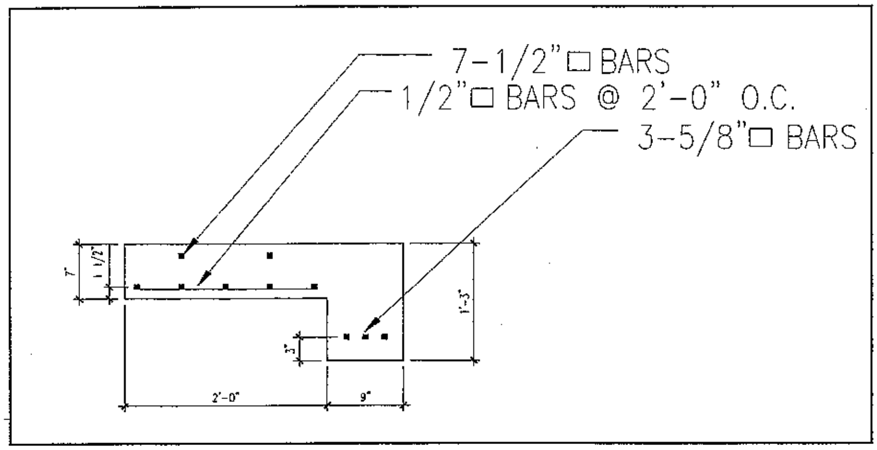
CHECKED BY		JOB # :	0404

FILENAME C:\FC-Jobs\0404-Chabot Tower\Excel\[InteriorPavilionFloorBeam-2.xls]"L" Shape Floor Beam

Pavilion Concrete Beam/"L" Shape Floor Beam



Material Properties (Lower Bound):

f'c = 2500 psi
fy = 33000 psi

(Note: Material properties come from the report by Tennebaum-Manehim Engineers dated May 26, 2004.)

Bending Capacity:

At Supports:

b = 9 in dt = 10.64 in Ast = 1.75 in^2

φMn = φ*Ast*fy*(dt-((Ast*fy)/(1.7*f'c*b))) = 475 k-in = 40 k-ft
φ = 0.9

At Midspan:

b = 9 in db = 12 in Asb = 1.17 in^2

g = ((Asb*fy)/((b/2*3)*0.85*f'c*0.5) = 2.69 in

φMn = φ*Asb*fy*(db-g/3) = 386 k-in = 32 k-ft
φ = 0.9

Shear Capacity:

Aconc. = 303 in^2

φVn = φVc = (φ*2*(f'c)^1/2*Aconc.) = 25755 lbs = 25.76 kips
φ = 0.85

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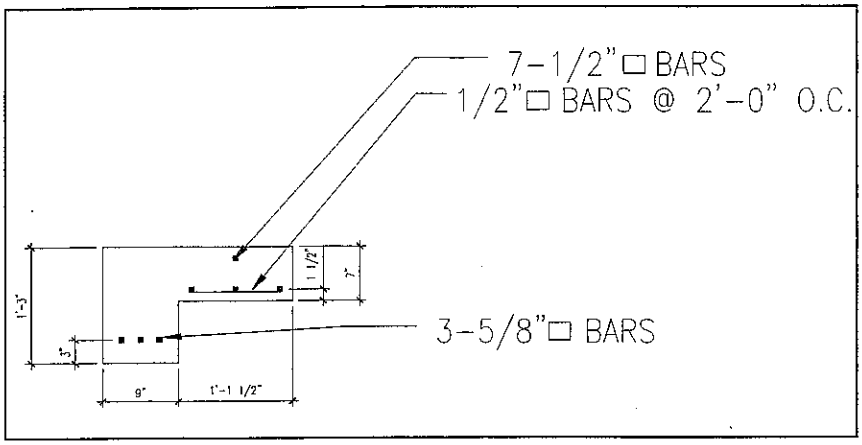
SHEET NO.		OF	
CALCULATED BY	FC	DATE	6/10/2004
CHECKED BY		JOB # :	0404

FILENAME C:\FC-Jobs\0404-Chabot Tower\Excel\Temp\[InteriorPavilionFloorBeam-3.xls]"L" Shape Floor Beam

Pavilion Concrete Beam/"L" Shape Floor Beam



Material Properties (Lower Bound):

f'c = 2500 psi
fy = 33000 psi

(Note: Material properties come from the report by Tennebaum-Manehim Engineers dated May 26, 2004.)

Bending Capacity:

At Supports:

b =	**9**	**in**	**dt =**	**10.5**	**in**	**Ast =**	**1**	**in^2**

φMn = φ*Ast*fy*(dt-((Ast*fy)/(1.7*f'c*b))) = 286 **k-in =** 24 **k-ft**
φ = 0.9

At Midspan:

b =	**9**	**in**	**db =**	**12**	**in**	**Asb =**	**1.17**	**in^2**

g = ((Asb*fy)/((b/2*3)*0.85*f'c*0.5) = 2.69 **in**

φMn = φ*Asb*fy*(db-g/3) = 386 **k-in =** 32 **k-ft**
φ = 0.9

Shear Capacity:

Aconc. = 229.5 **in^2**

φVn = φVc = (φ*2*(f'c)^1/2*Aconc.) = 19507.5 **lbs =** 19.51 **kips**
φ = 0.85

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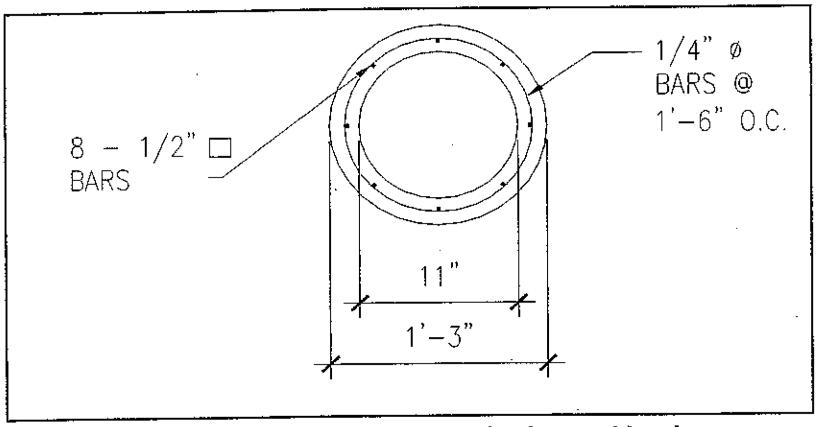
FILENAME C:\FC-Jobs\0404-Chabot Tower\Excel\[PavilionColumn.xls]Pavilion Column

Pavilion Concrete Column/Moment-Curvature Diagram



Material Properties (Lower Bound):

f'c = 2500 psi
fy = 33000 psi
Ec = 2850000 psi
Es = 29000000 psi

(Note: Material properties come from the report by Tennebaum-Manehim Engineers dated May 26, 2004.)

Determine is slenderness effects need to be considered:

L = 10 ft **K =** 2 **do =** 15 in
di = 11 in

A = Π(do^2-di^2)/4 = 81.68 **in^2** **I = Π(do^4-di^4)/64 =** 1766.36 **in^4**

r = (I/A)^1/2 = 4.65 **in**

KL/r = 51.61 **> 34 - 12(M1/M2) = 46, (Column is slender even in the best case when the column is in double curvature with M1/M2 = -1)**

Moment-Curvature Diagrams From PCA Column:

Moment- Axial Load Values w/ Reduction Factors From 2001 CBC. See Sheets 31-37 For PCA Column output.

φbMn(k*ft)	φcPn(k*ft)	Mn(k*ft)	Pn(k*ft)
0	131	0	234
16	131	10	215
20	123	20	195
25	109	30	171
30	91	40	141
33	80	49	100
35	67	51	82
36	57	48	50
35	48	40	20
34	40	30	-6
33	30	20	-28
31	20	10	-48
29	0	0	-66
25	-10		
20	-22		
15	-32		
10	-42		
5	-50		
0	-59		

Moment- Axial Load Diagram values w/ no Reduction. See sheets 38-44 For PCA Column output.

(Note: In order to account for inadequate detailing of the column such as large tie spacing and inadequate development length of the vertical reinf. in the column. The capacities shown will be reduced by 1/3.)

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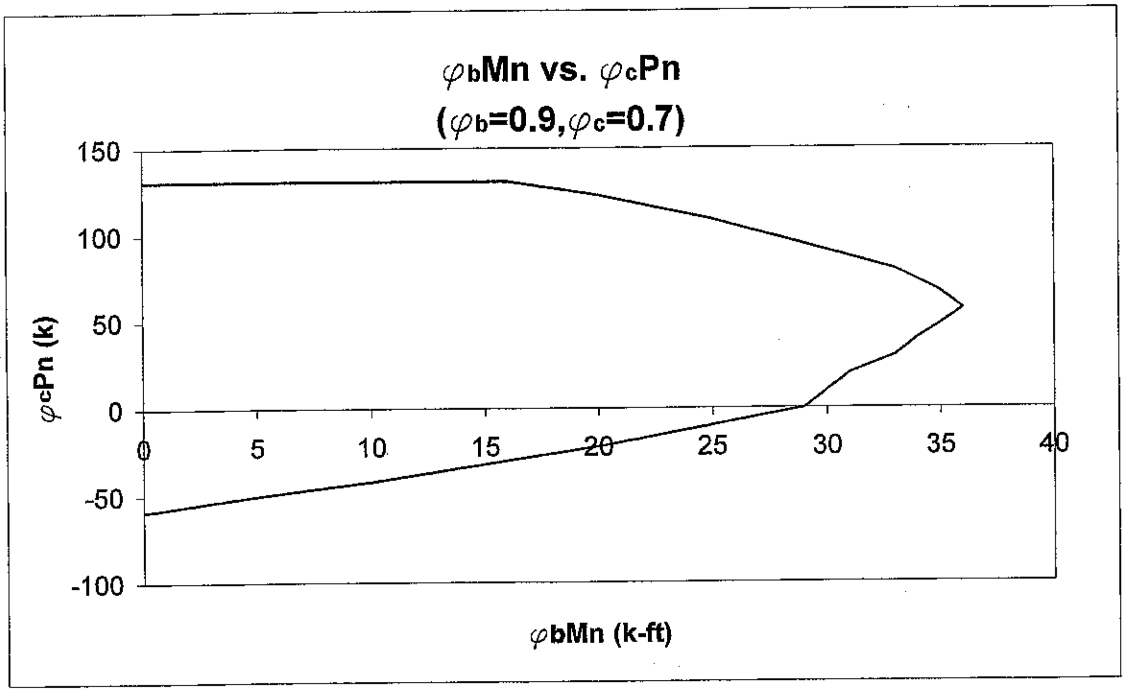
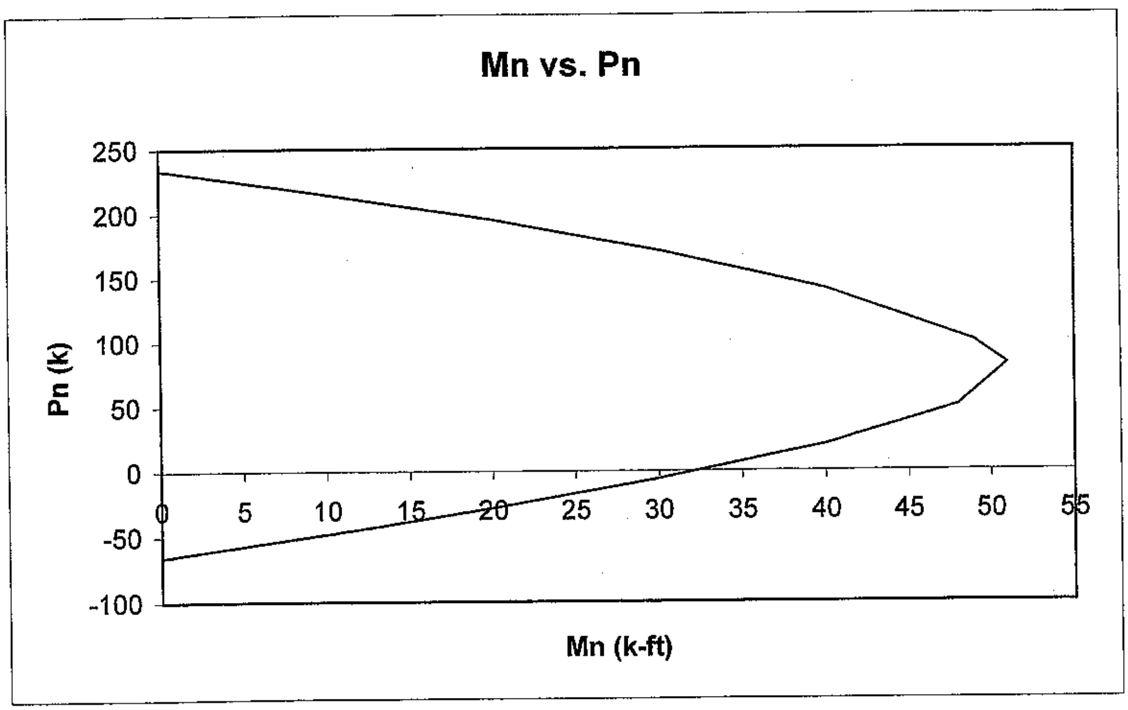
FILENAME C:\FC-Jobs\0404-Chabot Tower\Excel\[PavilionColumn.xls]Pavilion Column

Pavilion Concrete Column/Moment-Curvature Diagram

See Pages 31-37 For PCA Column output.



See Pages 38-44 For PCA Column output.



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FILENAME C:\FC-Jobs\0404-Chabot Tower\Excel\[PavilionColumn.xls]Pavilion Column

Pavilion Concrete Column/Moment-Curvature Diagram

Case #1 : Maximum Moment

Pu =	14.54	kips	Mu =	54.8	k-ft
φPn =	7	kips	φMn =	30	k-ft

Demand-To-Capacity Ratio(DCR) = Pu/0.667*Pn+Mu/0.667*φMn = 5.85

Case #2 : Maximum Axial Force(Compression)

Pu =	46.24	kips	Mu =	49.67	k-ft
φPn =	24	kips	φMn =	32	k-ft

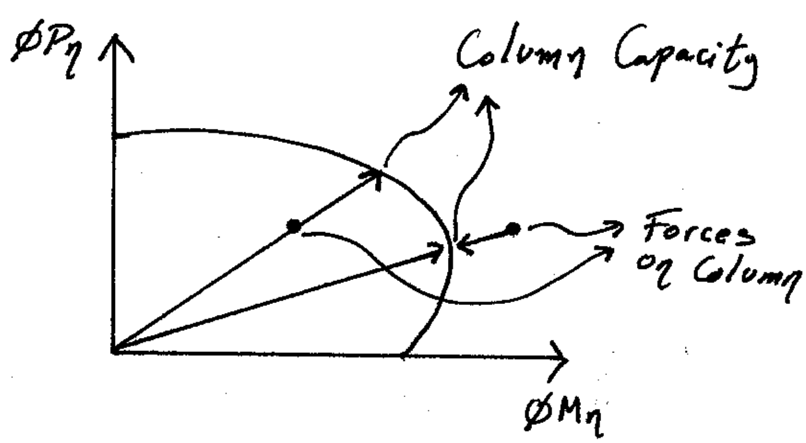
Demand-To-Capacity Ratio(DCR) = Pu/0.667*Pn+Mu/0.667*φMn = 5.22

Case #3 : Maximum Axial Force(Tension)

Pu =	-21.34	kips	Mu =	40.6	k-ft
φPn =	-12	kips	φMn =	24	k-ft

Demand-To-Capacity Ratio(DCR) = Pu/0.667*Pn+Mu/0.667*φMn = 5.20

Note: The axial and beding capacity of the column was obtained by first drawing a line connecting the origin of the Moment-Curvature diagram to the point representing the axial force and moment the column has to withstand. Next you extend the line until it connects with the perimeter of the curve. The point where the curve and the line connect represents the capacity of the column. See sketch below.



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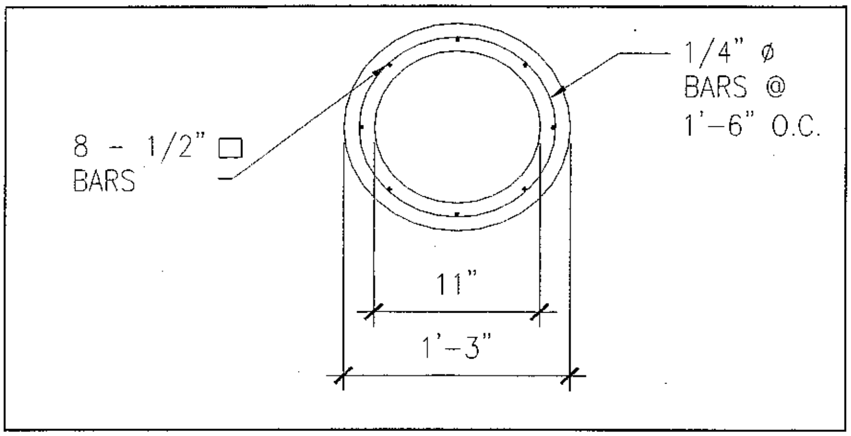
FILENAME *C:\FC-Jobs\0404-Chabot Tower\Excel\[PavilionColumn.xls]Pavilion Column*

Pavilion Concrete Column/Shear Capacity



Material Properties (Lower Bound):

f'c =	**2500**	**psi**
fy =	**33000**	**psi**

(Note: Material properties come from the report by Tennebaum-Manehim Engineers dated May 26, 2004.)

b =	**15**	**in**	**d =**	**13**	**in**	**Av =**	**0.0982**	**in^2**
do =	**15**	**in**	**di =**	**11**	**in**	**s =**	**18**	**in**

Ac = Π(do^2-di^2)/4 = 81.68 **in^2**

φVn = φVc + φVs =(0.85*2*(f'c)^1/2*Ac+0.85*Av*fy*d/s)

φVn =	8932	**lbs =**	8.93	**kips**	**Vu =**	**10.99**	**kips**

φ = 0.85 Demand-To-Capacity Ratio(DCR) = Vu/φVn = 1.23

B. INTAKE TOWER

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Weight of Tower For Seismic Calculations: (Case I)

Weight of Pavilion Roof $= 0.15\,K/ft^3\left((23ft + 1/12ft) - 2(8.25/12)ft - 2(2.75ft)\right)^2(7/12\,ft)$
$+ 0.15\,K/ft^3\,(2ft)(2.75ft)\left((23ft + \frac{1}{12}ft) - 2(8.25/12)ft - 2.75ft\right)(4)$
$+ 0.15\,K/ft^3\,(1.25ft)(1.5ft)\left(((23ft + 1/12ft) - 1.5ft)\right)(4) = 109.83K$

Weight of Pavilion Columns $= 18(0.15\,K/ft^3)(\pi)\left((1.75ft)^2 - (11/12ft)^2\right) \times (1/4) \times (9ft)$
$= 42.4K$

Weight of Pavilion Floor $= 0.15\,K/ft^3\,(3/12ft)(2(2.83ft) + 2.33ft)(14.71ft)$
$+ 0.15\,K/ft^3\,(9/12ft)(15/12ft)(2) \times (14.71ft)$
$+ 0.15\,K/ft^3\,(0.375ft(7/12ft) + (8/12ft)(9/12ft)(2) + (1.875ft)(7/12ft))$
$+ 0.15\,K/ft^3\,(3ft)(2ft)(16.71ft + 2.75ft(2))$
$+ 0.15\,K/ft^3\,(4.5ft)(3ft)(18.33ft)(2$
$= 103.3K$

Weight of Brick $= 0.12\,K/ft^3\,(1)(1.5ft)(17ft)(12.67ft)$
$= 38.77K$

Weight of stone $= 0.16\,K/ft^3\,(1)(45ft - 4.5ft - 1.5ft) \times (17ft)(12.67ft)$
$= 1{,}344K$

Total weight $= 109.83K + 42.4K + 103.3K + 38.77K + 1{,}344K$
$= 1{,}638.33K \leftarrow$ "Total Weight Including Both walls Forming The Tower"

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Weight of Tower For Seismic Calculations: (Case II)

Weight of Pavilion Roof $= 0.15\,K/ft^3\left(\left((23ft + 1/12ft) - 2(8.25/12)ft + 2(2.75ft)\right)^2 (7/12ft)\right.$
$+ 0.15K/ft^3 (2ft)(2.75ft)\left((23ft+\frac{1}{12}ft) - 2(8.25/12)ft - 2.75ft\right)(4)$
$+ 0.15K/ft^3 (1.25ft)(1.5ft)\left(\left((23ft+1/12ft) - 1.5ft\right)\right)(4) = 109.83K$

Weight of Pavilion Columns $= 18(0.15K/ft^3)(\pi)\left((1.75ft)^2 - (11/12ft)^2\right) \times (1/4) \times (9ft)$
$= 42.4K$

Weight of Pavilion Floor $= 0.15K/ft^3 (3/12ft)(2(2.83ft) + 2.33ft)(14.71ft)$
$+ 0.15K/ft^3 (9/12ft)(15/12ft)(2) \times (14.71ft)$
$+ 0.15K/ft^3 \left(2.375ft(7/12ft) + (8/12ft)(9/12ft)(2) + (1.875ft)(7/12ft)\right)$
$+ 0.15K/ft^3 (3ft)(2ft)(16.71ft + 2.75ft(2))$
$+ 0.15K/ft^3 (4.5ft)(3ft)(18.33ft)($
$= 103.3K$

Weight of Brick $= 0.12K/ft^3 (1)(1.5ft)(17ft)(12.67ft)$
$= 38.77K$

Weight of stone $= 0.16K/ft^3(1)(27.1ft) \times (17ft)(12.67ft)$
$= 933.93K$

Total weight $= 109.83K + 42.4K + 103.3K + 38.77K + 933.93K$

$= 1,228.23K$ ← "Total weight Including Both walls Forming The Tower"

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PROJECT	Chabot Tower
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FILENAME *C:\FC-Jobs\0404-Chabot Tower\Excel\[Ubc97Vb.xls]UBC97-Case I*

UBC 97 STATIC SEISMIC LOAD CALCULATION

STRENGTH DESIGN Date= 7/8/2004

MAIN TOWER - CASE I

IMPORTANT FACTOR	1.5				
BUILDING TYPE					
BUILDING HEIGHT(hn)(Ft)	45	FT			
PLAN IRREGULARITIES	YES				
VERTICAL IRREGULARITIES	YES		Given Na? No		
FAULT TYPE	A		Given Nv? No		
NEAR SOUCE DISTANCE(km)	0.5	Km	*If "Yes", Input Data*		
Z	0.4		Na(Given) 1.5		
Ct	0.02		Nv(Given) 2		
S	SB				
R	2		Ωo		
PERIOD T_B (fomula 30-10)	0.35	s	W =	1638.33	kips
Meet 1629.4.2. Requirement?	No	("Yes" or "No")			

Na(Code)	1.5	Na(Used)	1.5		1.5
Nv(Code)	2	Nv(Used)	2		
Ca			0.6		
Cv			0.8		
$T_A = Ct(hn)^{3/4}$			0.35	s	
USED T for strength design			0.35	s	
$V = \boxed{\frac{Cv*I*W}{RT}}$		strength design	1.7143	W	
		drift check	1.7143	W	
$Vmax = \boxed{\frac{2.5*Ca*I*W}{R}}$			1.125	W	
$Vmin = \boxed{\frac{0.8*Z*Nv*I*W}{R}}$			0.48	W	
$Vmin = \boxed{0.11*Ca*I*W}$			0.099	W (Omit for drift checking)	

FOR STRENGTH DESIGN

V= 1.125 W = 1843.1 kips

FOR DRIFT CHECKING

V= 1.125 W = 1843.1 kips

C:\FC-Jobs\0404-Chabot Tower\Excel\[Ubc97Vb.xls]UBC97-Case I

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FILENAME *C:\FC-Jobs\0404-Chabot Tower\Excel\[Ubc97Vb.xls]UBC97Case-II*

UBC 97 STATIC SEISMIC LOAD CALCULATION

STRENGTH DESIGN Date= 7/8/2004

MAIN TOWER - CASE II

IMPORTANT FACTOR	1.5				
BUILDING TYPE					
BUILDING HEIGHT(hn)(Ft)	33.1	FT			
PLAN IRREGULARITIES	YES				
VERTICAL IRREGULARITIES	YES		Given Na?	No	
FAULT TYPE	A		Given Nv?	No	
NEAR SOUCE DISTANCE(km)	0.5	Km	*If "Yes", Input Data*		
Z	0.4		Na(Given)	1.5	
Ct	0.02		Nv(Given)	2	
S	SB				
R	2		Ωo		
PERIOD T_B (fomula 30-10)	0.28	s	W =	1228.23	kips
Meet 1629.4.2. Requirement?	No	("Yes" or "No")			

Na(Code)	1.5	Na(Used)	1.5		1.5
Nv(Code)	2	Nv(Used)	2		
Ca			0.6		
Cv			0.8		
$T_A = Ct(hn)^{3/4}$			0.28	s	
USED T for strength design			0.28	s	

$V = \boxed{\dfrac{Cv*I*W}{RT}}$ strength design 2.1429 W

drift check 2.1429 W

$Vmax = \boxed{\dfrac{2.5*Ca*I*W}{R}}$ 1.125 W

$Vmin = \boxed{\dfrac{0.8*Z*Nv*I*W}{R}}$ 0.48 W

$Vmin = \boxed{0.11*Ca*I*W}$ 0.099 W (Omit for drift checking)

FOR STRENGTH DESIGN

V= 1.125 **W =** **1381.8** kips

FOR DRIFT CHECKING

V= 1.125 **W =** **1381.8** kips

C:\FC-Jobs\0404-Chabot Tower\Excel\[Ubc97Vb.xls]UBC97Case-II

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TABLE A-4 Allowable Working Stresses in Unreinforced Unit Masonry*

MATERIAL	Type M Compression[1]	Type S Compression[1]	Type M or Type S Mortar: Shear or Tension in Flexure[2] [3]		Type M or Type S Mortar: Tension in Flexure[4]		Type N Compression[1]	Type N: Shear or Tension in Flexure[2] [3]	
Special Inspection Required	No	No	Yes	No	Yes	No	No	Yes	No
Solid Brick Masonry									
4500 plus p.s.i.	250	225	20	10	40	20	200	15	7.5
2500–4500 p.s.i.	175	160	20	10	40	20	140	15	7.5
1500–2500 p.s.i.	125	115	20	10	40	20	100	15	7.5
Solid Concrete Unit Masonry									
Grade A	175	160	12	6	24	12	140	12	6
Grade B	125	115	12	6	24	12	100	12	6
Grouted Masonry									
4500 p.s.i.	350	275	25	12.5	50	25			
2500–4500 p.s.i.	275	215	25	12.5	50	25			
1500–2500 p.s.i.	225	175	25	12.5	50	25			
Hollow Unit Masonry[5]	170	150	12	6	24	12	140	10	5
Cavity Wall Masonry Solid Units[5]									
Grade A or 2500 p.s.i. plus	140	130	12	6	30	15	110	10	5
Grade B or 1500–2500 p.s.i.	100	90	12	6	30	15	80	10	5
Hollow Units[5]	70	60	12	6	30	15	50	10	5
Stone Masonry									
Cast Stone	400	360	8	4	—	—	320	8	4
Natural Stone	140	120	8	4	—	—	100	8	4
Gypsum Masonry	20	20	—	—	—	—	20		
Unburned Clay Masonry	30	30	8	4	—	—			

1 Allowable axial or flexural compressive stresses in pounds per square inch gross cross-sectional area (except as noted). The allowable working stresses in bearing directly on concentrated loads may be 50 per cent greater than these values.
2 This value of tension is based on tension across a bed joint, i.e., vertically in the normal masonry work.
3 No tension allowed in stack bond across head joints.
4 The values shown here are for tension in masonry in the direction of running bond, i.e., horizontally between supports.
5 Net area in contact with mortar or net cross-sectional area.
* UBC Table 24-B.

*These Masonry Values were Averaged with Those In The TME Report. The Average Values were Used In The Calculation For The Report See Sheet #60 And #61.

TABLE A-5 Allowable Shear on Bolts for all Masonry Except Gypsum and Unburned Clay Units*

DIAMETER OF BOLT (Inches)	EMBEDMENT[2] (Inches)	SOLID MASONRY (Shear in Pounds)	GROUTED MASONRY (Shear in Pounds)
1/2	4	350	550
5/8	4	500	750
3/4	5	750	1100
7/8	6	1000	1500
1	7	1250	1850[1]
1-1/8	8	1500	2250[1]

1 Permitted only with not less than 2500 pounds per square inch units.
2 It is recommended that these embedment lengths be increased 30% if they anchor a beam or girder on top of a column or pilaster.
* UBC Table 24-G.

Source: "Reinforced Masonry Engineering Handbook. Brick And Other structural clay units" By J.E. Amrhein

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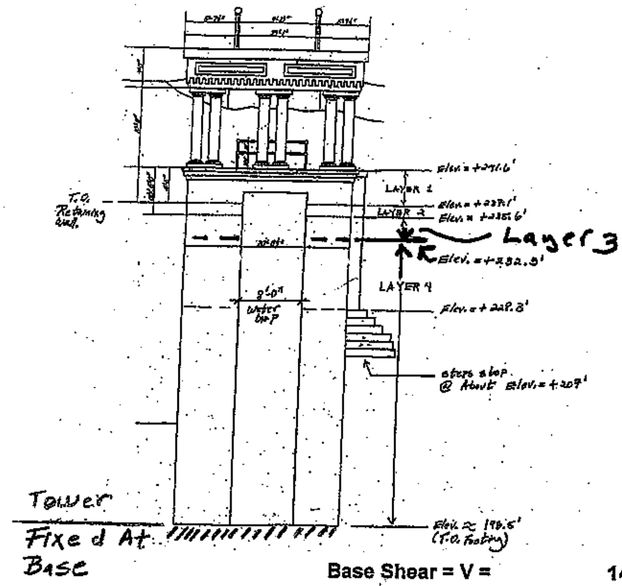
FILENAME C:\FC-Jobs\0404-Chabot Tower\Excel\[MainTowerForces.xls]Main Tower-Case I

Main Tower Forces & Stresses - Case I



Base Shear = V = 1843.1 kips
(Main Tower)

Base Shear = V = 147.4 kips
(Pavilion)

Layer #	Material Type	Layer Height(ft)	Layer Length(ft)	Layer Width(ft) (Per Wall)	Layer Weight(k) (Total)	Seismic Shear Force(k) Per Layer [1]	Cumulative Seismic Force Per Layer At Each Wall/1.4 (k)
1	Concrete	4.5	17	6.33	255.53	520.62	238.58
2	Brick	1.5	17	6.33	294.3	73.45	264.81
3	Dressed Stone	3.3	17	6.33	408.03	202.44	337.11
4	Stone Masonry	35.7	17	6.33	1638.3	1046.59	710.89

Layer #	Allowable Compressive Stresses (psi) [2]	Allowable Tensile Stresses(psi) [3,4]	Allowable Shear Stresses(psi) [3,4]	Compressive Stress At Each Layer (psi) *	Tensile Stress At Each Layer (psi) **	Shear Stress At Each Layer (psi)
1	2500	250	100	48	-32	15
2	900	17.5	21	73	-54	17
3	1800	14	31	137	-111	22
4	1800	14	31	1321	-1215	46

Layer #	DCR [5] For Compressive Stresses	DCR [5] For Tensile Stresses	DCR [5] For Shear Stresses
1	0.02	0.13	0.15
2	0.08	3.07	0.81
3	0.08	7.91	0.70
4	0.73	86.81	1.48

Notes:

1. The shear force for each layer was determined based on EQ. 30-15 in the 2001 California Building Code(CBC) for vertical distribution of forces.
2. The compressive strength values were obtained from the report by Tennebaum-Manehim Engineers dated May 26, 2004.
3. The tensile and shear strengths of the concrete were obtained from the 2001 CBC. Tensile Strength = $0.1f'_c$ and the Shear Strength = $2*f'_c{}^{1/2}$.
4. The tensile and shear strengths for the brick and stone masonry were values averaged from those given in the report by Tennebaum-Manehim Engineers and a book titled "Reinforced Masonry Engineering Handbook" by J.E. Amrhein.
5. DCR = Demand To Capacity Ratio.

* Compressive stress = $P/A + Mc/I$

** Tensile stresses = $P/A - Mc/I$

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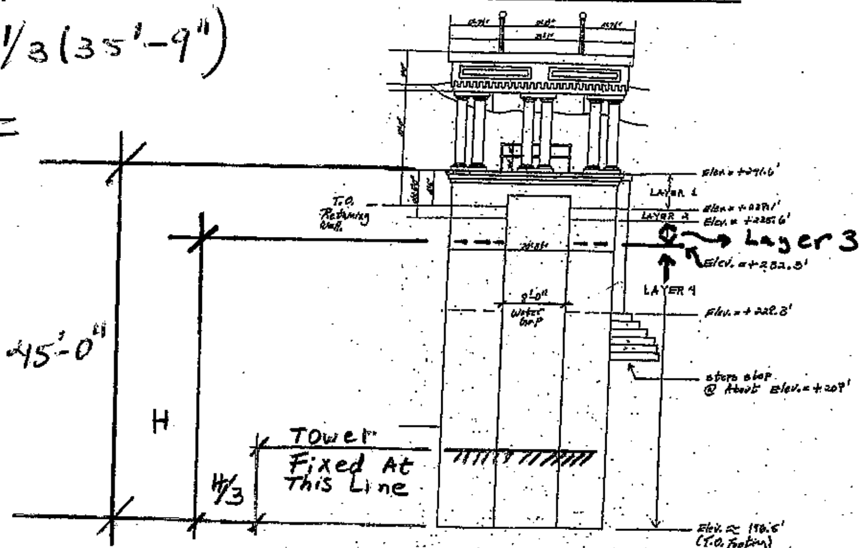
FILENAME C:\FC-Jobs\0404-Chabot Tower\Excel\[MainTowerForces.xls]Main Tower-Case II

H = 35'-9"
45'-0" - 1/3 (35'-9")
= 33'-1"

Main Tower Forces & Stresses - Case II



H = Height of Stone Masonry Layer.

* In This Calculation It Is Assumed That The Lowest 1/3 Section of The Stone Masonry Layer Is Embedded In concrete. Therefore The Unbraced Length Of The Intake Tower Decreases.

Base Shear = V = 1381.8 kips
(Main Tower)

Base Shear = V = 147.4 kips
(Pavilion)

Layer #	Material Type	Layer Height(ft)	Layer Length(ft)	Layer Width(ft) (Per Wall)	Layer Weight(k) (Total)	Seismic Shear Force(k) Per Layer [1]	Cumulative Seismic Force Per Layer At Each Wall/1.4 (k)
1	Concrete	4.5	17	6.33	255.53	503.89	232.60
2	Brick	1.5	17	6.33	294.3	69.02	257.25
3	Dressed Stone	3.3	17	6.33	408.03	185.01	323.33
4	Stone Masonry	23.8	17	6.33	1228.23	623.88	546.14

Layer #	Allowable Compressive Stresses (psi) [2]	Allowable Tensile Stresses(psi) [3,4]	Allowable Shear Stresses(psi) [3,4]	Compressive Stress At Each Layer (psi) *	Tensile Stress At Each Layer (psi) **	Shear Stress At Each Layer (psi)
1	2500	250	100	48	-31	15
2	900	17.5	21	71	-52	17
3	1800	14	31	133	-107	21
4	1800	14	31	793	-714	35

Layer #	DCR[5] For Compressive Stresses	DCR[5] For Tensile Stresses	DCR[5] For Shear Stresses
1	0.02	0.12	0.15
2	0.08	2.98	0.79
3	0.07	7.65	0.67
4	0.44	50.97	1.14

Notes:

1. The shear force for each layer was determined based on EQ. 30-15 in the 2001 Californis Building Code(CBC) for vertical distribution of forces.
2. The compressive strength values were obtained from the report by Tennebaum-Manehim Engineers dated May 26, 2004.
3. The tensile and shear strengths of the concrete were obtained from the 2001 CBC. Tensile Strength = $0.1f'_c$ and the Shear Strength = $2*f'_c{}^{1/2}$.
4. The tensile and shear strengths for the brick and stone masonry were values averaged from those given in the report by Tennebaum-Manehim Engineers and a book titled "Reinforced Masonry Engineering Handbook" by J.E. Amrhein.
5. DCR = Demand To Capacity Ratio.

* Compressive stresses = $P/A + Mc/I$
** Tensile Stresses = $P/A - Mc/I$