

**Appendix 3.2: Seismic Evaluation of Chabot Tower, Quest Structural, Inc.,
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SEISMIC EVALUATION OF CHABOT TOWER



FINAL REPORT

Prepared for

URS Corporation
1333 Broadway, 8th Floor
Oakland, CA 94612

QUEST Structures, Inc.

Report No. QS04-02

3 Altarinda Road, Suite 203, Orinda, CA 94563

February 3, 2005

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Prepared for

URS Corporation
1333 Broadway Ave., Suite 800
Oakland, CA 94612Oakland, CA 94607

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Prepared by

**Quest Structures, Inc.
3 Altarinda Road, Suite 203
Orinda, CA 94563**

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ATTACHMENT I: TME Report

ATTACHMENT II: OLMM Report

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 7 1/4 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 possibility 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.

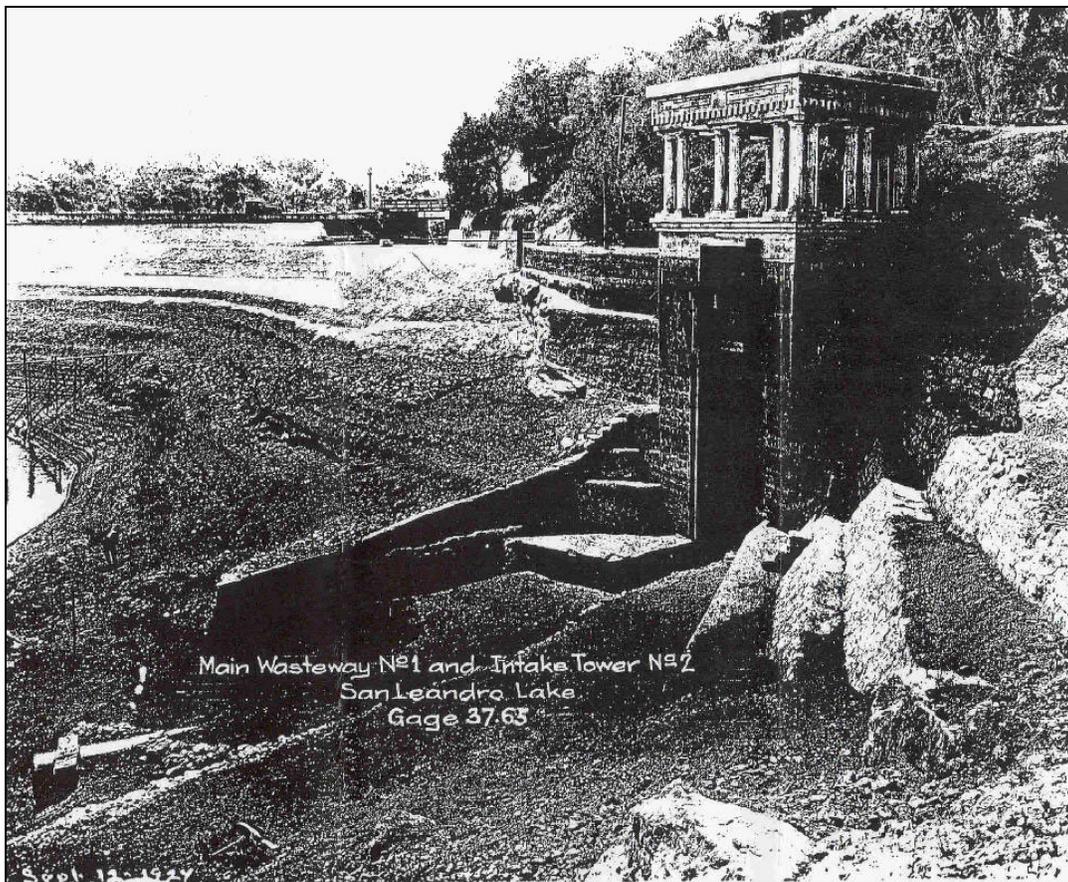


Figure 2-1: Construction photo taken on September 12, 1924

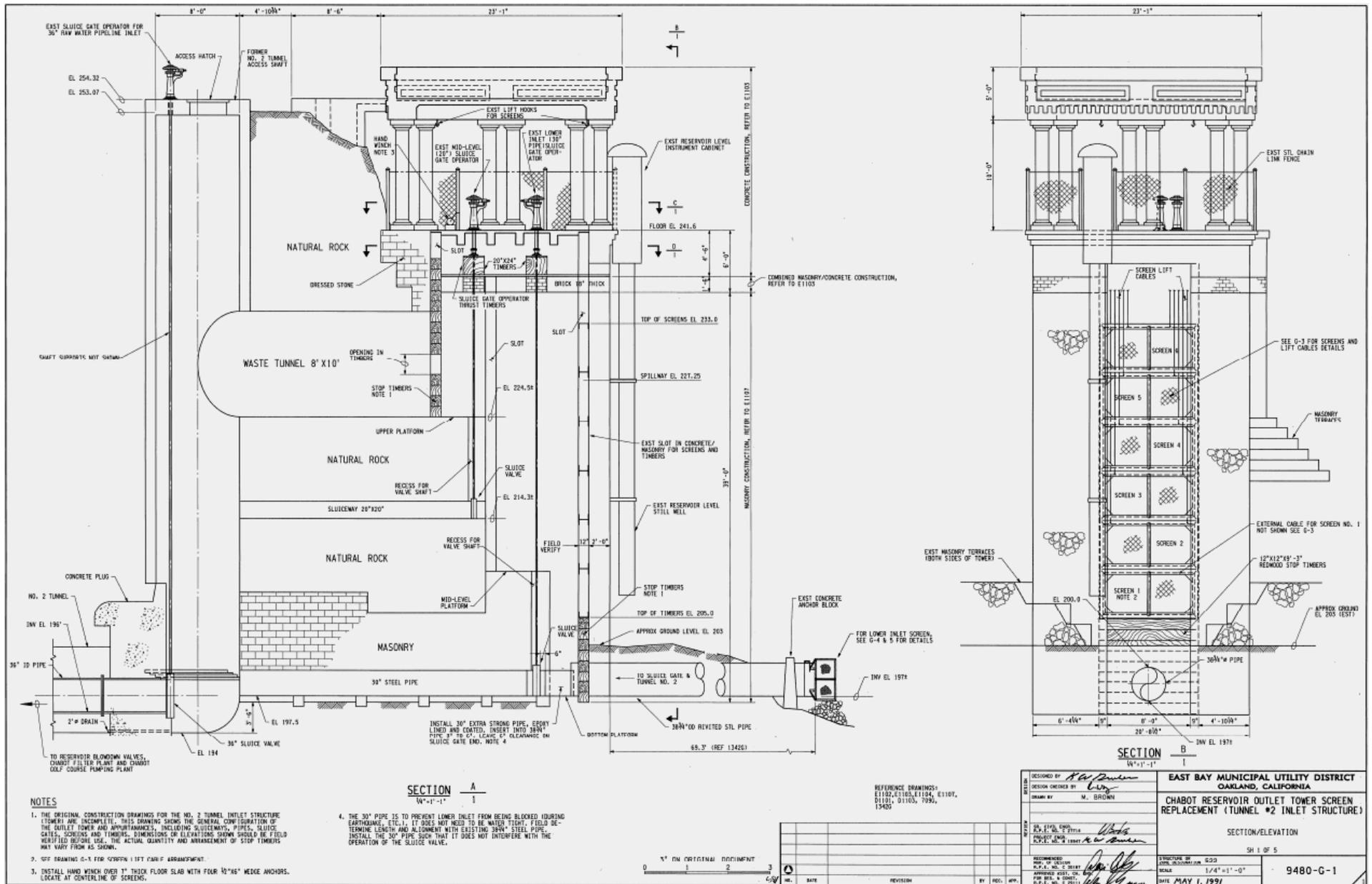


Figure 2-2: Section/elevation view showing multi-level entry portal structure and 8-foot brick-lined outlet shaft

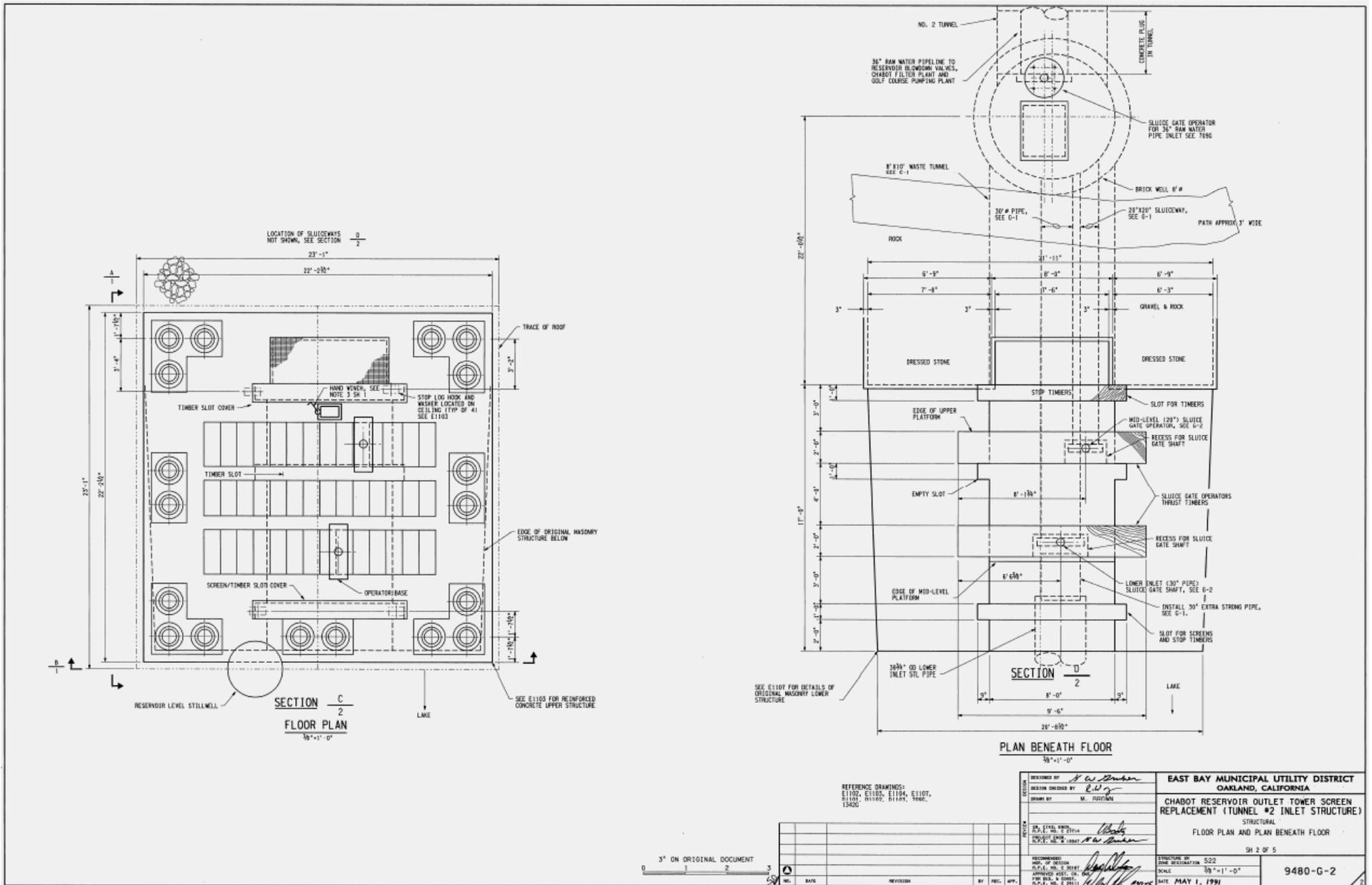


Figure 2-3: Floor plan and plan beneath the floor of tower structure

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 OLM 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/EVALUATION 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 (< 0.2 sec), 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 $M_w 7.1/4$.

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	

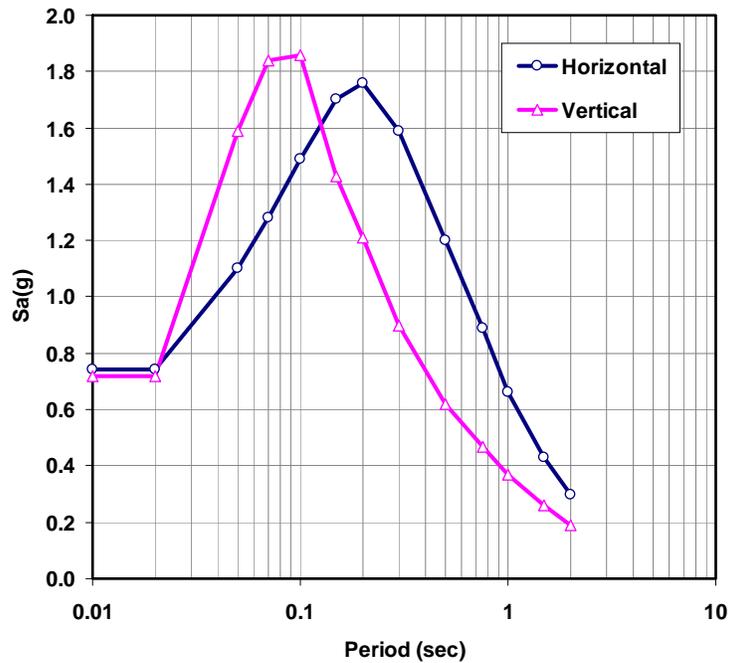


Figure 3-1: MDE Horizontal and Vertical Spectral Accelerations at 5% damping

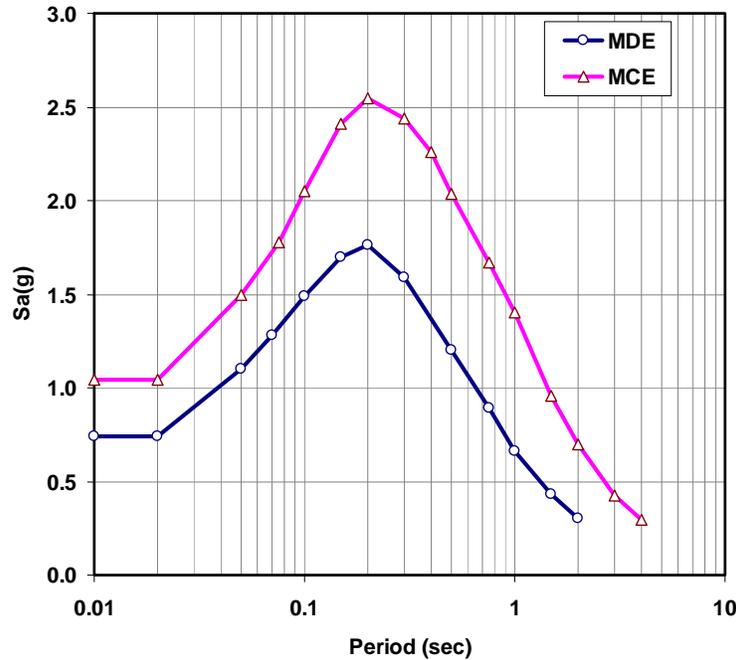


Figure 3-2: Comparison of MCE with MDE Horizontal Spectral Accelerations at 5% damping

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:

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

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 $\phi_p = 0.7$, and the bending moment reduction factor of $\phi_M = 0.9$ 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.

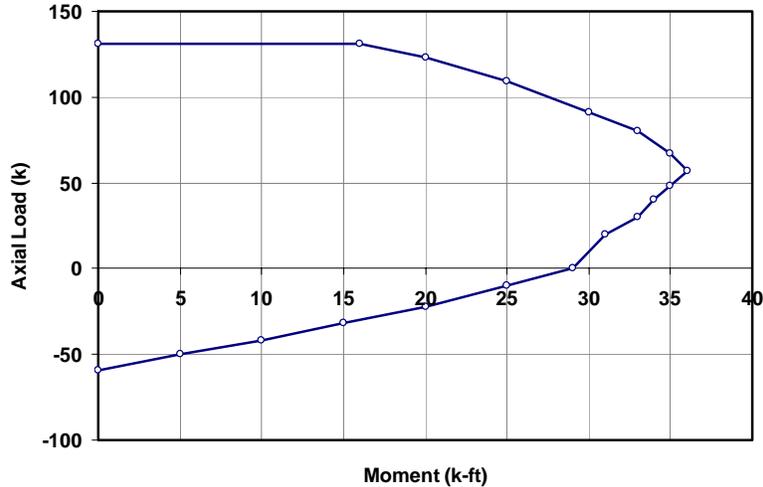


Figure 4-1: Axial load-bending moment interaction diagram for pavilion columns

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 OLM 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.

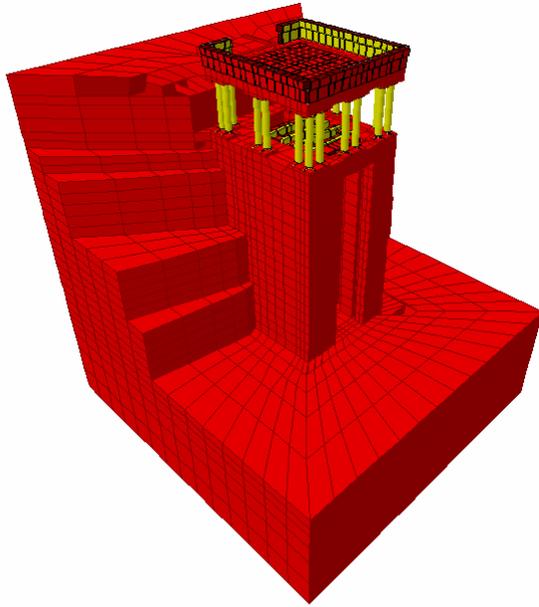


Figure 5-1: Front view of the tower, foundation, abutment, and pavilion model

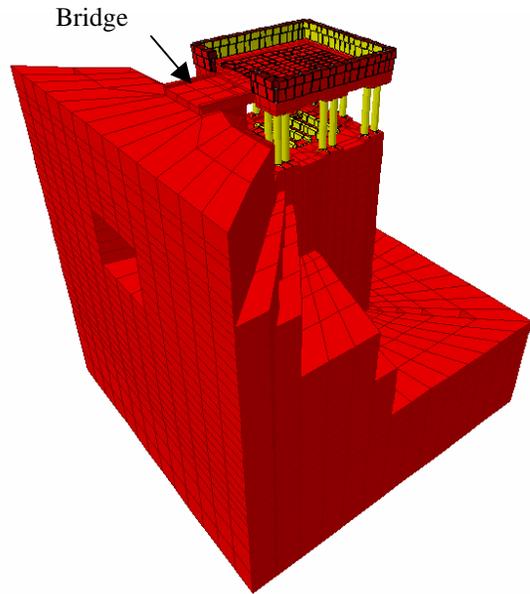


Figure 5-2: Back view of the tower, foundation, abutment, and pavilion model

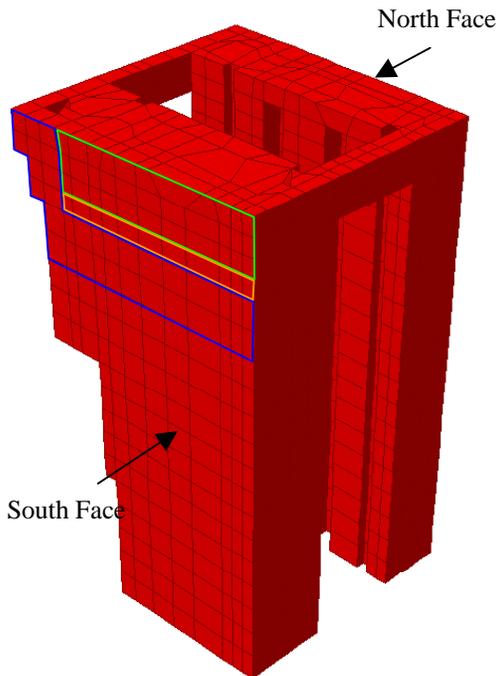


Figure 5-3: Front view of the tower model

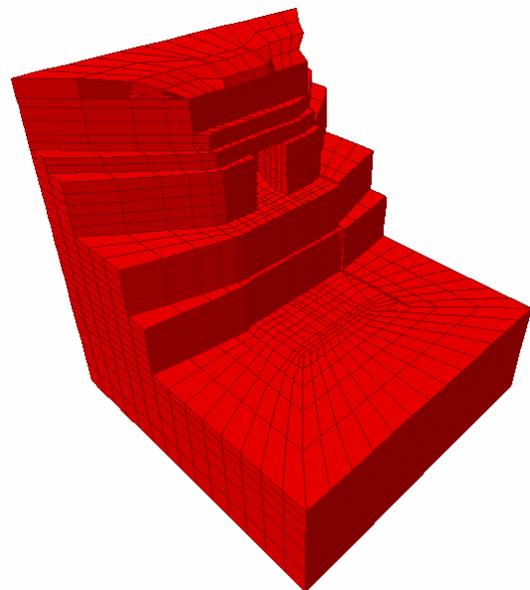


Figure 5-4: Front view of the foundation model

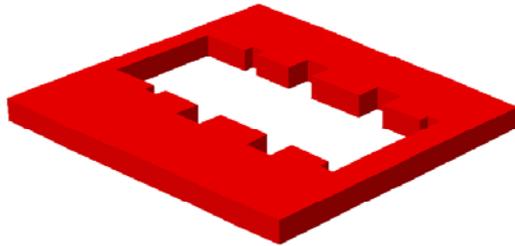


Figure 5-5: Top of Tower at El. 239 ft.

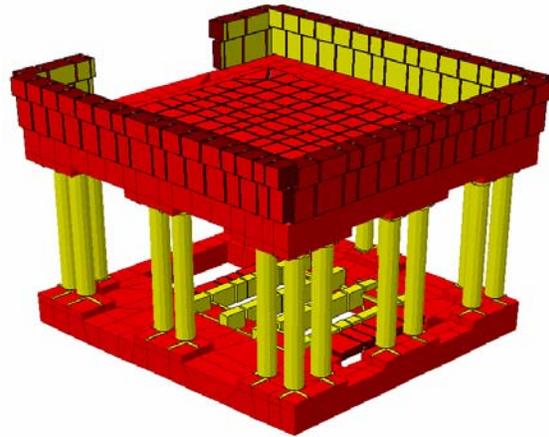


Figure 5-6: Pavilion model

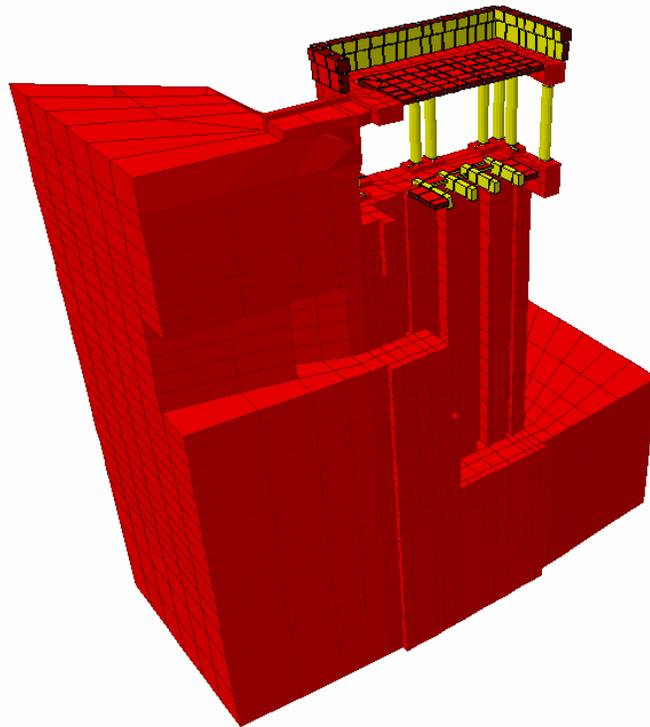


Figure 5-7: Vertical mid-section view showing stop timber slots and 8' x 10' waste tunnel. Also shown are pavilion roof beams and slab and floor beams and slab.

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.

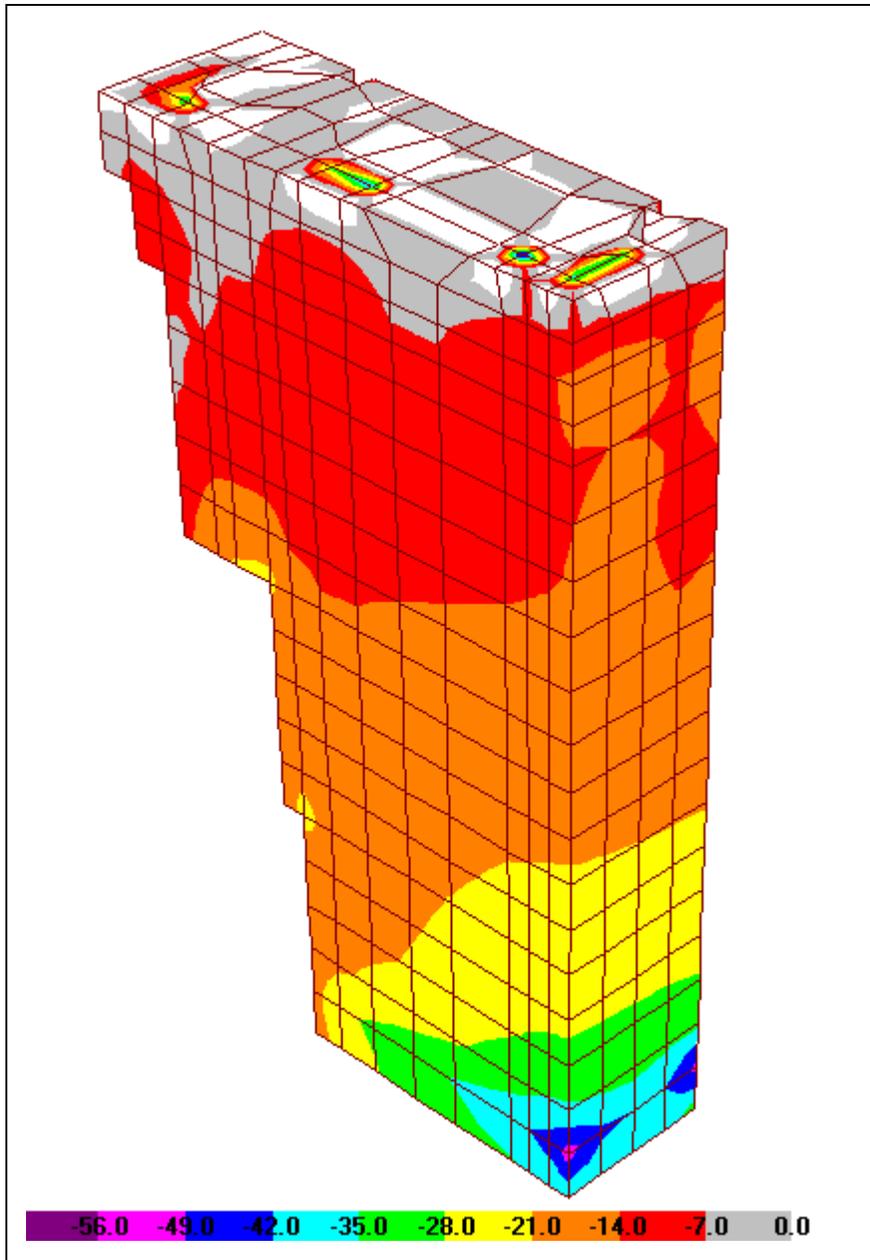


Figure 5-8: South-wall vertical stresses due to self weight (psi)

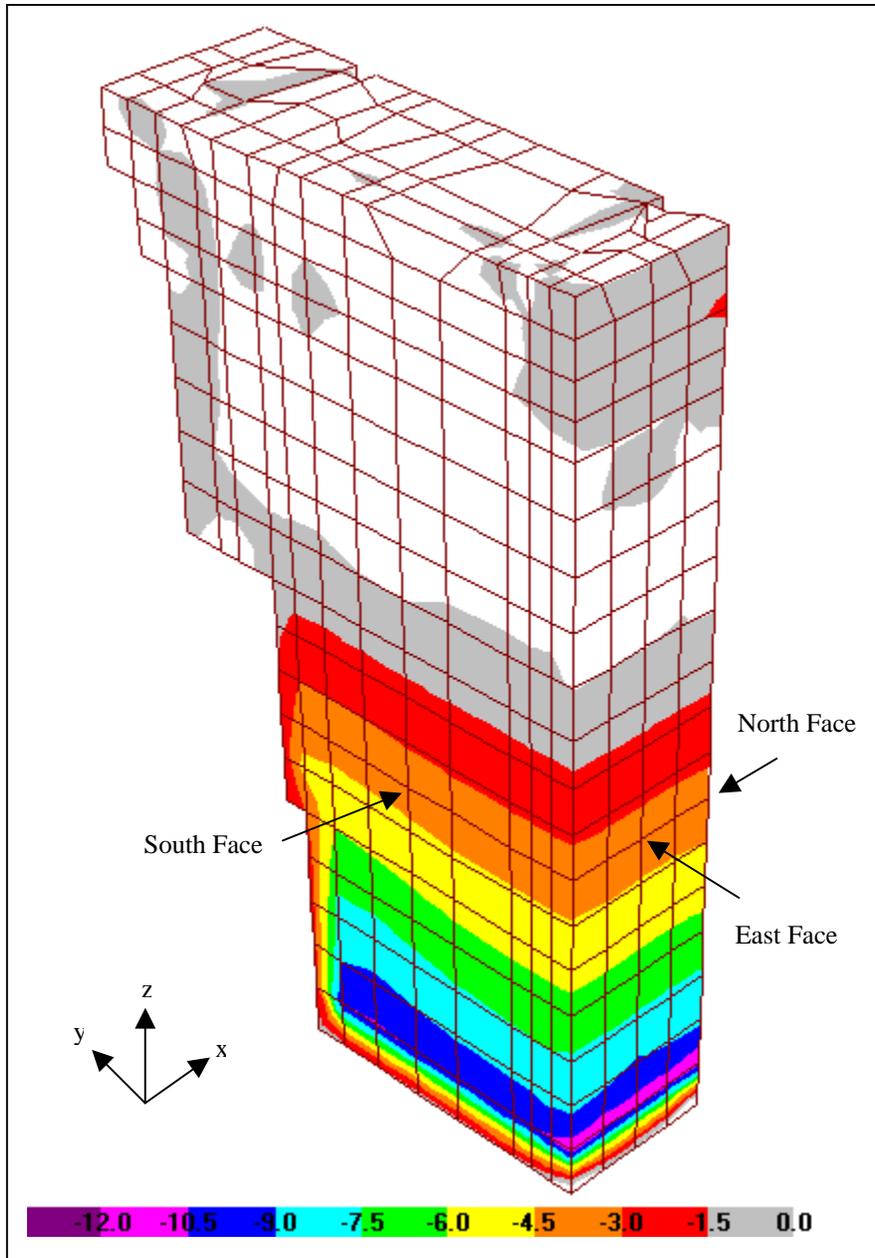


Figure 5-9a: South-wall horizontal stresses (σ_{xx}) due to hydrostatic pressures (psi)

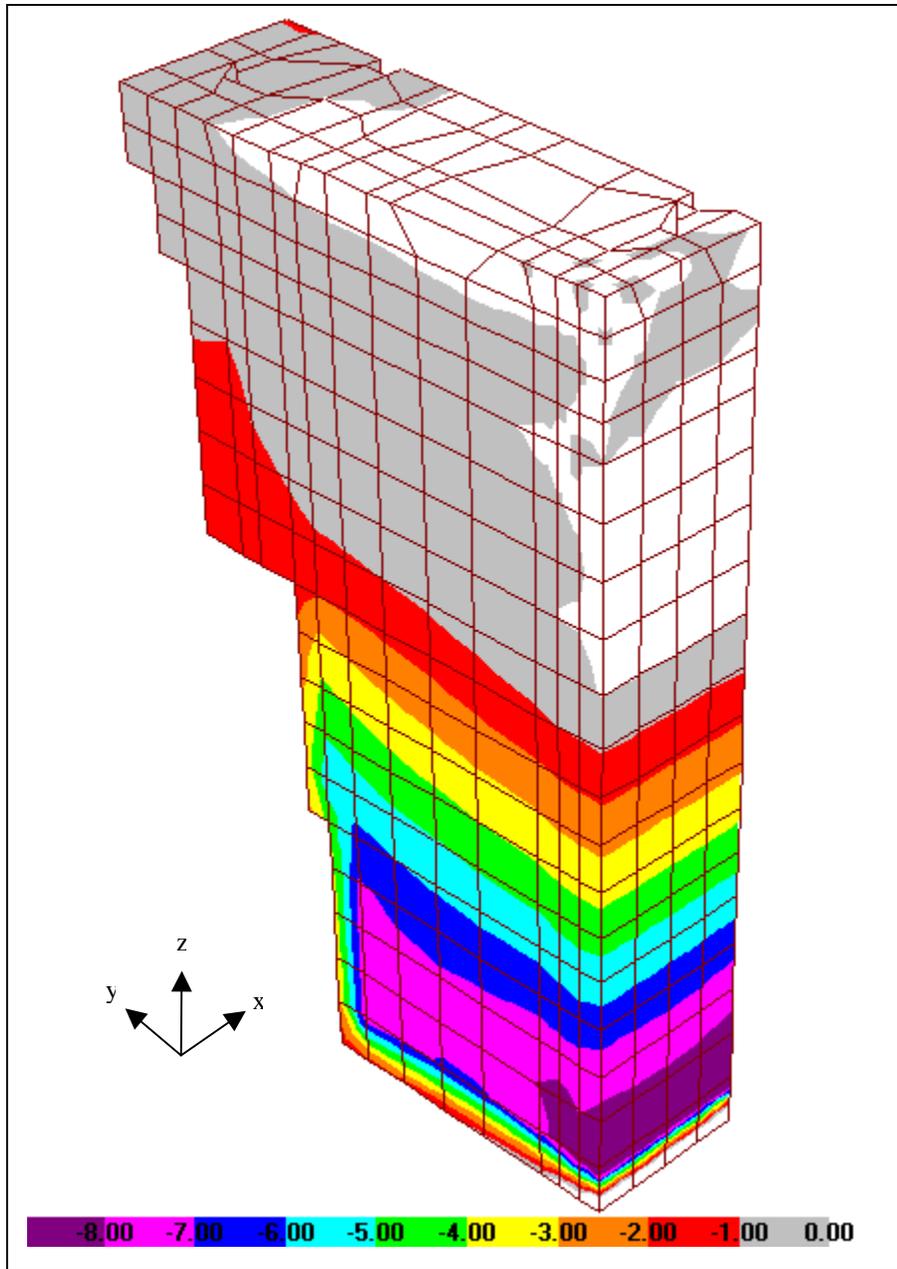


Figure 5-9b: South-wall horizontal stresses (σ_{yy}) due to hydrostatic pressures (psi)

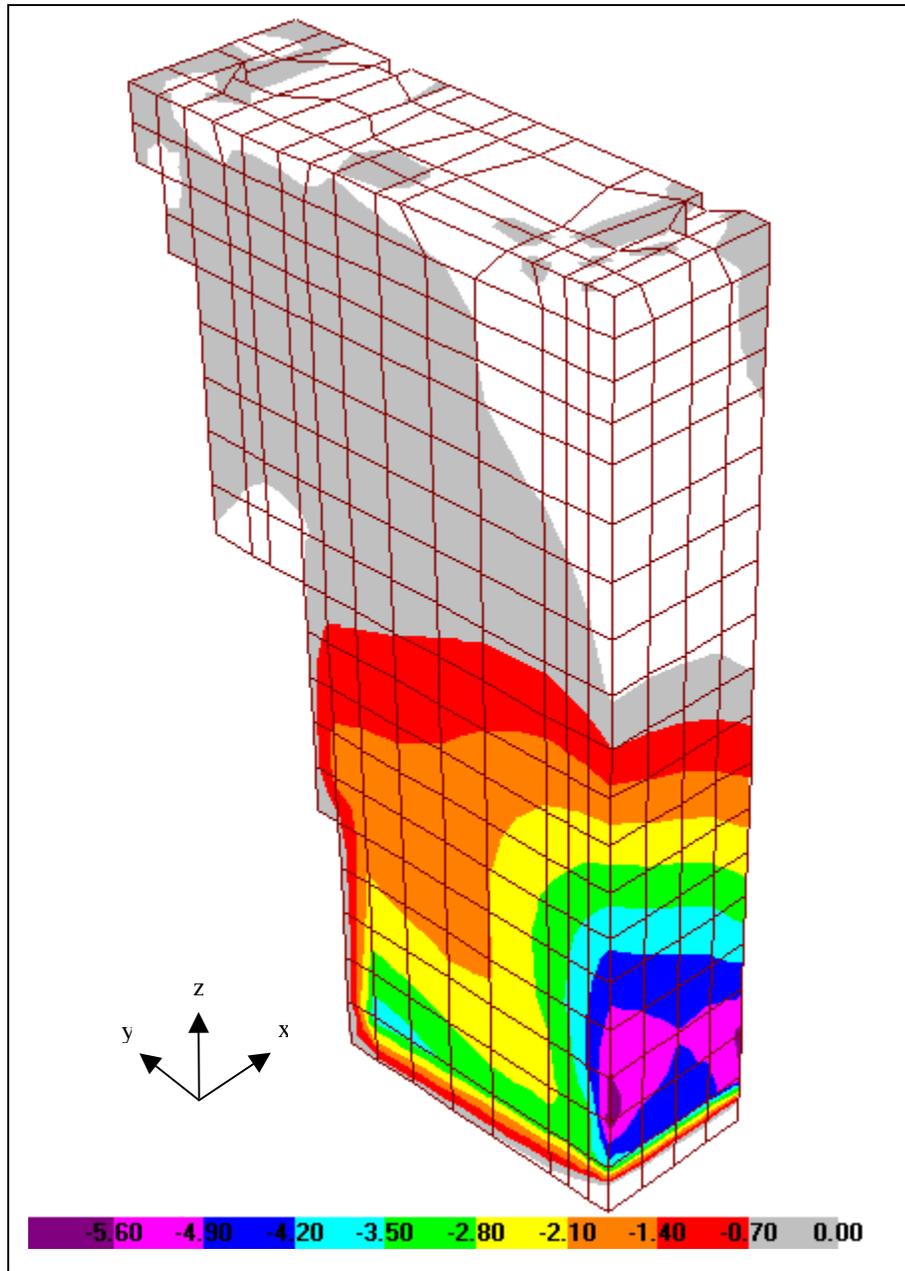


Figure 5-9c: South-wall vertical stresses (σ_{zz}) due to hydrostatic pressures (psi)

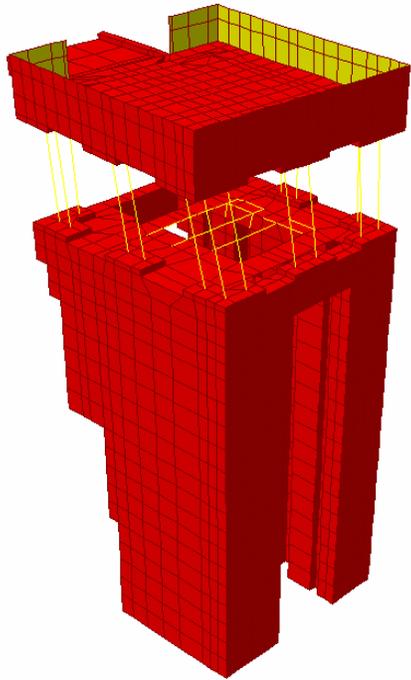
5.3 MODE SHAPES AND PERIODS

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

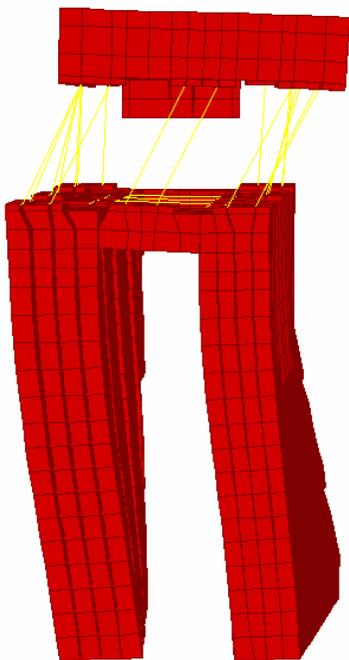
Mode	Period (sec)	Individual Mode (%)			Cumulative (%)		
		N-S	E-W	Vertical	N-S	E-W	Vertical
1	0.085	7.81	0	0	7.81	0	0
2	0.059	39.22	0	0.015	47.03	0	0.015
3	0.052	0	0.04	1.73	47.04	0.04	1.74
5	0.035	7.03	0.02	0.01	54.36	0.72	1.76
6	0.031	0	8.34	3.97	54.36	9.06	5.73
7	0.027	0.01	3.25	44.64	54.38	12.31	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	0.14	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



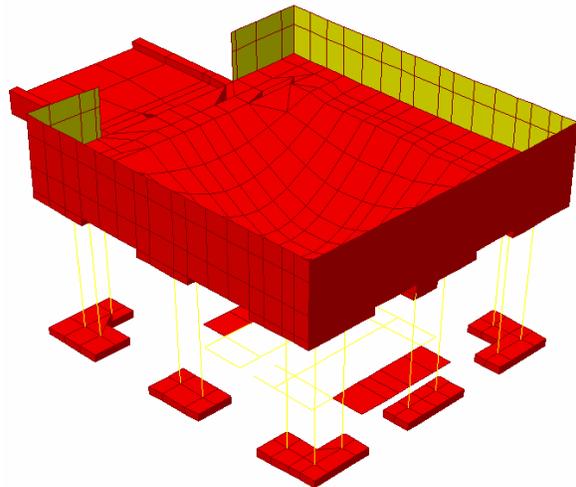
a) Mode-1 viewed from south east
 $T_1 = 0.0850$ sec



(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.

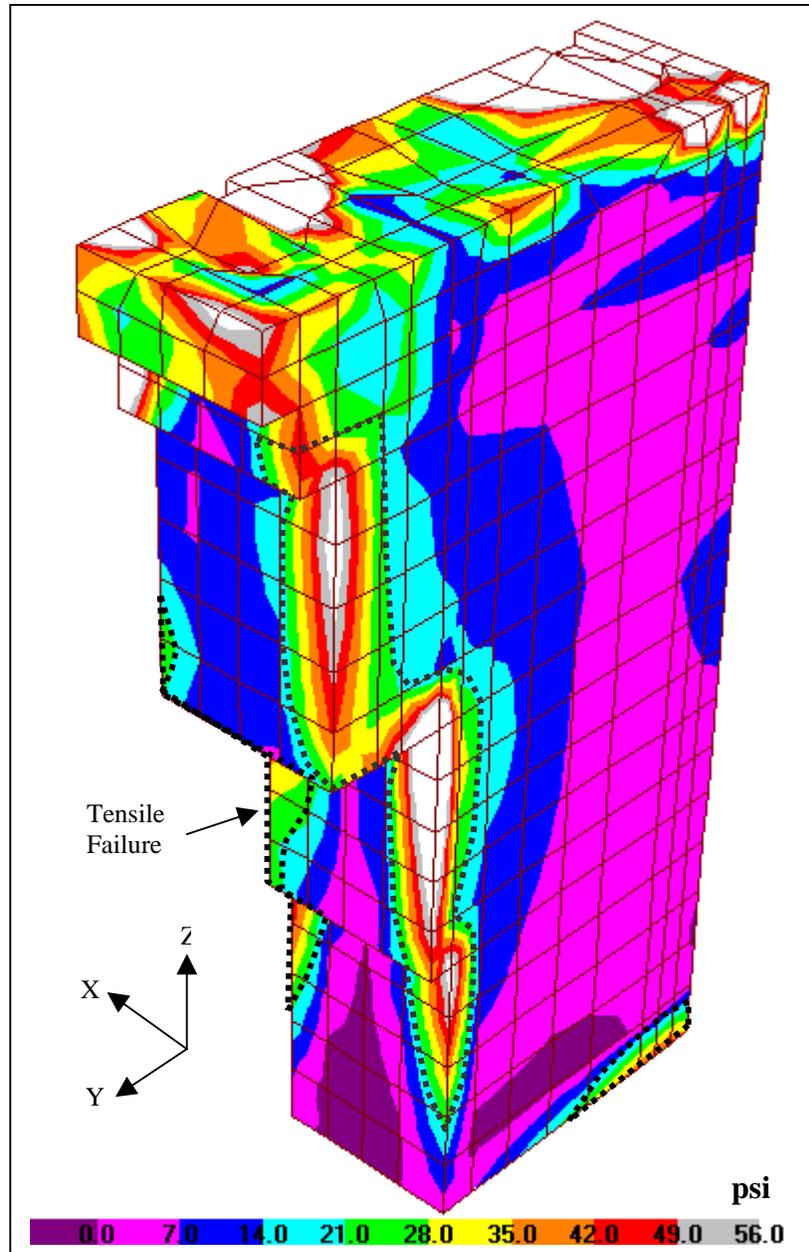
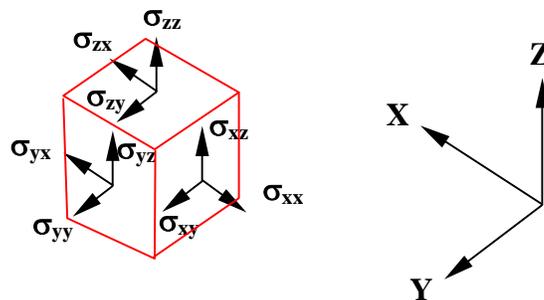


Figure 5-11a: Maximum horizontal normal stresses (σ_{xx}) for the south wall due to static plus earthquake loads. Regions within the dotted lines indicate potential tension failure.



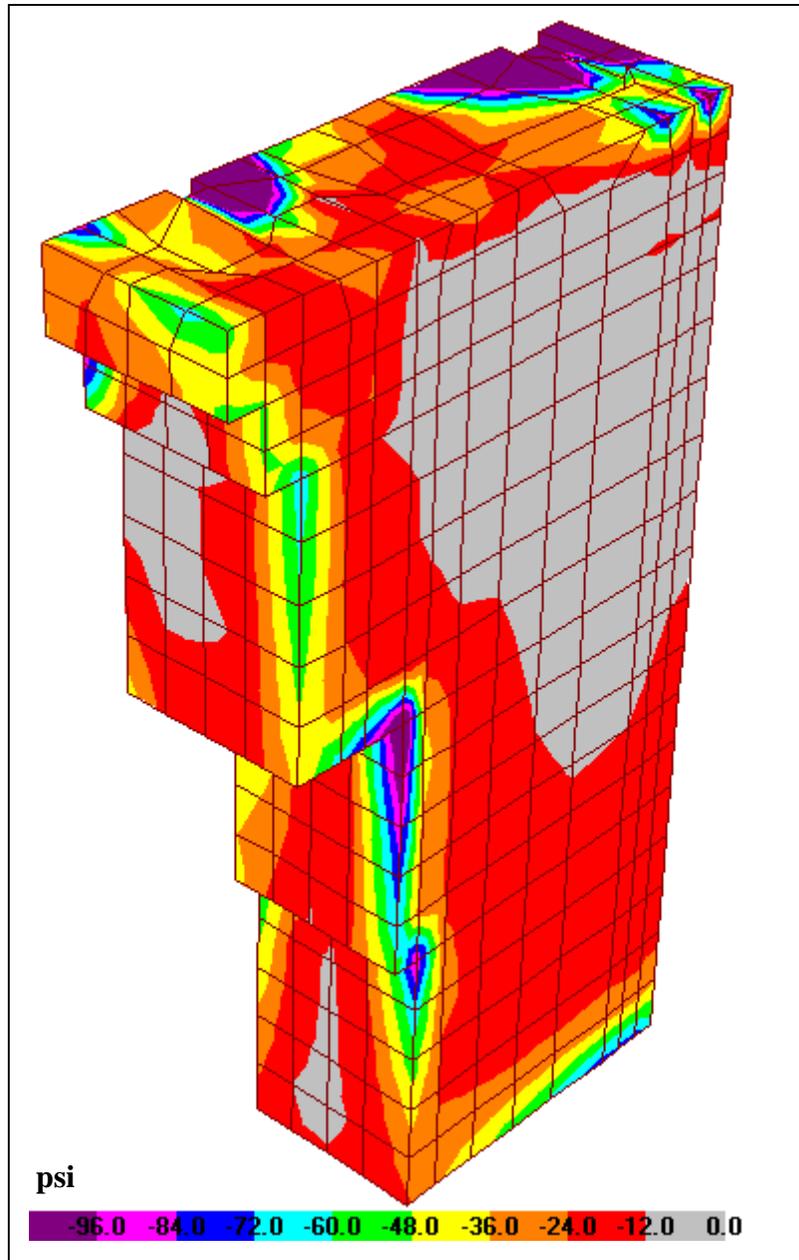
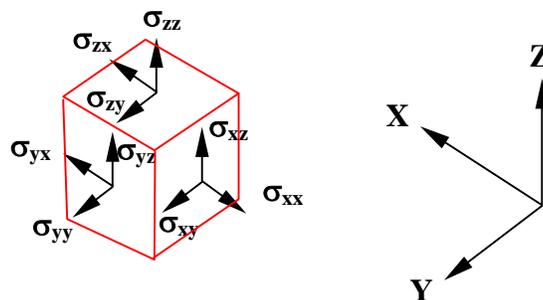


Figure 5-11b: Minimum horizontal normal stresses (σ_{xx}) for the south wall due to static minus earthquake loads.



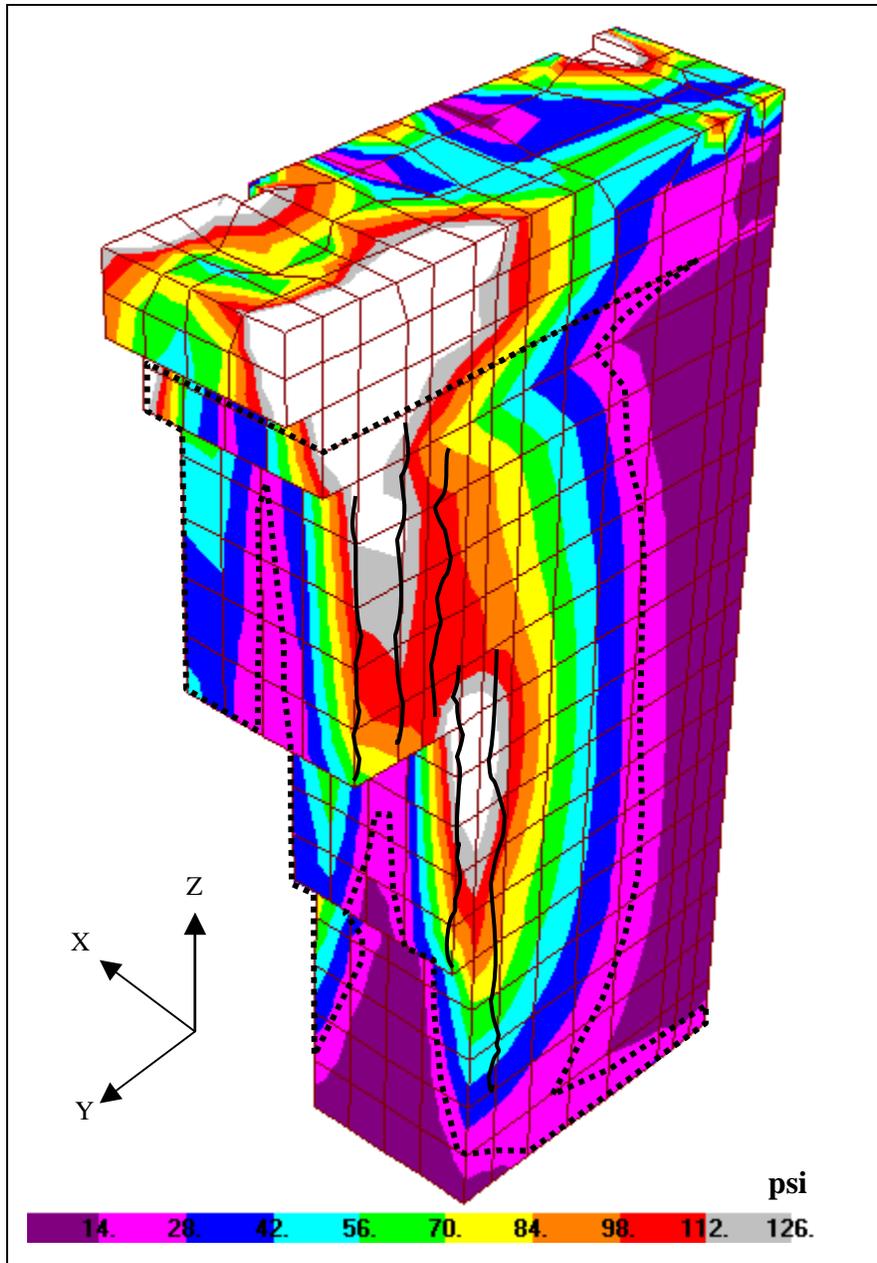
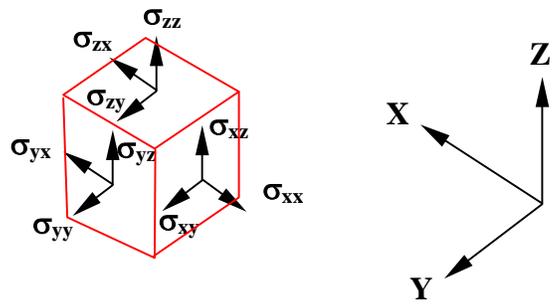


Figure 5-12a: Maximum horizontal normal stresses (σ_{yy}) for the south wall due to static plus earthquake loads. Regions within the dotted lines indicate potential tension failure.



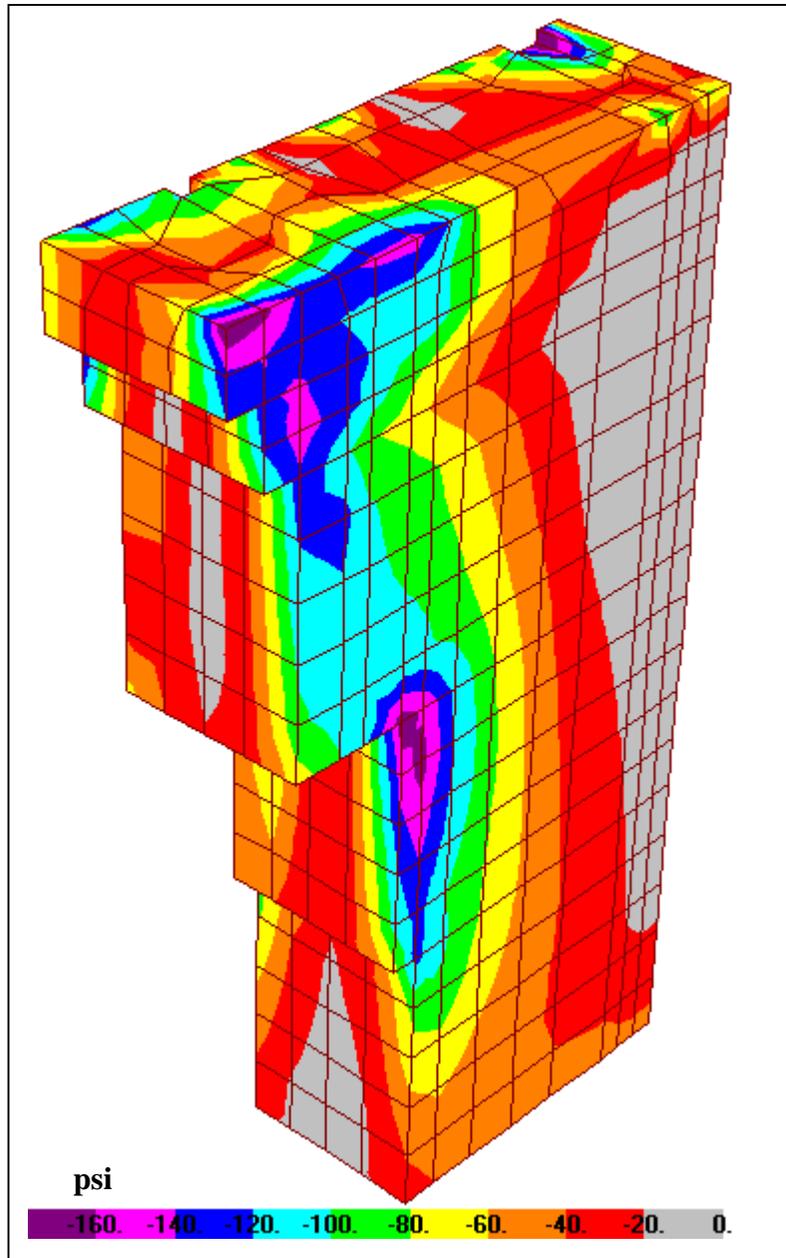
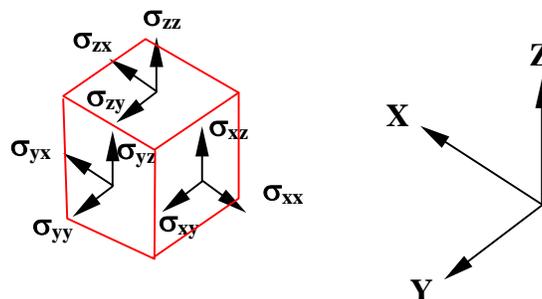


Figure 5-12b: Minimum horizontal normal stresses (σ_{yy}) in the south wall due to static minus earthquake loads.



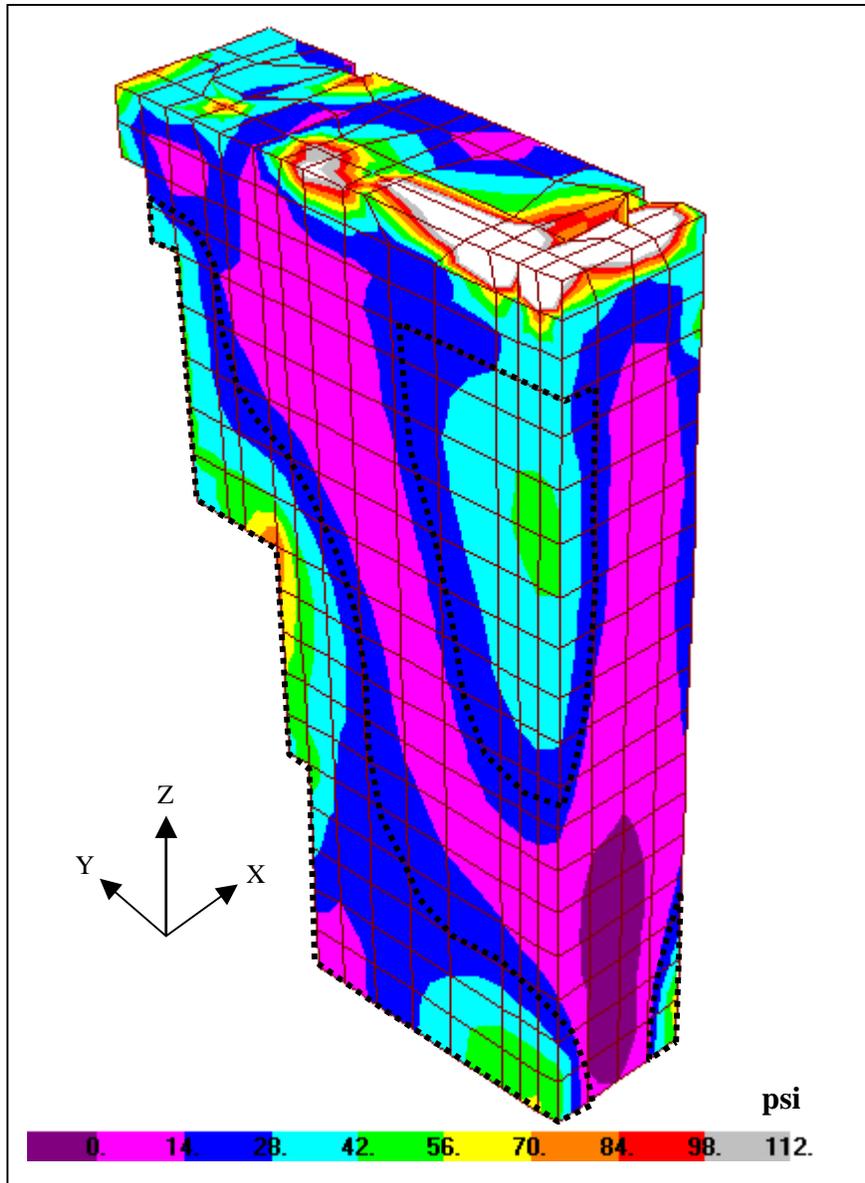
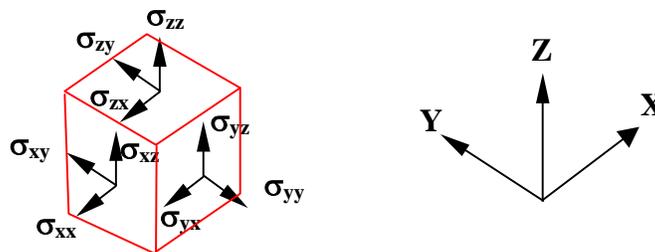


Figure 5-13a: Maximum vertical stresses (σ_{zz}) in the south wall due to static plus earthquake loads. Regions within the dotted lines indicate potential tension failure.



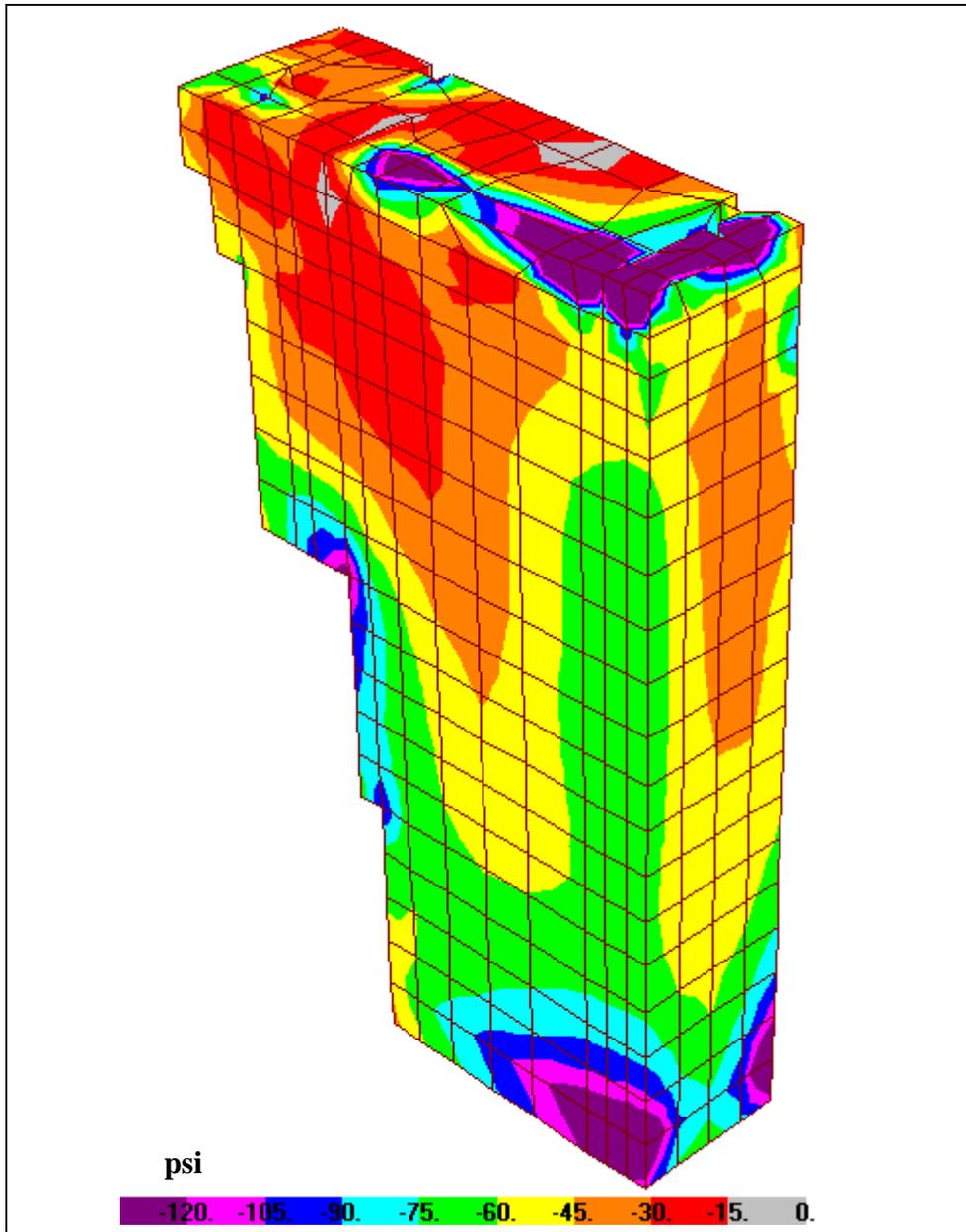
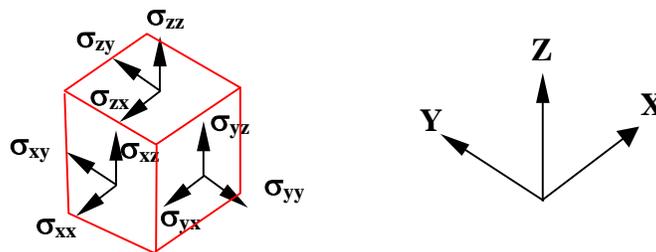


Figure 5-13b: Minimum vertical stresses (σ_{zz}) in the south wall due to static minus earthquake loads.



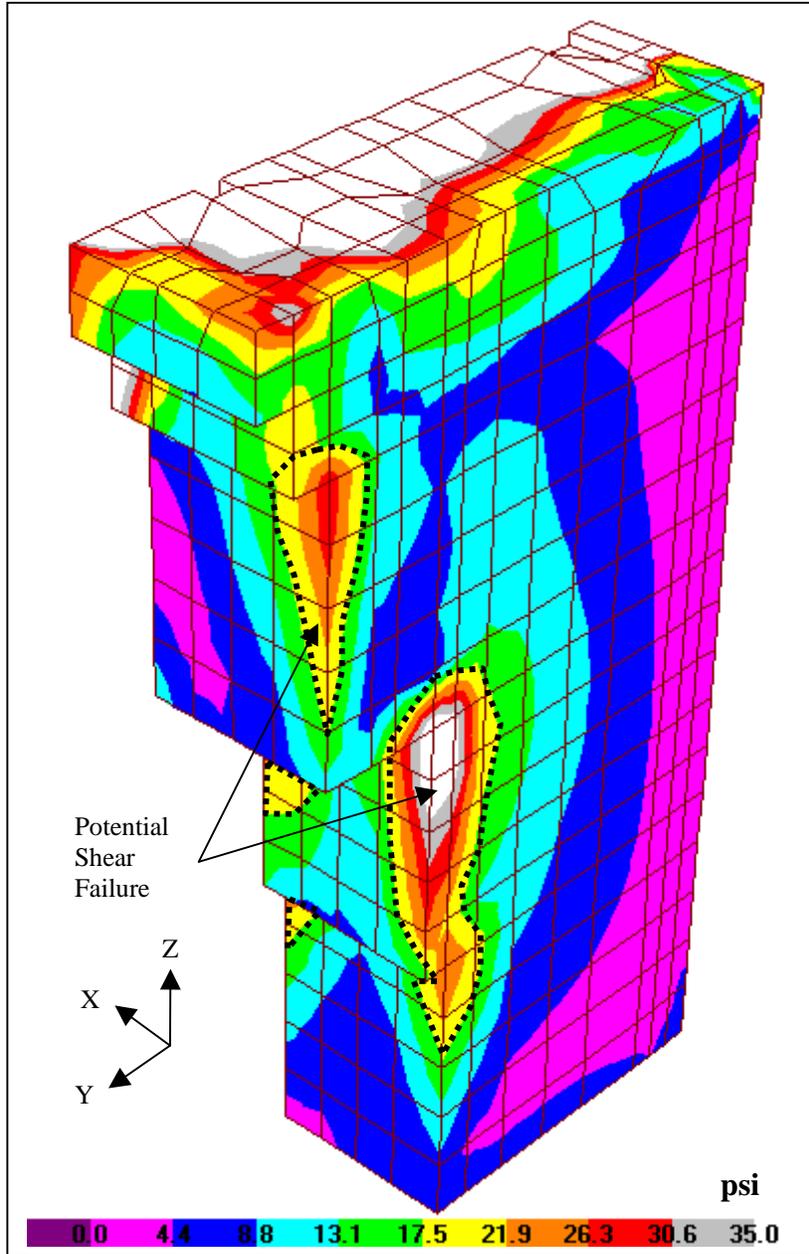
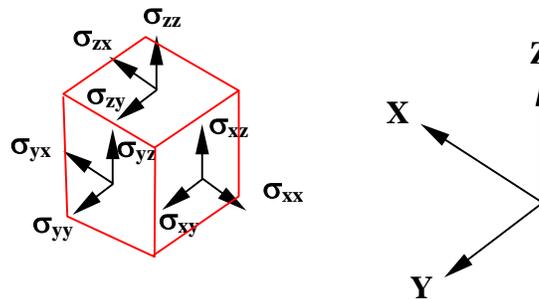


Figure 5-14a: Maximum out-of-plane shear stresses (σ_{xy}) for the south wall due to static plus earthquake loads.



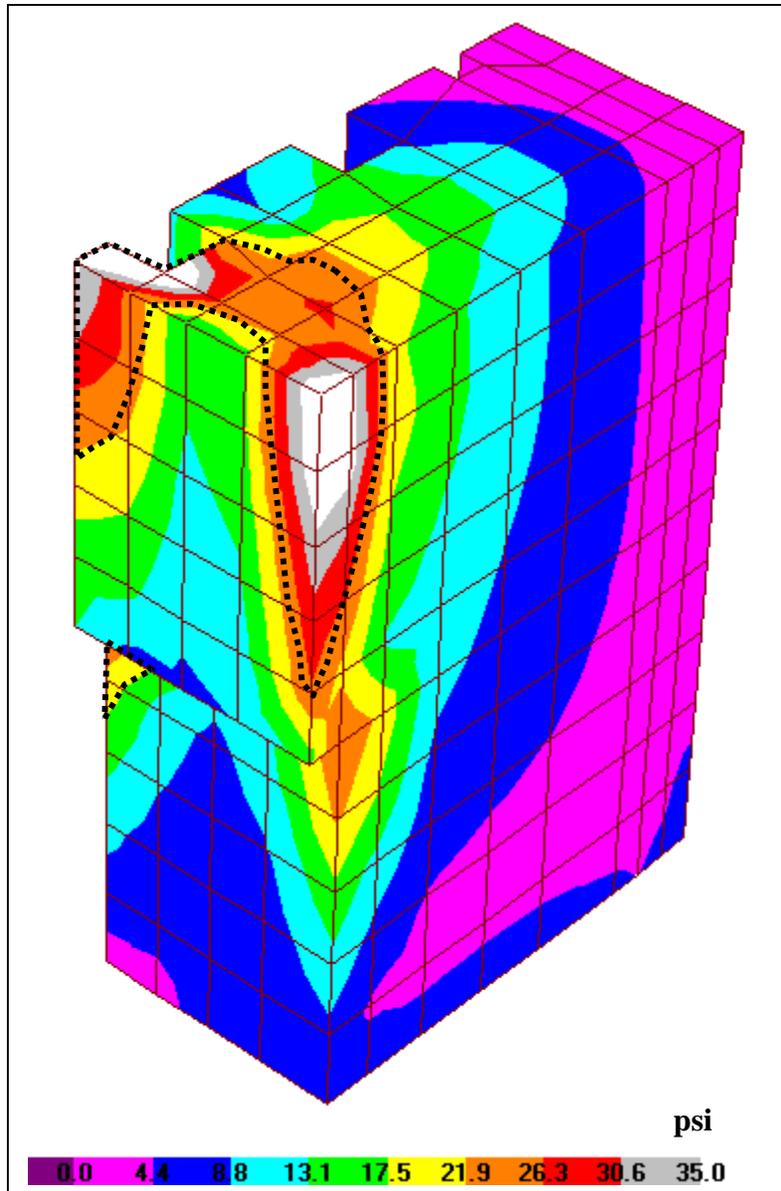
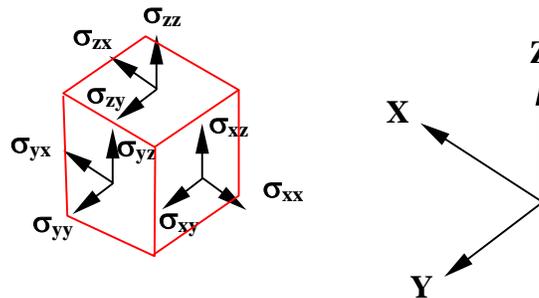


Figure 5-14b: Maximum out-of-plane shear stresses (σ_{xy}) on bottom half of the south wall due to static plus earthquake loads.



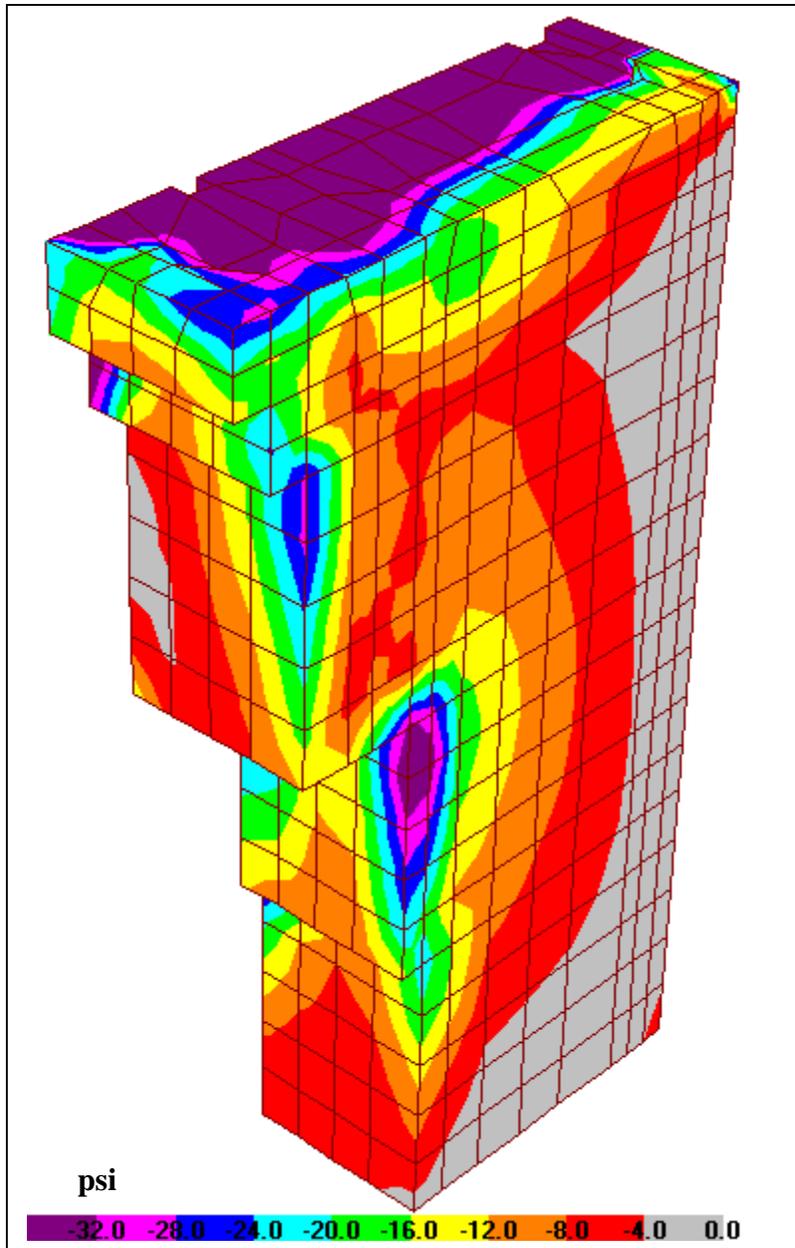
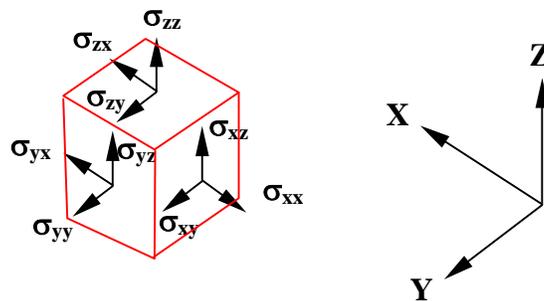


Figure 5-14c: Minimum out-of-plane shear stresses (σ_{xy}) for the south wall due to static minus earthquake loads



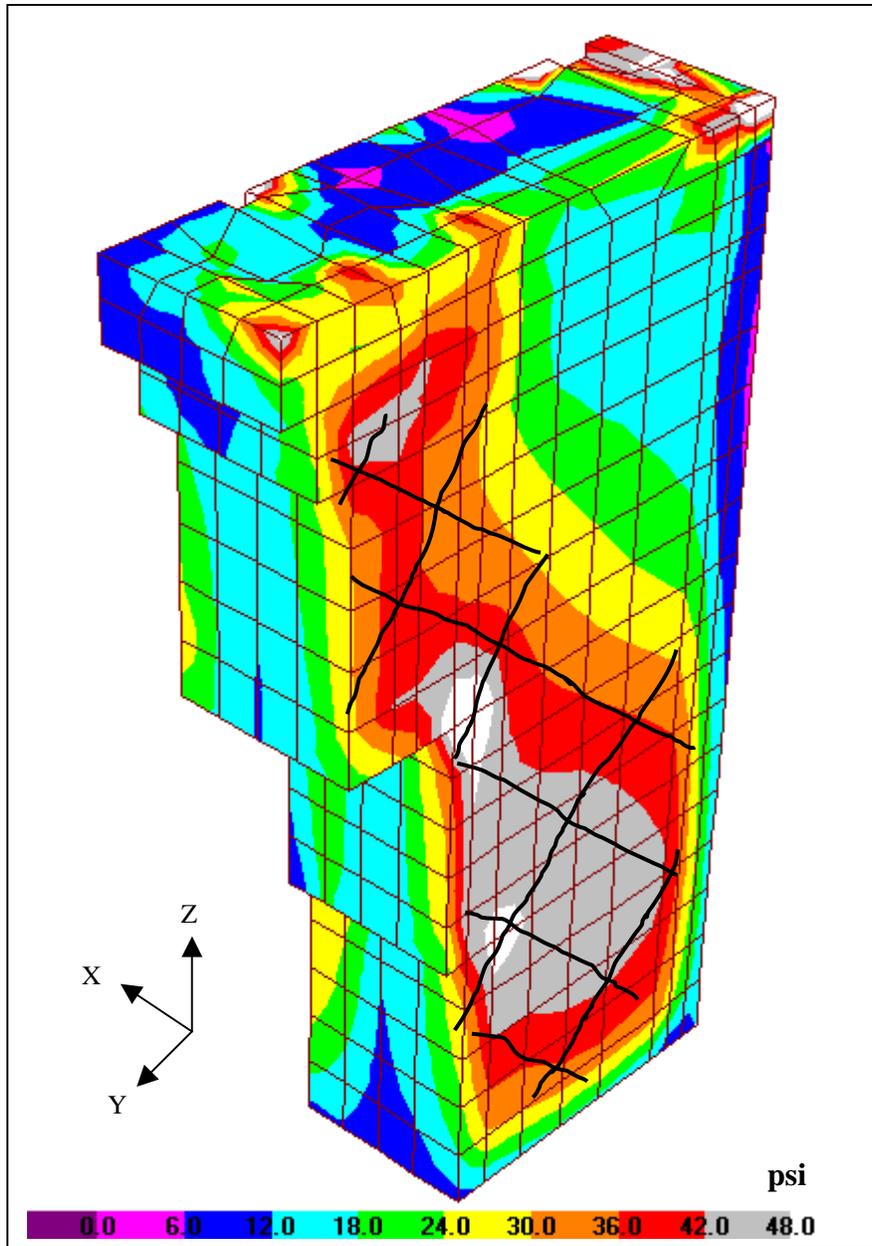
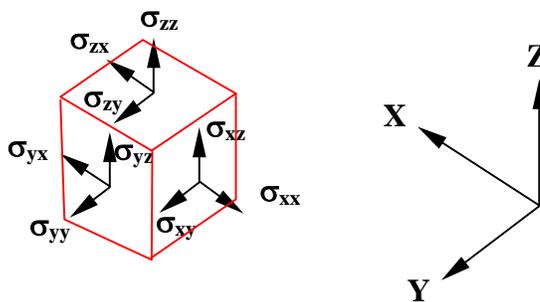


Figure 5-15a: Maximum in-plane shear stresses (σ_{yz}) in the south wall due to static plus earthquake loads



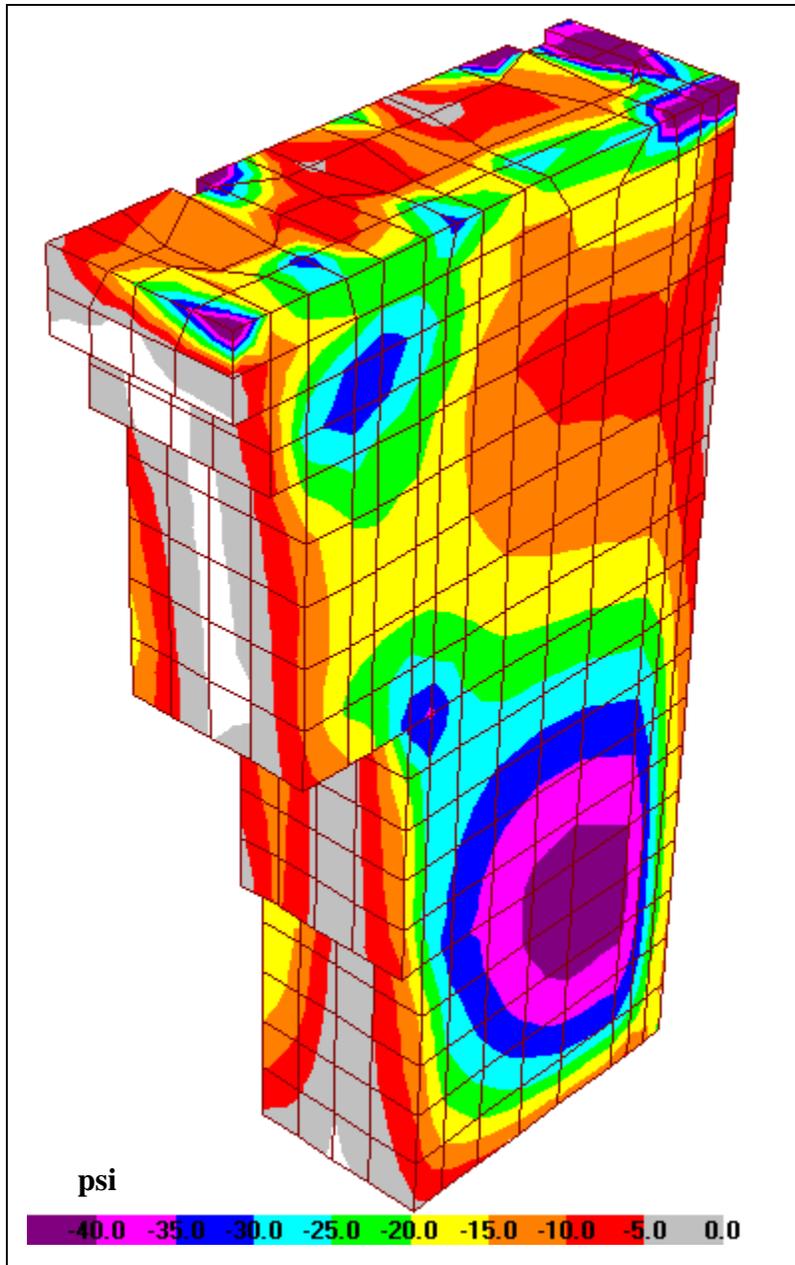
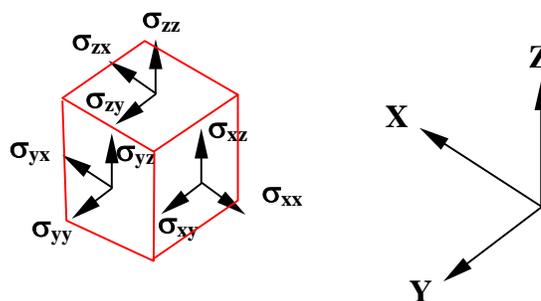


Figure 5-15b: Minimum in-plane shear stresses (σ_{yz}) in the south wall due to static minus earthquake loads



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 OLM 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	<u>7.2</u>	<u>3.07</u>
Dressed Stone	239	126	14	<u>9.0</u>	<u>7.91</u>
Stone	223	126	14	<u>9.0</u>	<u>86.81</u>

** The code values were obtained from the report by OLM 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 OLM 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.

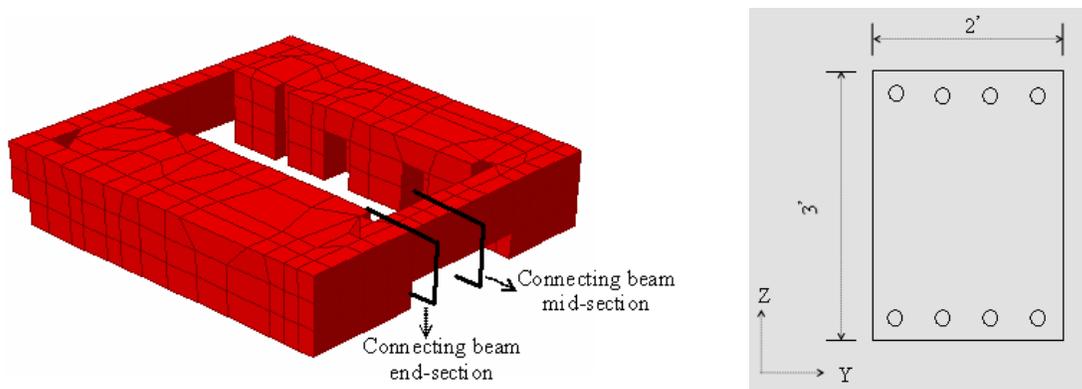


Figure 5-16: Sections chosen to assess the extent of damage for the connecting beam

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 OLM 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	<u>1.80</u>	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	<u>1.69</u>	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	<u>2.82</u>
	End	55	32	<u>1.72</u>	<u>3.96</u>

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	<u>5.93</u>	20.09	<u>2.25</u>
2	19	89.20	21.3	<u>4.19</u>	19.48	<u>2.18</u>
3	8	85.70	25.8	<u>3.32</u>	18.75	<u>2.10</u>
4	7	85.98	26.2	<u>3.28</u>	18.73	<u>2.10</u>
5	19	88.53	21.3	<u>4.16</u>	19.35	<u>2.17</u>
6	32	91.29	15	<u>6.09</u>	20.03	<u>2.24</u>
7	19	90.61	21.3	<u>4.25</u>	19.88	<u>2.23</u>
8	19	90.69	21.3	<u>4.26</u>	19.82	<u>2.22</u>
9	5	93.38	27	<u>3.46</u>	19.94	<u>2.23</u>
10	5	92.02	27	<u>3.41</u>	19.87	<u>2.23</u>
11	4	94.85	27.4	<u>3.46</u>	20.22	<u>2.26</u>
12	4	93.51	27.4	<u>3.41</u>	20.16	<u>2.26</u>
13	18	104.48	21.7	<u>4.81</u>	22.70	<u>2.54</u>
14	18	99.25	21.7	<u>4.57</u>	21.67	<u>2.43</u>
15	27	108.71	17.5	<u>6.21</u>	23.51	<u>2.63</u>
16	7	102.94	26.2	<u>3.93</u>	22.43	<u>2.51</u>
17	6	110.00	26.6	<u>4.14</u>	24.08	<u>2.70</u>
18	25	103.59	18.5	<u>5.60</u>	22.62	<u>2.53</u>

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.

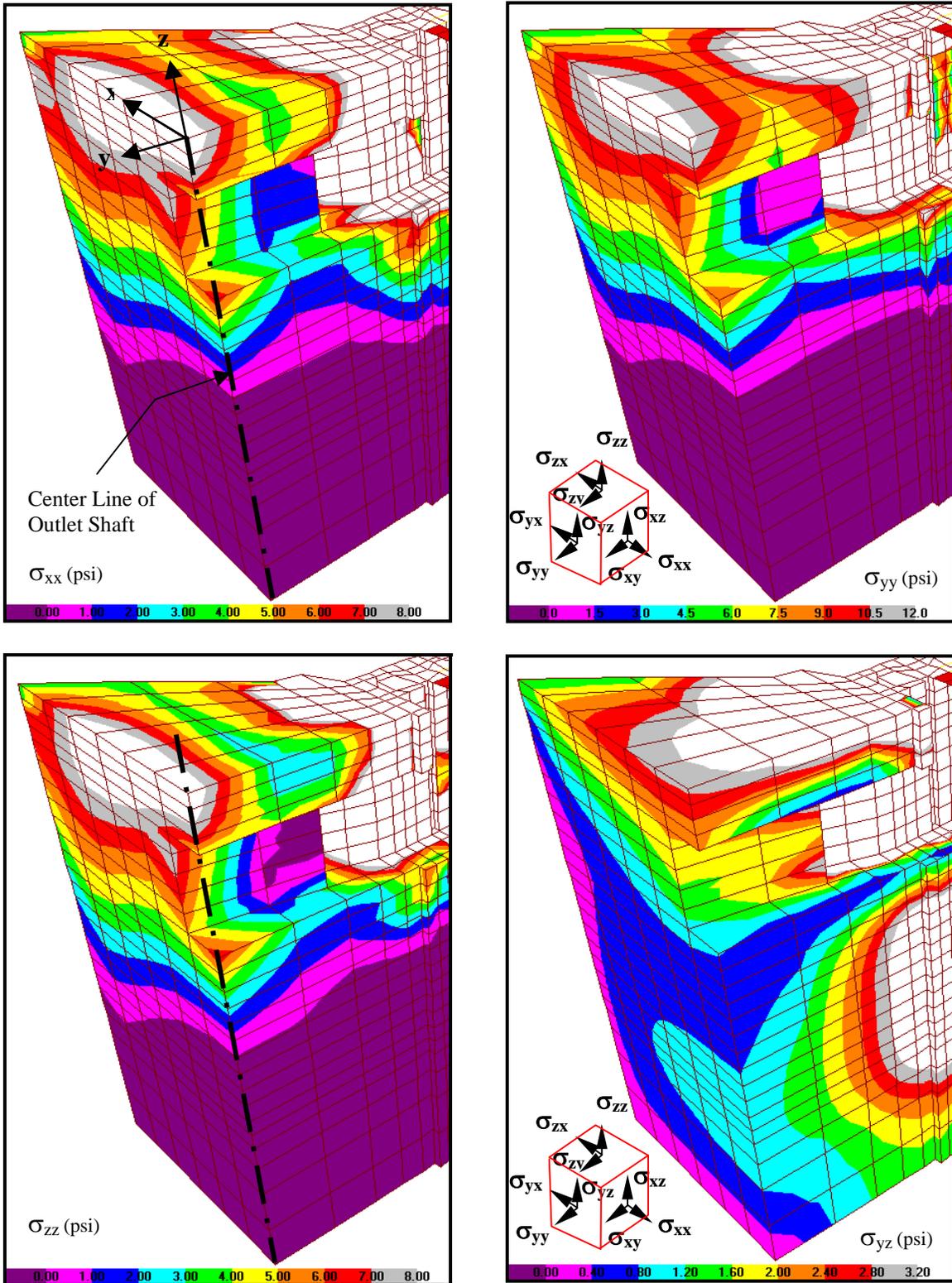


Figure 5-17: Normal and shear stresses in the vicinity of the outlet shaft

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 (Table 5-2) 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 possibility 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

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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

Tennebaum-Manheim Engineers
414 Mason Street, Suite 605
San Francisco, CA 94102

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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:

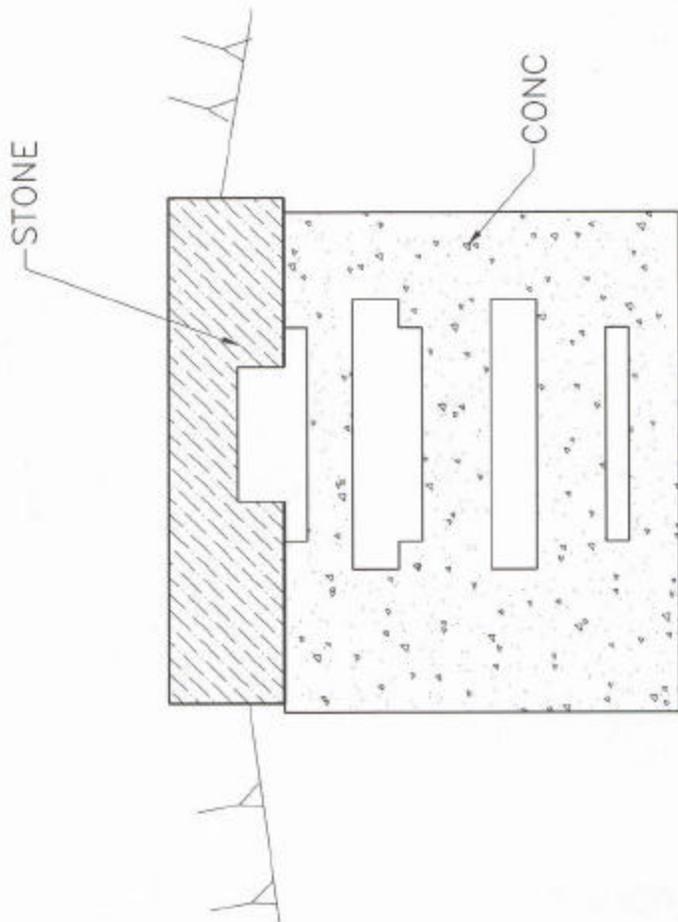
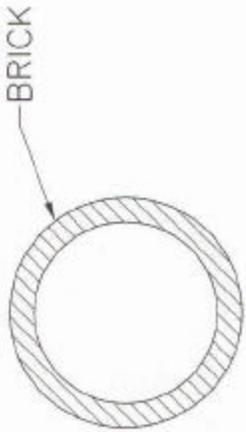
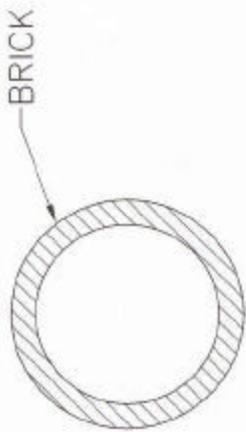
- 4'-6" of concrete from the floor slab to the top of brick.
- 18" of brick to the top of dressed stone.
- 3'-3" of dressed stone to the top of stone masonry which extends to the base of the tower.
- Stone masonry retaining begins about 9" below the bottom of dressed stone on the north face and continues 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.

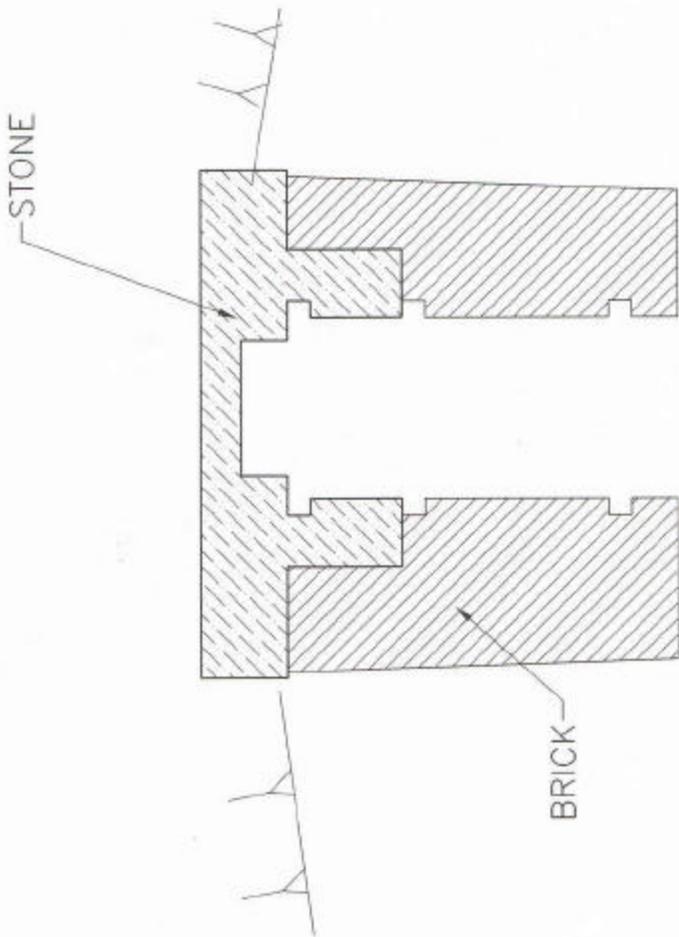
The information used in this report is based on the drawings listed on the following table; five photographs taken in 1924 (3 reproduced here), and two field visits on 5/7/04 on land and 5/18/04 by boat.

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



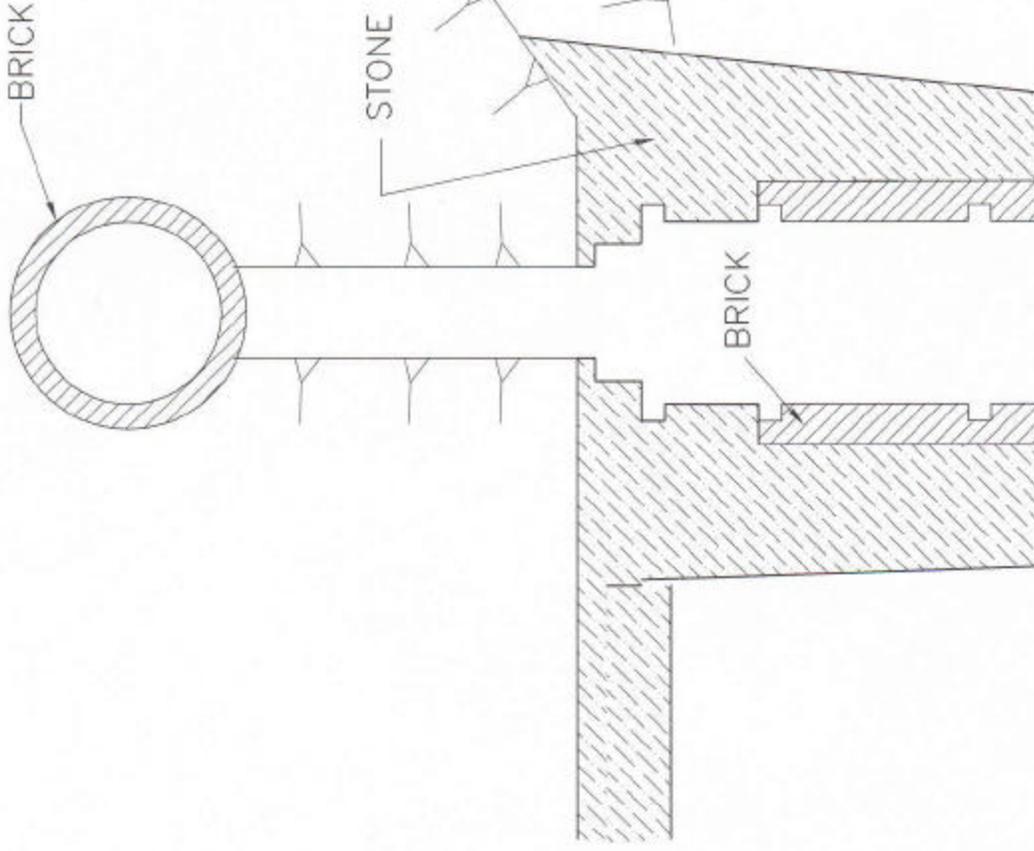
EL 241.6 TO 237.1



EL 237.1 TO 235.6

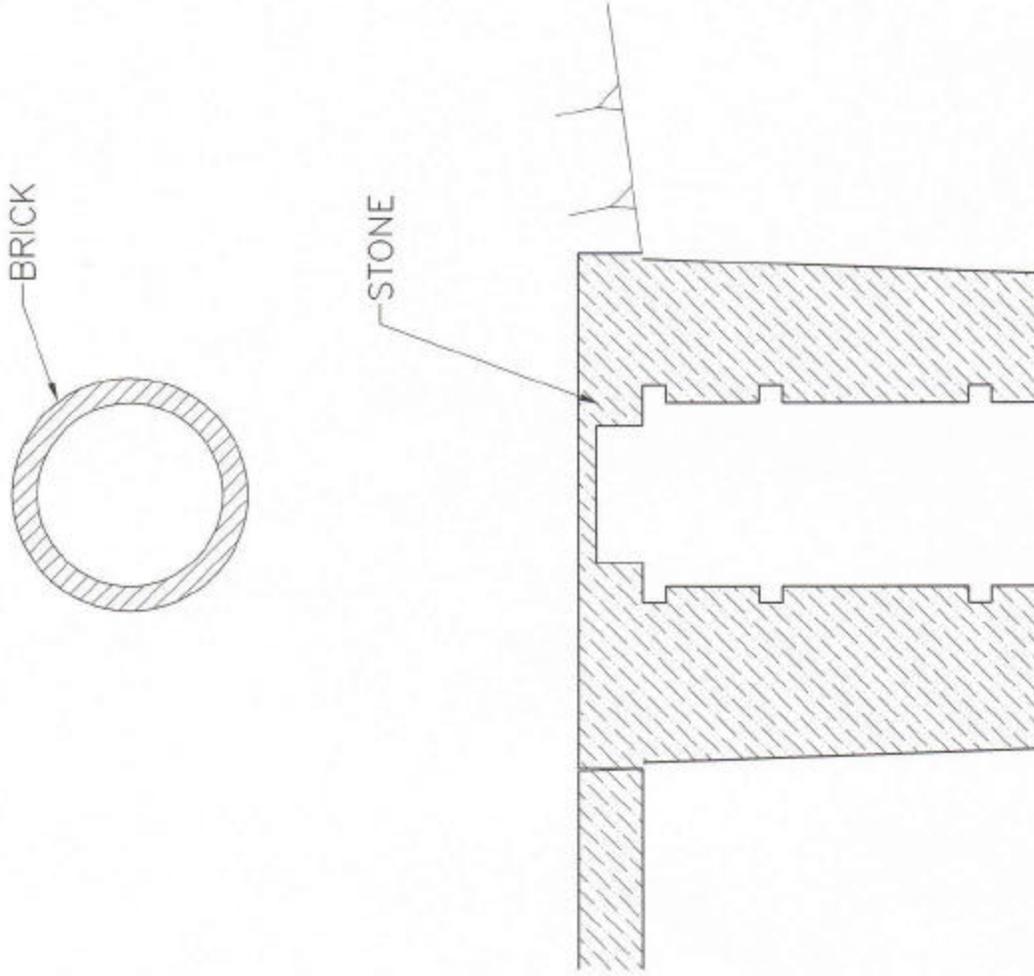
PLAN DETAIL @ TUNNEL
#2 OUTLET TOWER





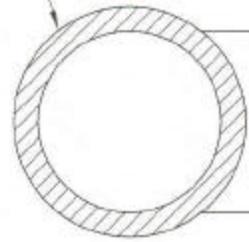
EL 232.3 TO 228.3

PLAN DETAIL @ TUNNEL
#2 OUTLET TOWER



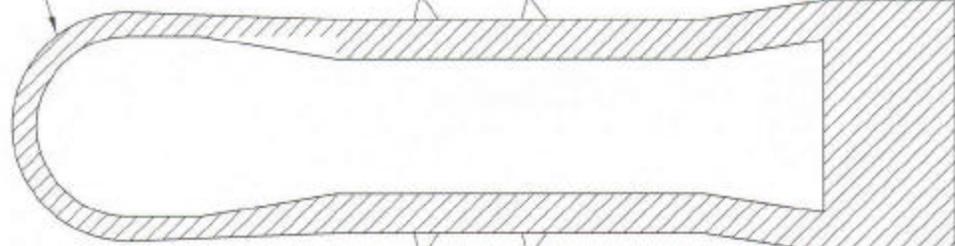
EL 235.6 TO 232.3

BRICK

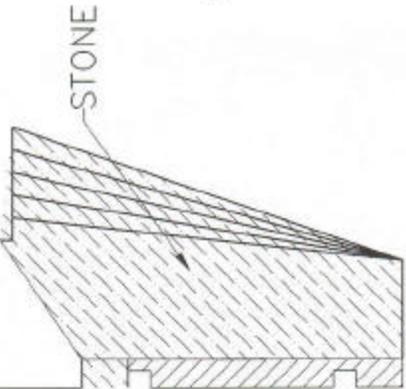


CONDITIONS SHOWN BASED ON E1107
AND 1924 PHOTOGRAPHS

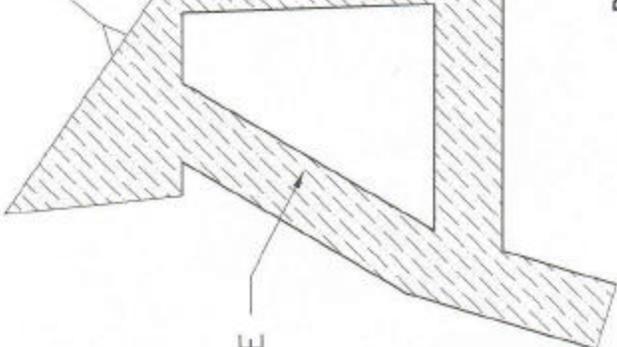
BRICK



STONE



STONE



PLAN DETAIL @ TUNNEL
#2 OUTLET TOWER



EL 228.3 TO ~207.0

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 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		Brick Density: 120 pcf	Stone Density: 160 pcf
	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		

* 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 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 1 order 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

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 4,500 psi 2,500 psi 1,500 psi	350 225 160 115	300 200 140 100
Grouted masonry, of clay or shale; sand-lime or concrete: 4,500 plus, psi 2,500 psi 1,500 psi	275 215 175	200 140 100
Solid masonry of solid concrete masonry units: 3,000 plus, psi 2,000 psi 1,200 psi	225 160 115	200 140 100
Masonry of hollow load-bearing units: 2,000 plus, psi 1,500 psi 1,000 psi 700 psi	140 115 75 60	120 100 70 55
Hollow walls (cavity or masonry bonded) ² solid units: 2,500 plus, psi 1,500 psi	160 115	140 100
Hollow units	75	70
Stone ashlar masonry: Granite Limestone or marble Sandstone or cast stone Rubble stone masonry Coarse, rough or random	720 450 360 120	640 400 320 100
Unburned clay masonry	30	—

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

²Where 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) × 25.4 for mm	EMBEDMENT ¹ (inches)	SOLID MASONRY (shear in pounds)	GROUTED MASONRY (shear in pounds) × 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

¹An 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.

²Permitted 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

Property	Masonry Condition ¹		
	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 Strength³			
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 Properties¹

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 Periods¹*

Year	Grade	Structural ²	Intermediate ²	Hard ²	60	70	75
		33	40	50			
	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

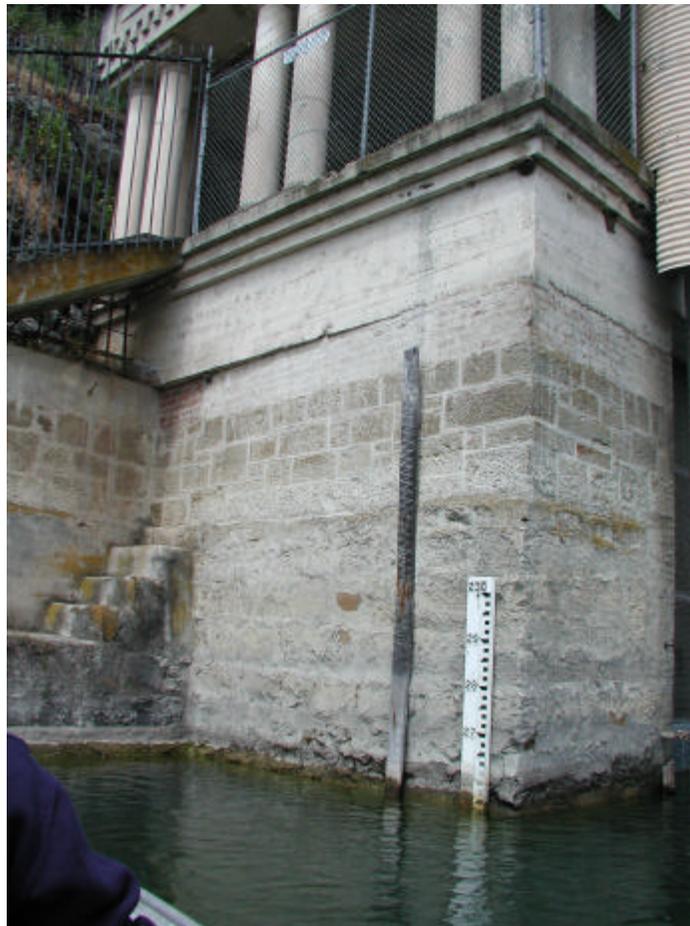


VIEW INSIDE TOWER

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

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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



OLMM
CONSULTING
ENGINEERS

www.olmm.com

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

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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 ASSESSMENT OF EXISTING CONDITIONS

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

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have adversely affected the capacities of the members, because there are no perceptible excessive deflections. However, as in any case where reinforcement has begun 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 R_w 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.

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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:

- $N_a = 1.2$
- $N_v = 2.0$

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- $C_a = 0.6$
- $C_v = 0.8$
- $V = 1.125 \times (\text{Weight of Structure})$ (EQ. 30-5, 2001 CBC)
- $V_{min} (\text{Other Non-Building Structure}) = 0.96 \times (\text{Weight of Structure})$ (EQ. 34-3, 2001CBC)

The original EBMUD report used a base shear of $V = 0.54 \times (\text{Weight of Structure})$.

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

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	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

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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

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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".

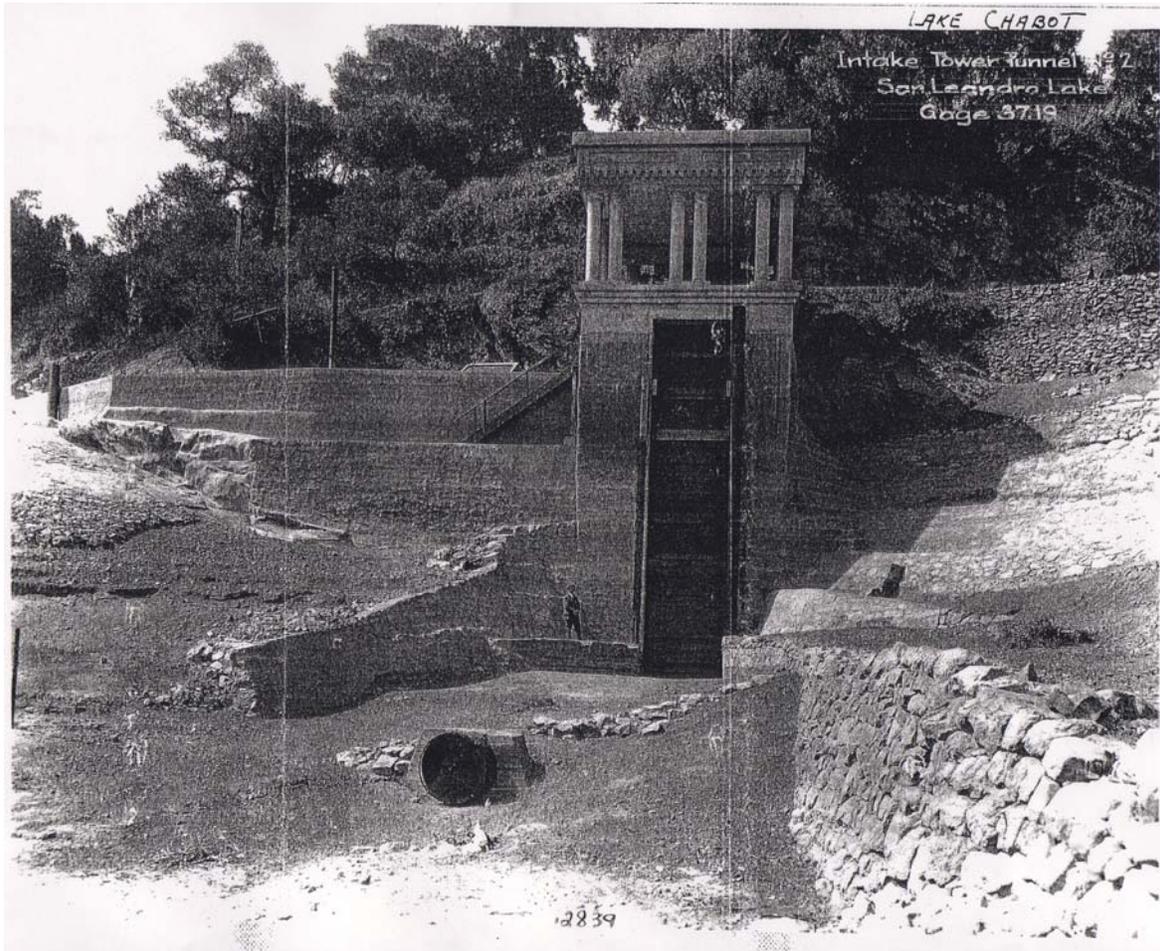
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2. East Bay Municipal Utility District report dated April 25, 1991” “Seismic Evaluation of Chabot Reservoir - Tunnel No. 2, Inlet Structure”.
3. “Reinforced Masonry Engineering Handbook. Brick and Other Structural Clay Units.” By J.M. Amrhein. Copyright © 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.

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PHOTOGRAPHS

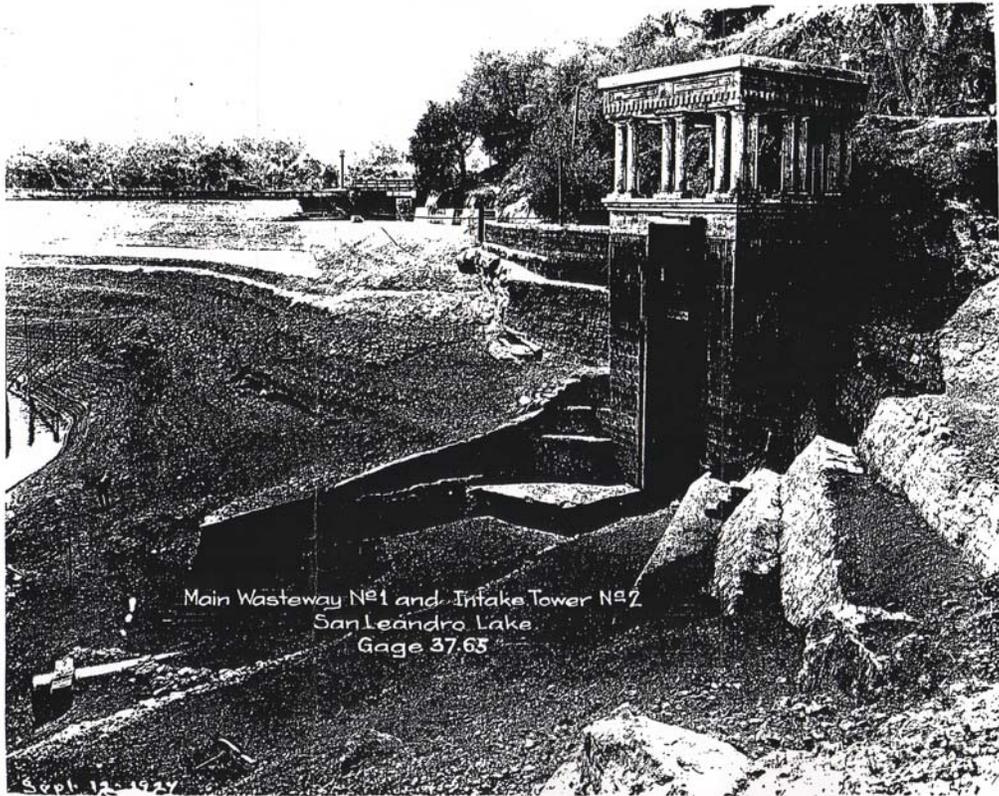
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Intake Tower #2
Chabot Reservoir
San Leandro, CA. 1924

Photograph 1.

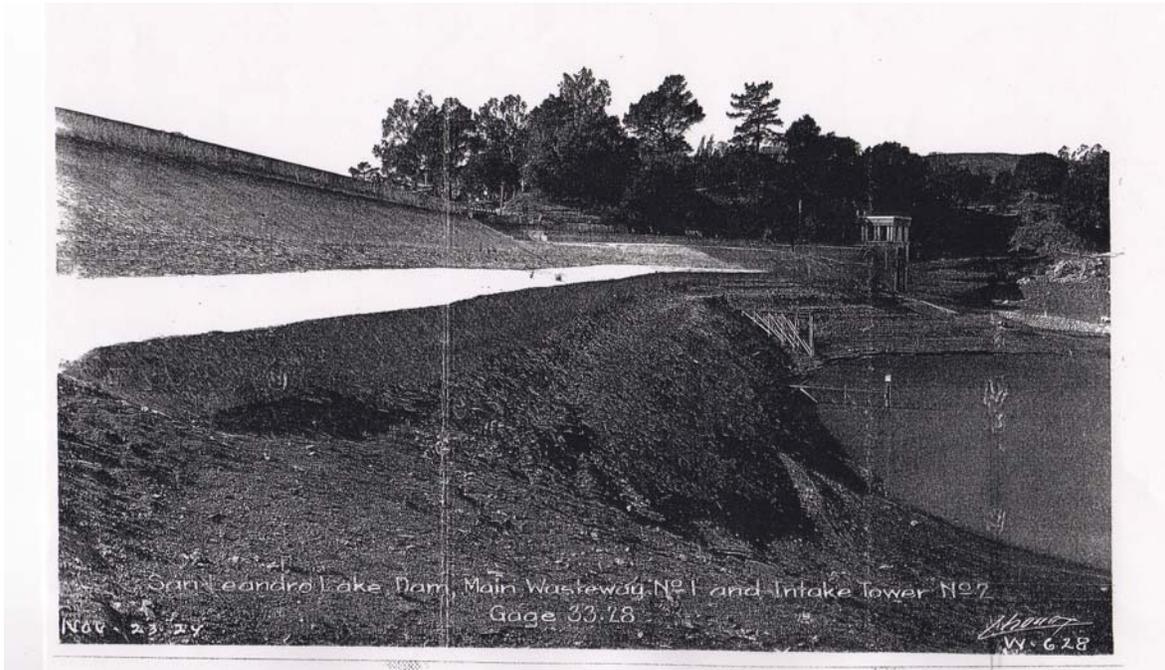
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Intake Tower #2
Chabot Reservoir
San Leandro, CA. 1924

Photograph 2.

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

Photograph 3.

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

Photograph 4.

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East Face of Intake Tower

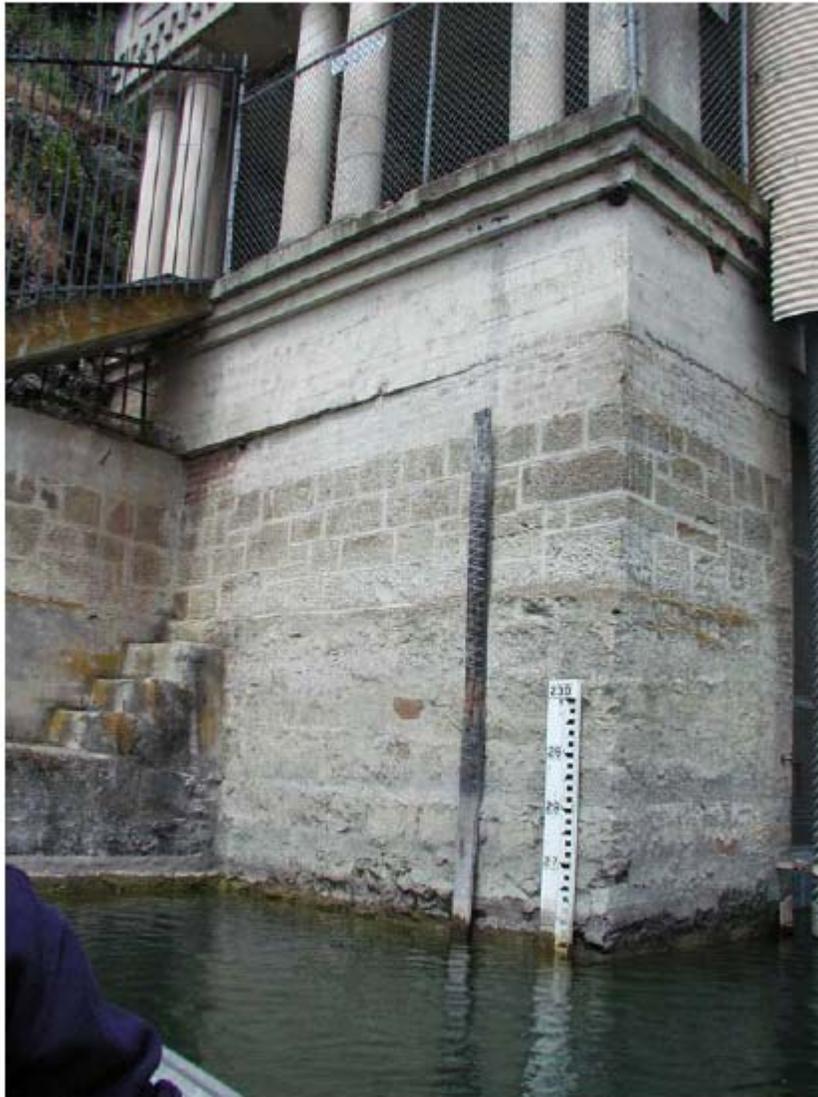
Photograph 5.



South View of Intake Tower

Photograph 6.

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

Photograph 7.

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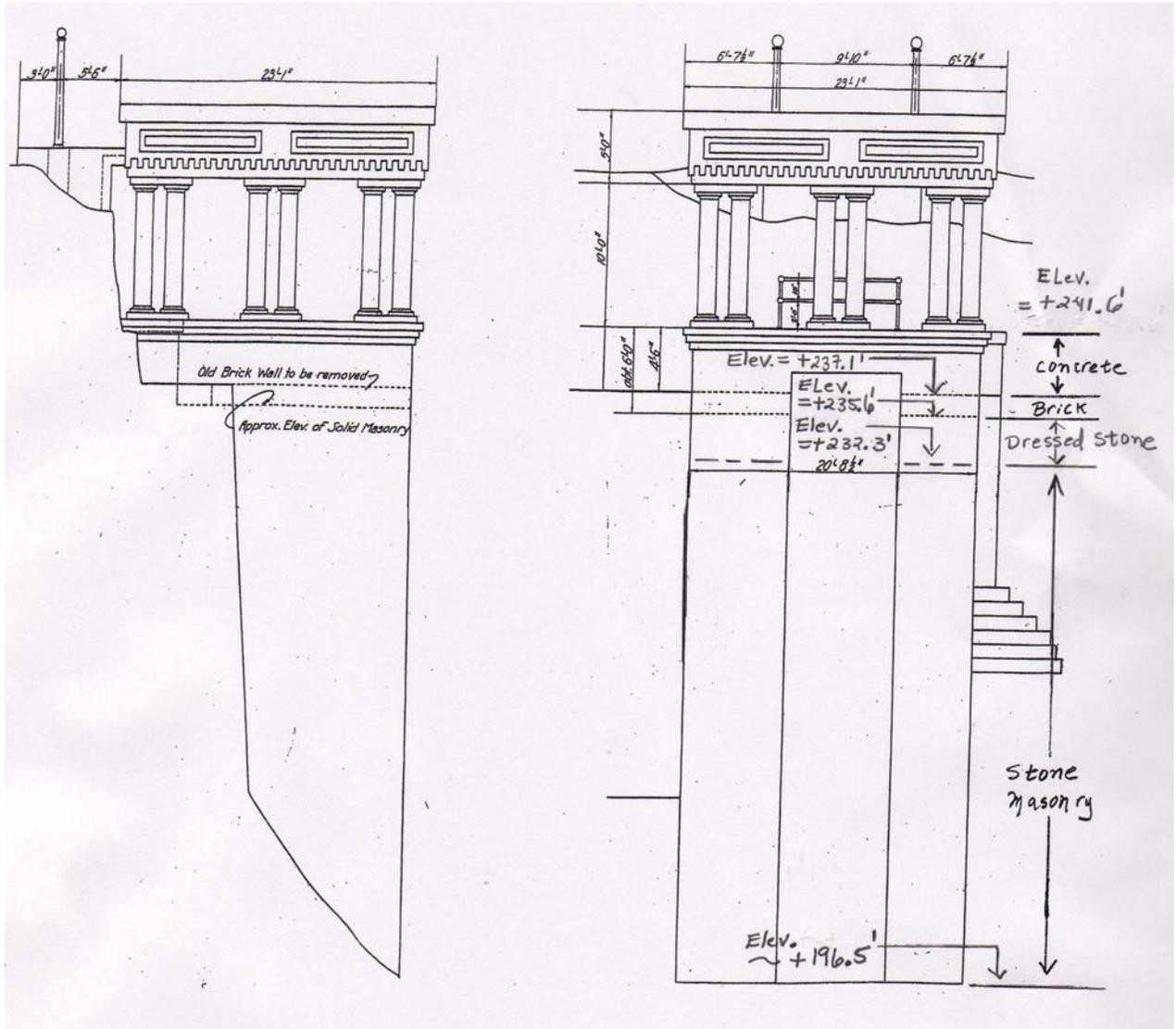


Concrete Spalling at Pavilion Roof

Photograph 8.

FIGURES

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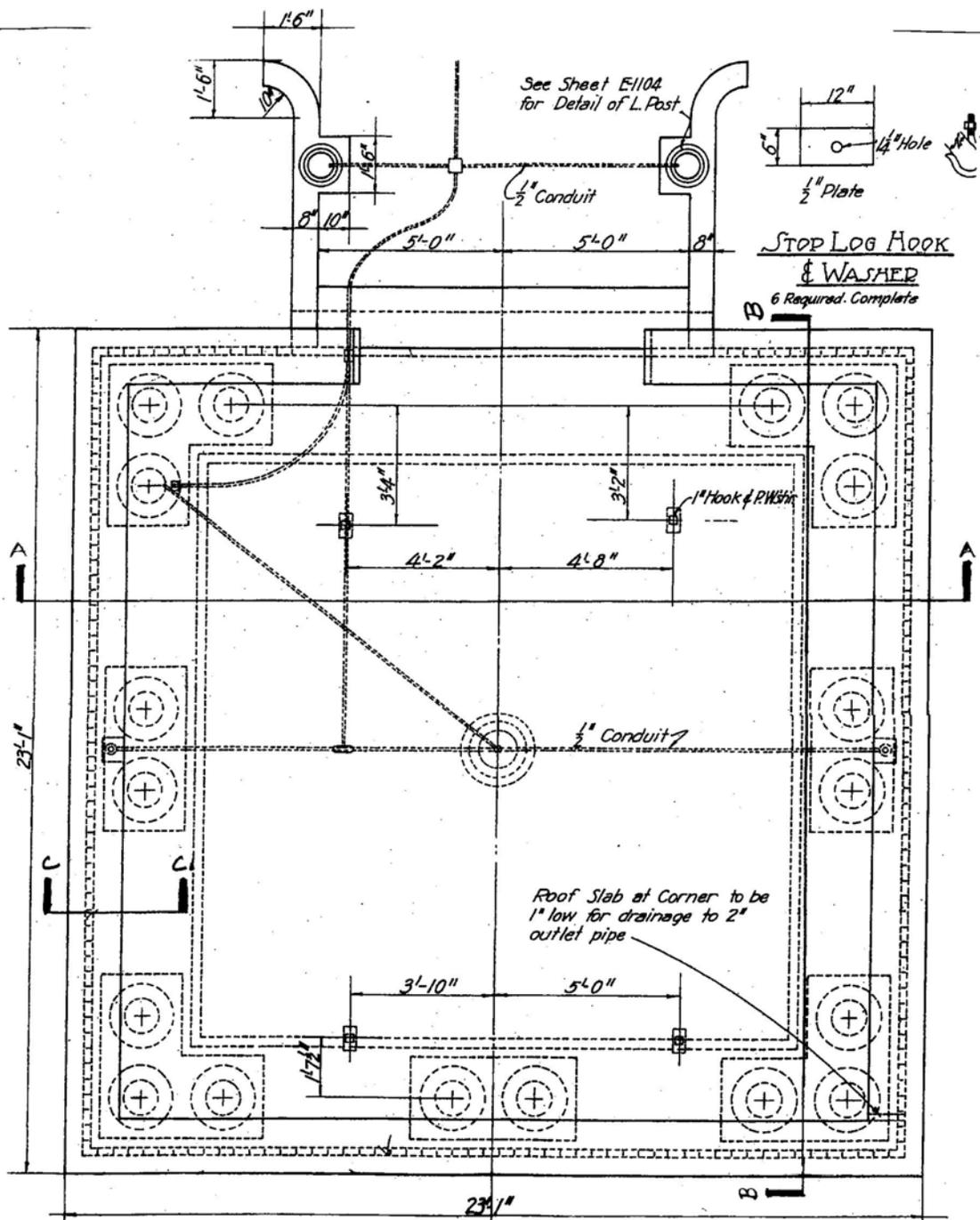
South Elevation

East Elevation

**South and East Elevations
 Of Intake Tower**

Figure 1.

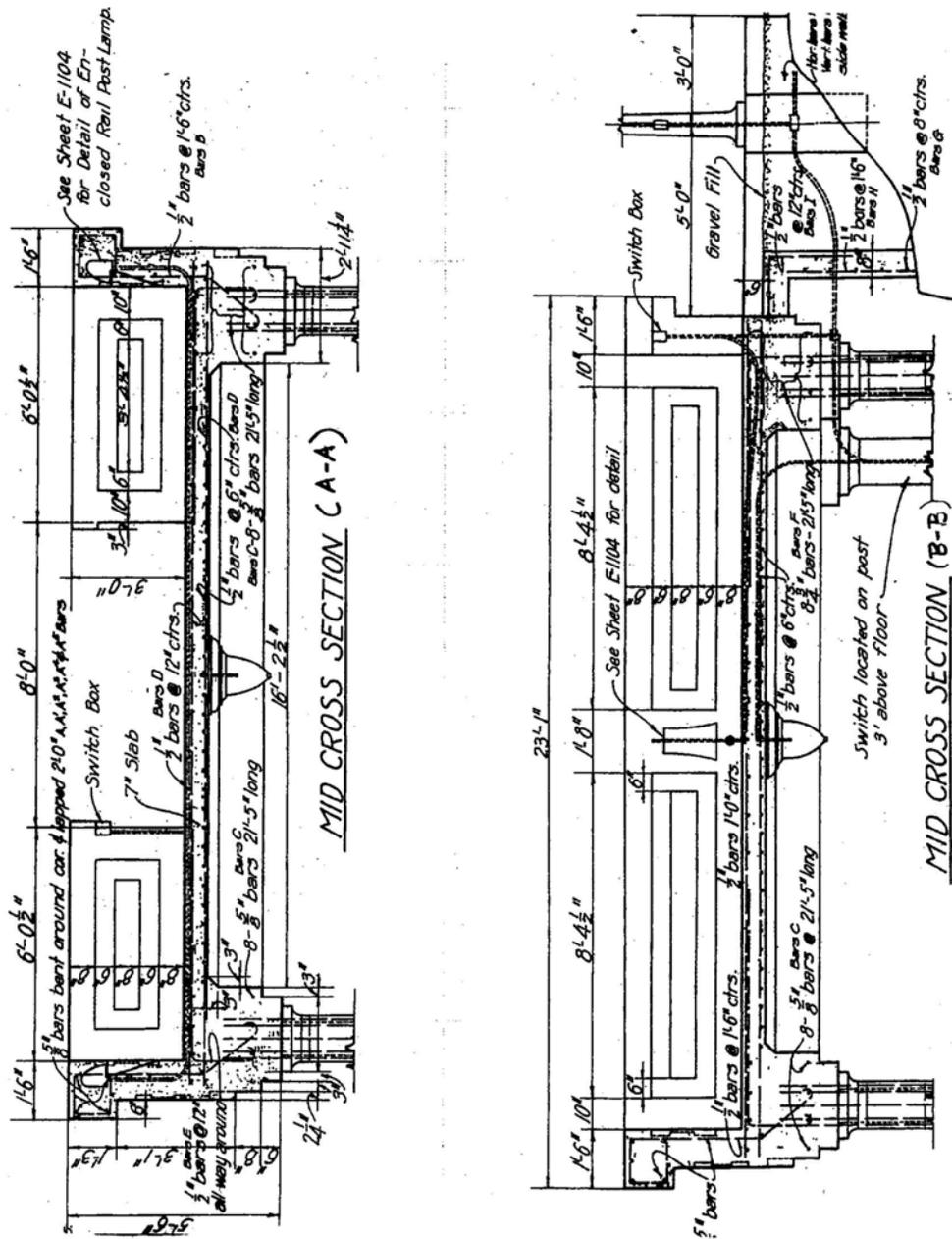
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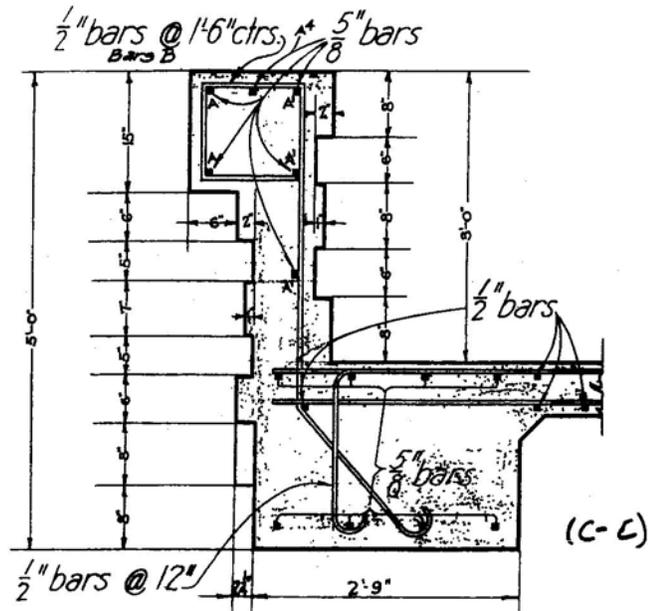
Pavilion Roof Framing Plan

Figure 2.

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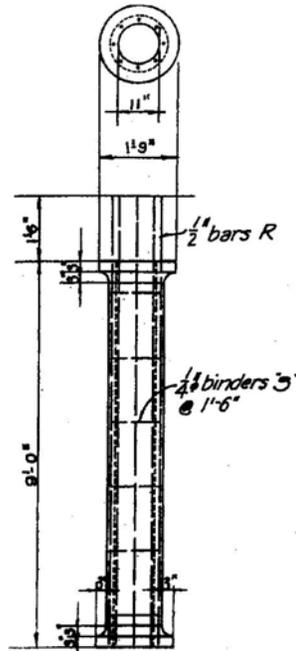


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Cross-Section of Pavilion Roof Beam

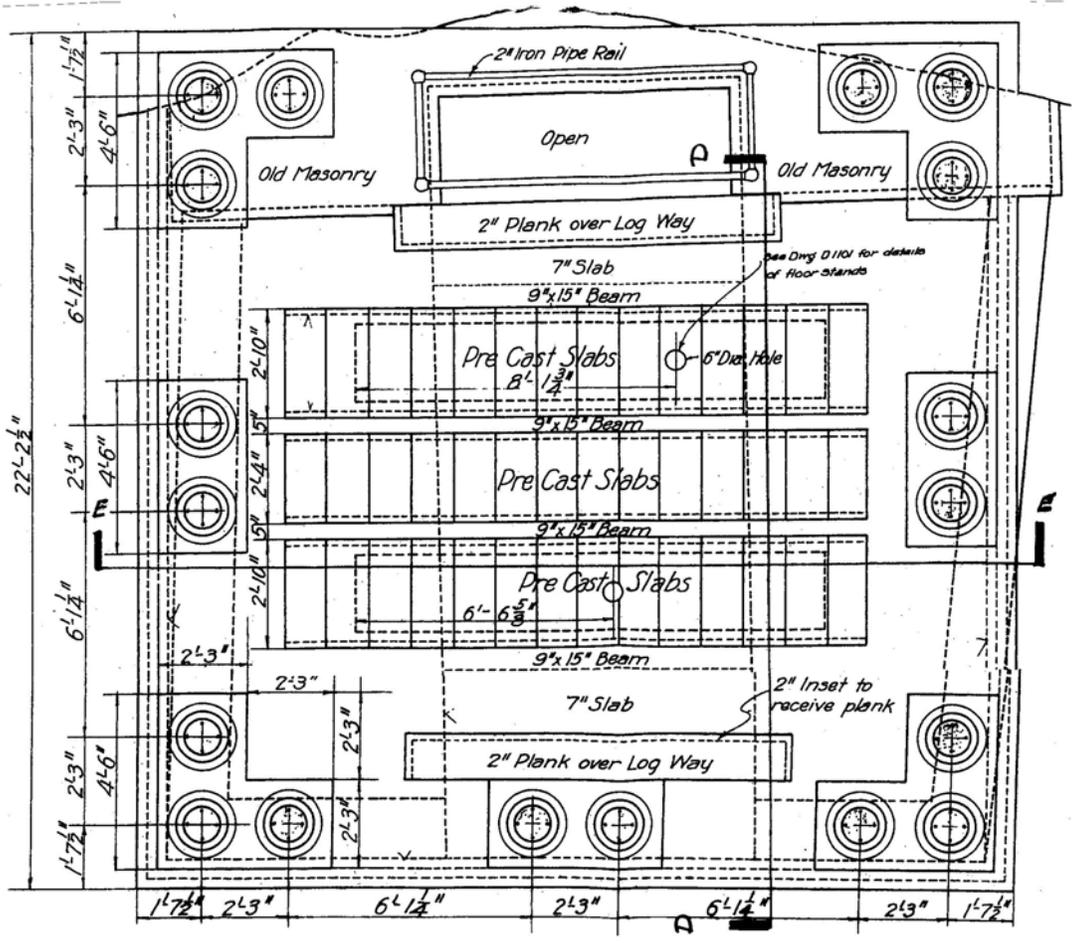
Figure 4.



Elevation of Pavilion Column

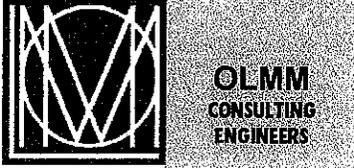
Figure 5.

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**Pavilion Floor
 Framing Plan**

Figure 6.



APPENDIX – A

Quantitative Analysis and Evaluation Calculations

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



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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

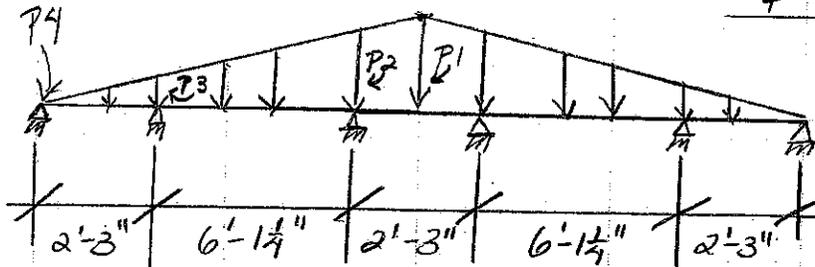
CHECKED BY _____

DATE

SCALE _____

Roof Beam Loads:

7" Thick 2-Way slab



$$\text{Parapet} \rightarrow W_{DL} = 0.15 \text{ k/ft}^3 (1.5 \text{ ft}) (1.25 \text{ ft}) + 0.15 \text{ k/ft}^3 (1 \text{ ft}) (3 \text{ ft} - 1.25 \text{ ft})$$

$$= 0.544 \text{ k/ft} \quad (23 + 1/12) \text{ ft} - 2.75 \text{ ft} - ((2.25 + 6) / 12) (2)$$

$$P_{IDL} = 0.544 \text{ k/ft} + (18.96 \text{ ft}/2) (7/12 \text{ ft}) (0.15 \text{ k/ft}^3) = 18.96 \text{ ft}$$

$$P_{IDL} = 0.544 \text{ k/ft} + 0.83 \text{ k/ft} = 1.374 \text{ k/ft}$$

$$P_{ILL} = (18.96 \text{ ft}/2) (20 / 1,000 \text{ k/sf}) = 0.19 \text{ k/ft}$$

↑
See Figure 41

$$P_{2DL} = 0.544 \text{ k/ft} + 0.83 \text{ k/ft} (8.35 \text{ ft} / 9.48 \text{ ft}) = 1.275 \text{ k/ft}$$

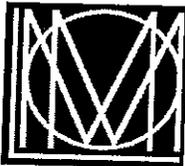
$$P_{2LL} = 0.19 \text{ k/ft} (8.35 \text{ ft} / 9.48 \text{ ft}) = 0.167 \text{ k/ft}$$

$$P_{3DL} = 0.544 \text{ k/ft} + 0.83 \text{ k/ft} (2.25 \text{ ft} / 9.48 \text{ ft}) = 0.741 \text{ k/ft}$$

$$P_{3LL} = 0.19 \text{ k/ft} (2.25 \text{ ft} / 9.48 \text{ ft}) = 0.045 \text{ k/ft}$$

$$P_{4DL} = 0.544 \text{ k/ft}$$

$$P_{4LL} = 0$$



OLMM
CONSULTING
ENGINEERS

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

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

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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		
FAULT TYPE	A		Given Na? No
NEAR SOURCE DISTANCE(km)	0.5	Km	Given Nv? No
Z	0.4		If "Yes", Input Data
Ct	0.02		Na(Given) 1.5
S	SB		Nv(Given) 2
R	2		
PERIOD T _B (formula 30-10)	0.11	s	Ω _o
Meet 1629.4.2. Requirement?	No		W = 131 kips
		("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) ^{-1.5}	0.11	s
USED T for strength design	0.11	s
V= $\frac{Cv \cdot I \cdot W}{RT}$ strength design	5.4545	W
drift check	5.4545	W

V_{max}= $\frac{2.5 \cdot Ca \cdot I \cdot W}{R}$ 1.125 W

V_{min}= $\frac{0.8 \cdot Z \cdot Nv \cdot I \cdot W}{R}$ 0.48 W

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

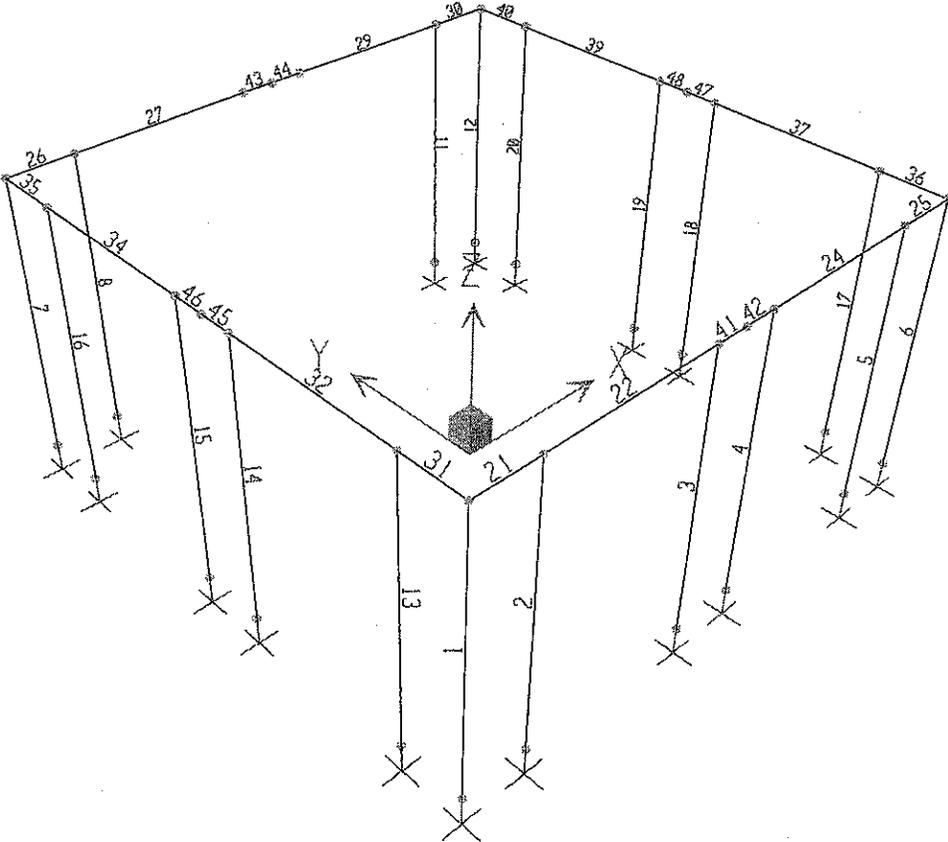
FOR STRENGTH DESIGN

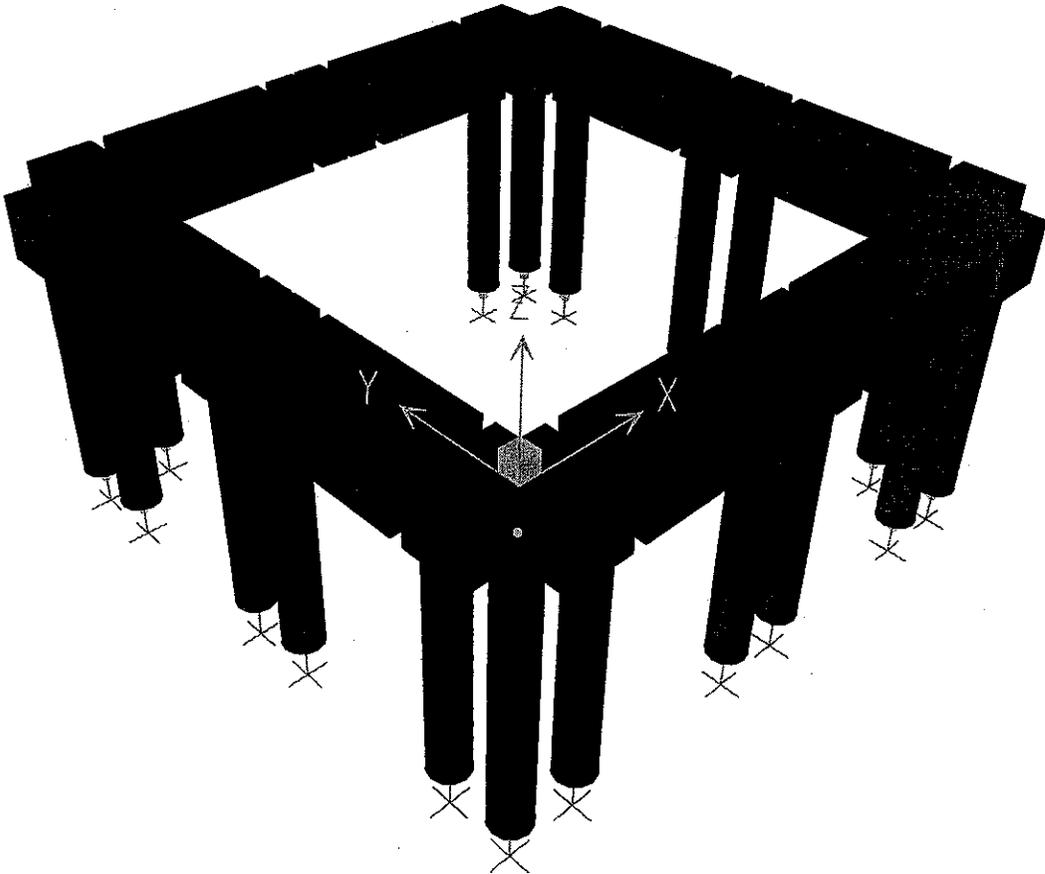
V= 1.125 W = 147.4 kips

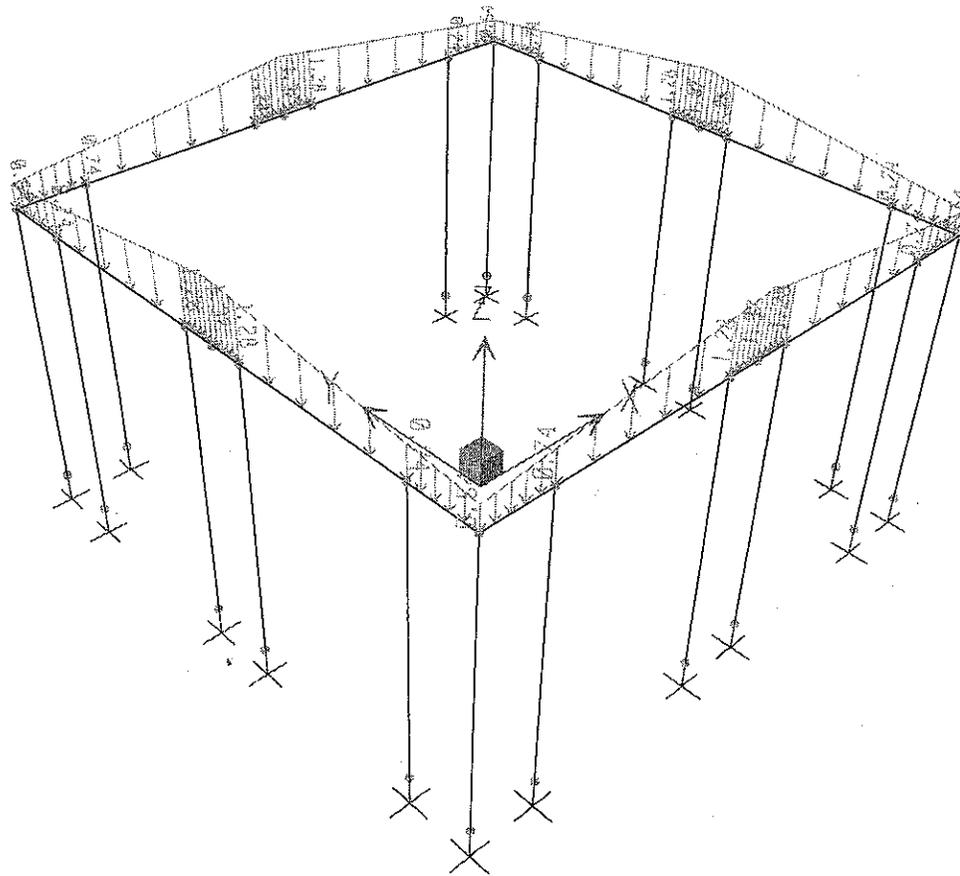
FOR DRIFT CHECKING

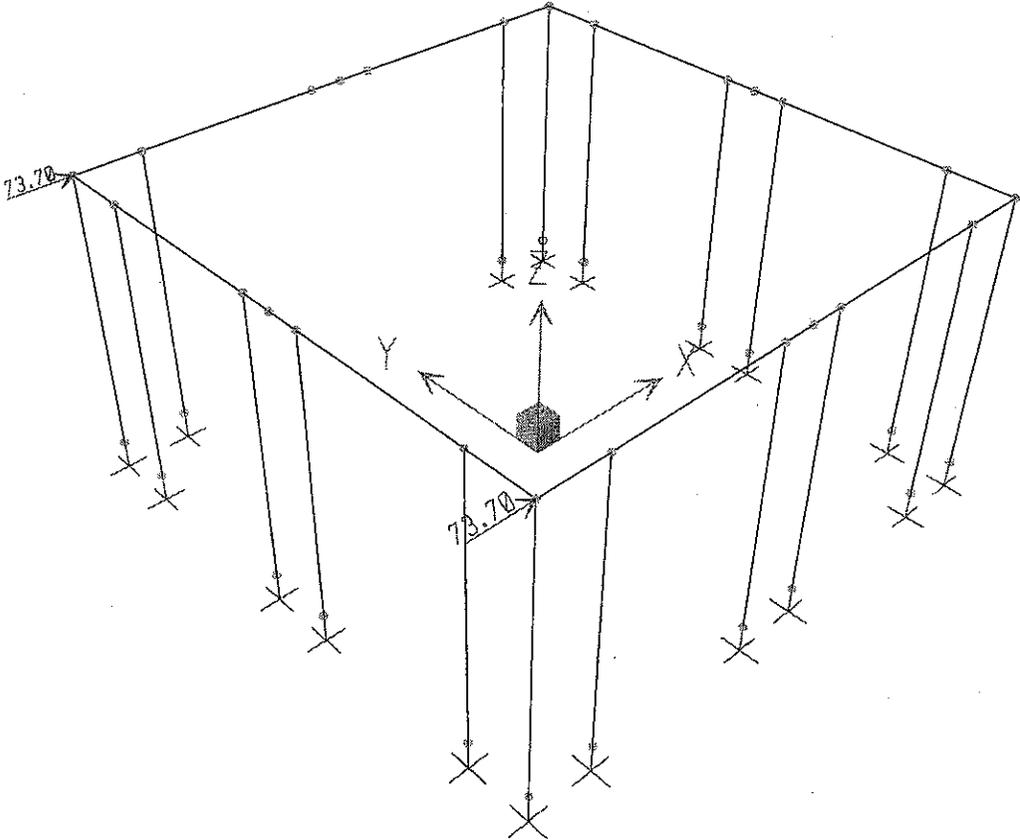
V= 1.125 W = 147.4 kips

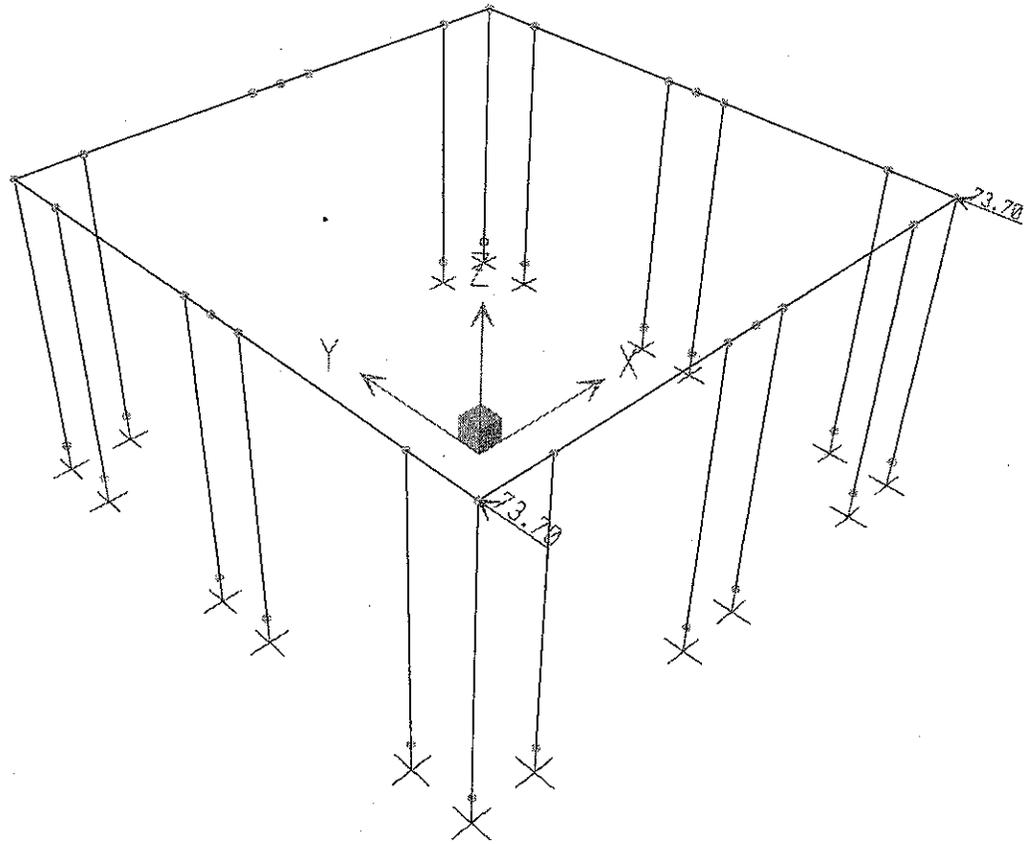
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FACTORED LOAD CASES & COMBINATIONS:

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 $0.5CaID = 0.5 \cdot 0.60 \cdot 1.5 \cdot DL = 0.45DL$ (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).

<u>LC #</u>	<u>Load Case</u>	<u>2001 CBC EQ #/Pg #</u>
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

LOAD COMBINATION MULTIPLIERS

$$0.5Ca I DL = 0.5(0.6)(1.5)(DL) = 0.45DL$$

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

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

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

Max Compression @ Column

Max Shear @ Column

Max Moment @ Column

10.99

46.24

660.82

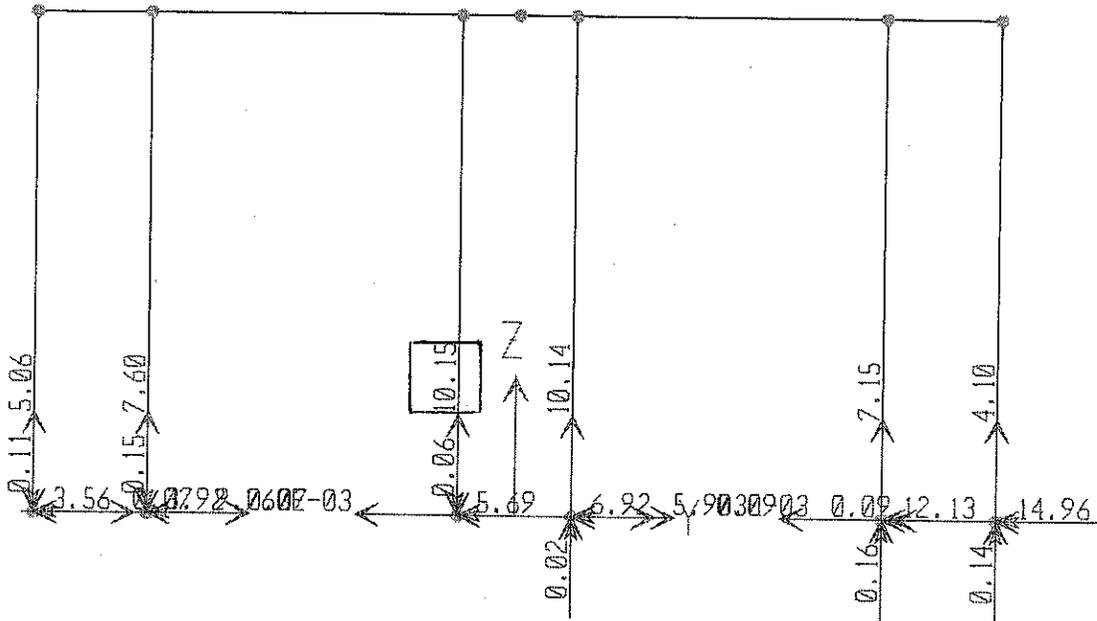
15	Minima	-18.02 COMB7	-7.28 COMB5	-9.37 COMB9	-10.82 COMB11	-562.52 COMB14	-485.69 COMB5
15	Maxima	8.412E-01 EQX	6.94 EQX	9.36 COMB14	10.85 COMB4	563.57 COMB9	472.41 EQX
16	Minima	-22.75 COMB5	-9.14 COMB5	-8.85 COMB17	-12.70 COMB11	-539.85 COMB17	-578.52 COMB5
16	Maxima	9.51 EQX	8.57 EQX	9.03 COMB6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

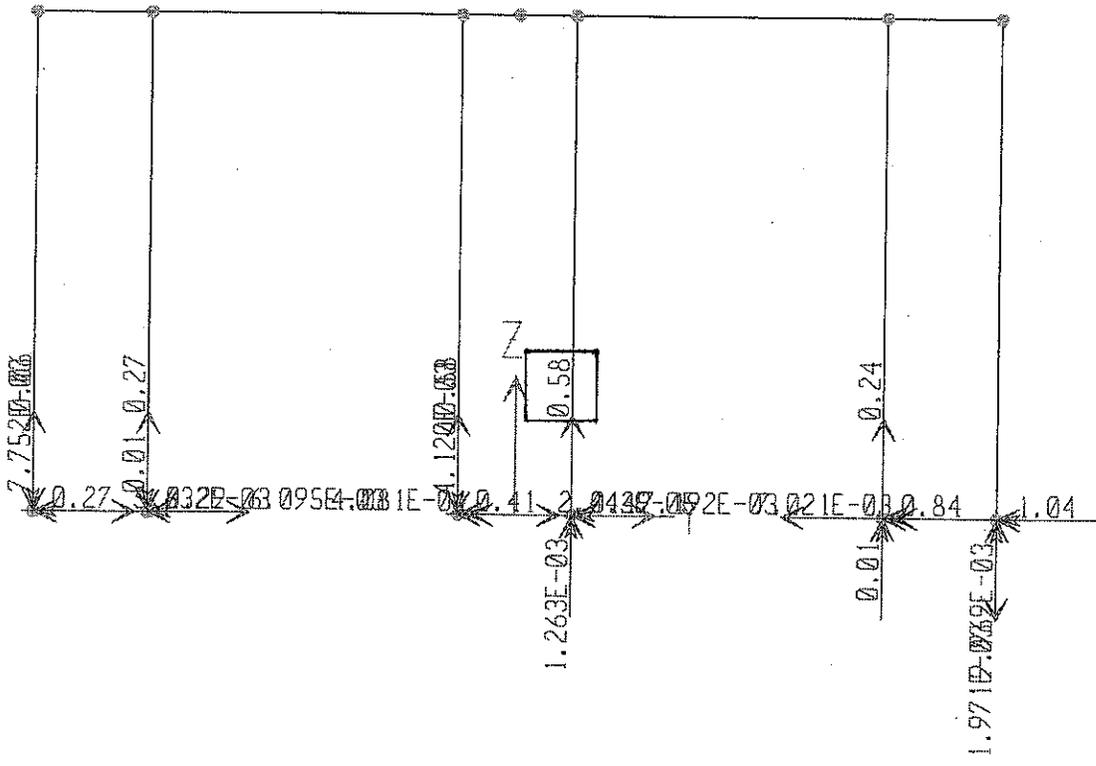
24	Maxima	28.03	26.56	6.97	352.48	483.49	1073.34
		COMB12	COMB3	COMB7	COMB14	COMB17	COMB4
25	Minima	-20.73	-30.11	-14.28	-775.82	-424.59	-1234.25
		COMB3	COMB5	COMB16	COMB9	COMB7	COMB10
25	Maxima	19.88	29.70	14.54	754.43	404.00	1261.31
		COMB12	COMB2	COMB7	COMB14	COMB16	COMB5
26	Minima	-48.75	-29.70	-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

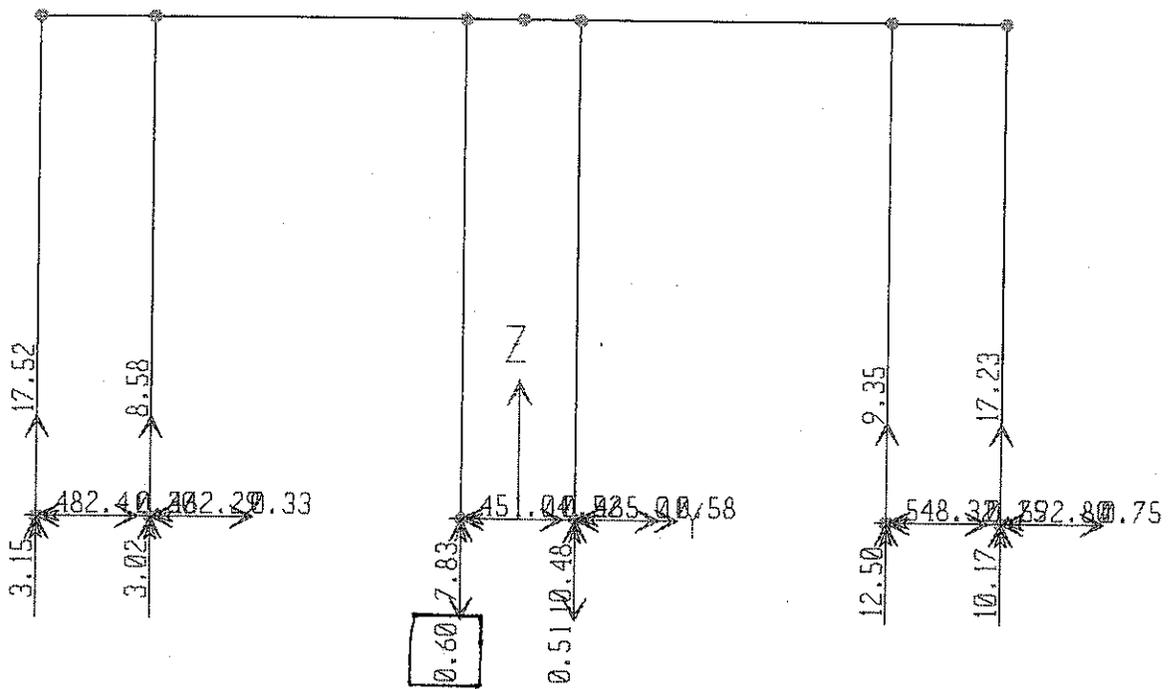
Max. Moment
@ Beam.

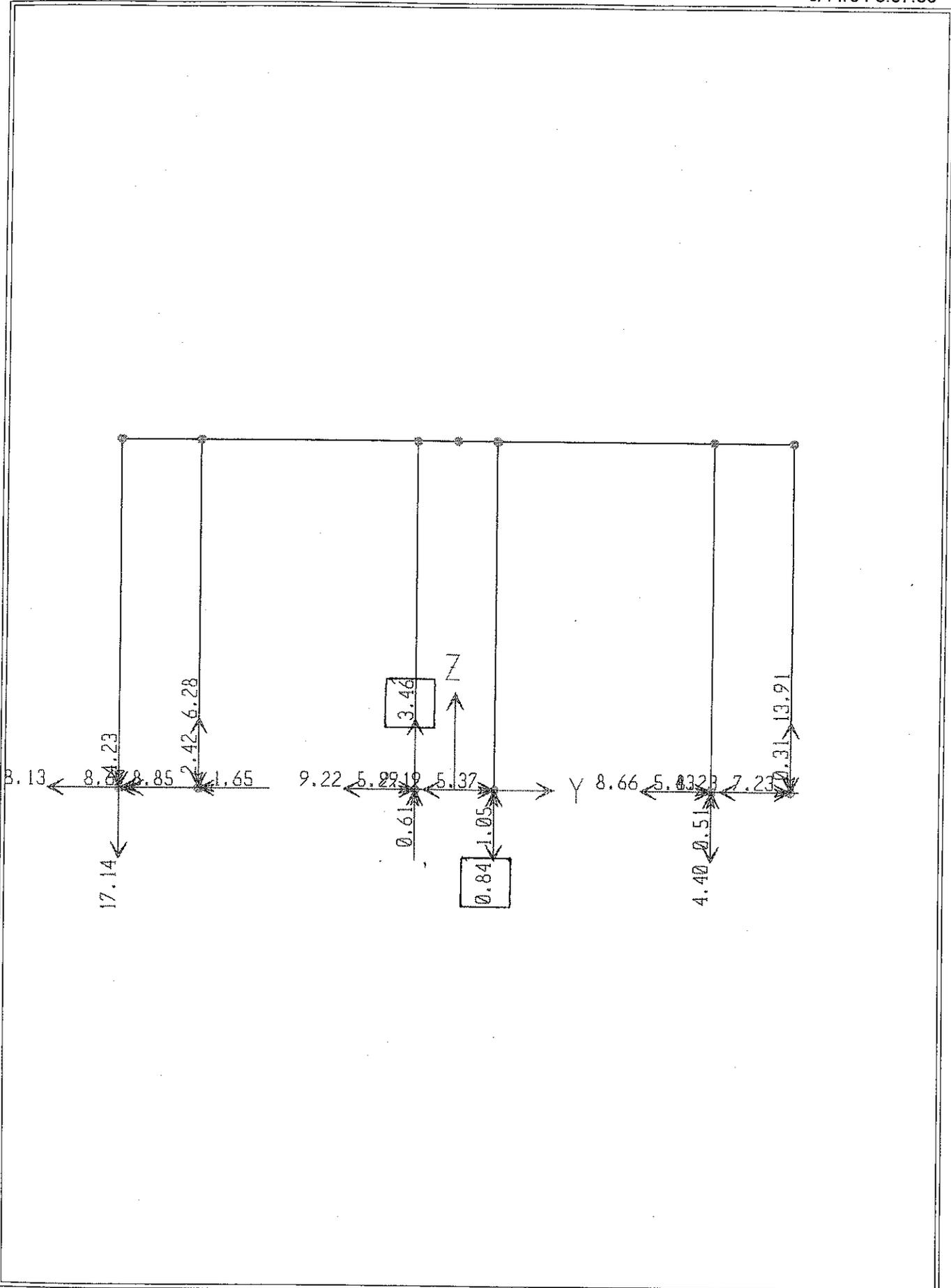
Max. Shear @ Beam.

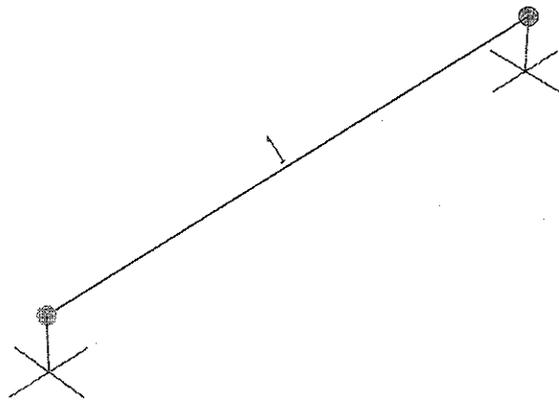
44	Maxima	COMB2 36.55 COMB13	COMB12 17.60 COMB3	COMB2 5.61 COMB13	COMB10 50.54 COMB13	COMB12 94.53 COMB3	EQX 905.18 COMB5
45	Minima	-35.28 COMB8 34.73 COMB15	-19.93 COMB9 15.72 COMB14	-2.88 COMB16 2.97 COMB7	-57.42 EQY 142.78 COMB7	-555.21 COMB4 527.69 COMB11	-278.17 COMB9 204.41 COMB14
46	Minima	-35.28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

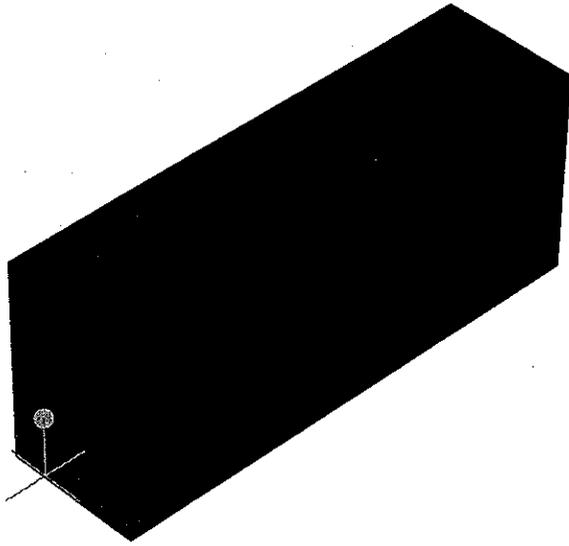


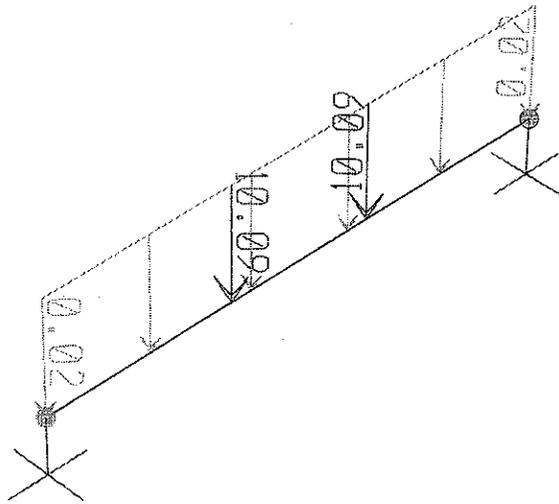


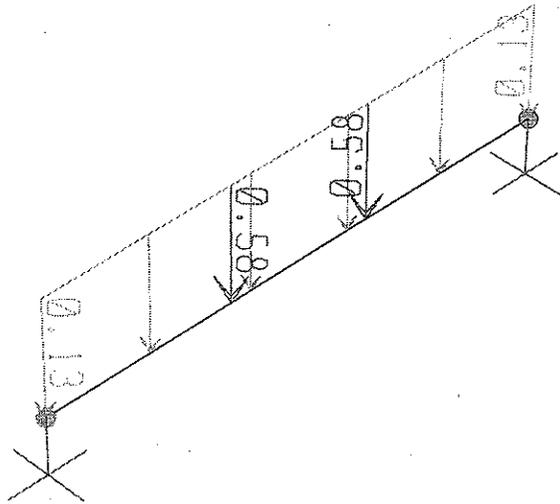


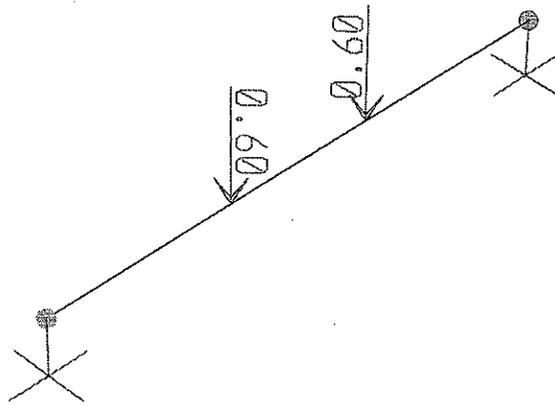


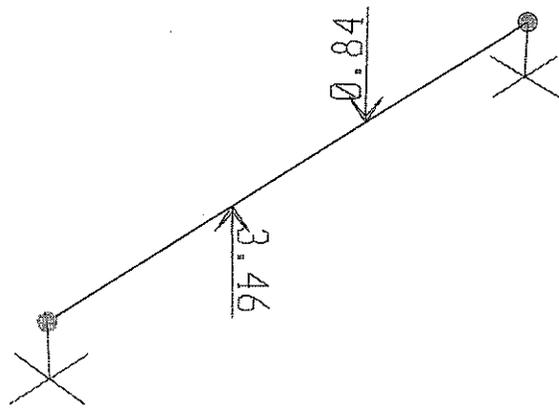












LOAD COMBINATION MULTIPLIERS

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

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

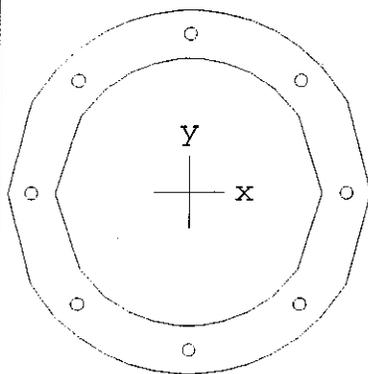
SAP2000 v7.44 File: PAVILIONFLOORBEAM Kip-in Units PAGE 2
6/11/04 8:39:45

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

FRAME	LOAD	LOC	P	V2	V3	T	M2	M3
1	Minima		0.00 COMB17	-25.54 COMB7	0.00 COMB3	0.00 COMB17	0.00 COMB7	-517.91 COMB7
1	Maxima		0.00 COMB17	24.01 COMB3	0.00 COMB7	0.00 COMB17	0.00 COMB7	279.81 COMB7

max Beam Shear.

max. Beam Moment



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 in²

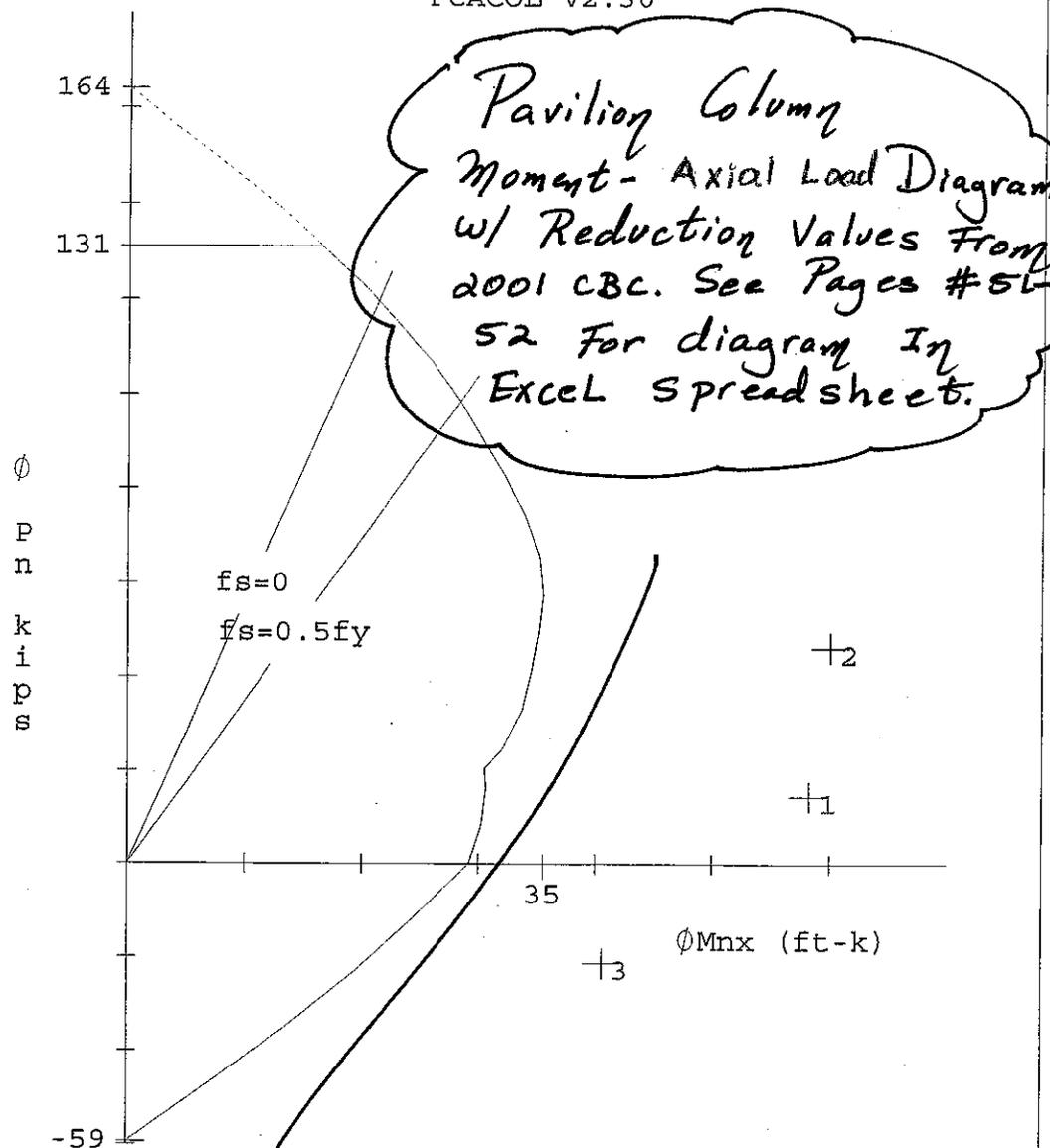
Ix = 1751 in⁴

Iy = 1685 in⁴

Xo = 0.00 in

Yo = 0.00 in

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Licensed To: OLM, 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

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

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) = 0.70, phi(b) = 0.90

```
0000000 00000 00000 00000 00000 00
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=====
Computer program for the Strength Design of Reinforced Concrete Sections
=====

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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 fy = 33 ksi
 Ec = 2850 ksi Es = 29000 ksi
 fc = 2.125 ksi erup = 0 in/in
 eu = 0.003 in/in
 Stress Profile: Block 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 in²
 Ix = 1751.43 in⁴ Xo = 0 in
 Iy = 1684.63 in⁴ 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, $A_s = 2.00 \text{ in}^2$ 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

Load Comb	Pc (kip)	----- Braced (X-axis) -----				----- Sway (X-axis) -----		
		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:

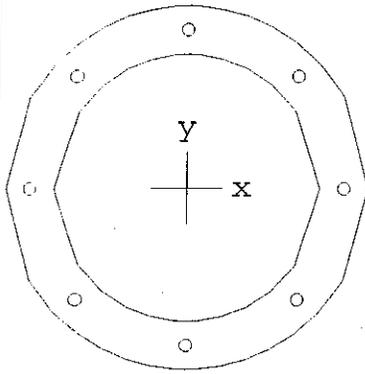
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No.	Load Case	Axial Load (kip)	Moments about X-axis		Moments about Y-axis	
			@ Top (ft-k)	@ Bot (ft-k)	@ Top (ft-k)	@ Bot (ft-k)
1	Dead	0	0	0	0	0
	Live	0	0	0	0	0
	Lat1	14.54	55.1	-54.8	0	0
2	Dead	0	0	0	0	0
	Live	0	0	0	0	0
	Lat1	46.24	44.37	-49.67	0	0
3	Dead	0	0	0	0	0
	Live	0	0	0	0	0
	Lat1	-21.34	36.15	-40.6	0	0

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Pt.	Load Comb	Applied Loads		Computed Strength		Computed/ Applied Ray length
		P (kips)	Mx (ft-k)	P (kips)	Mx (ft-k)	
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!



15.0 x 15.0 inch

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 in²

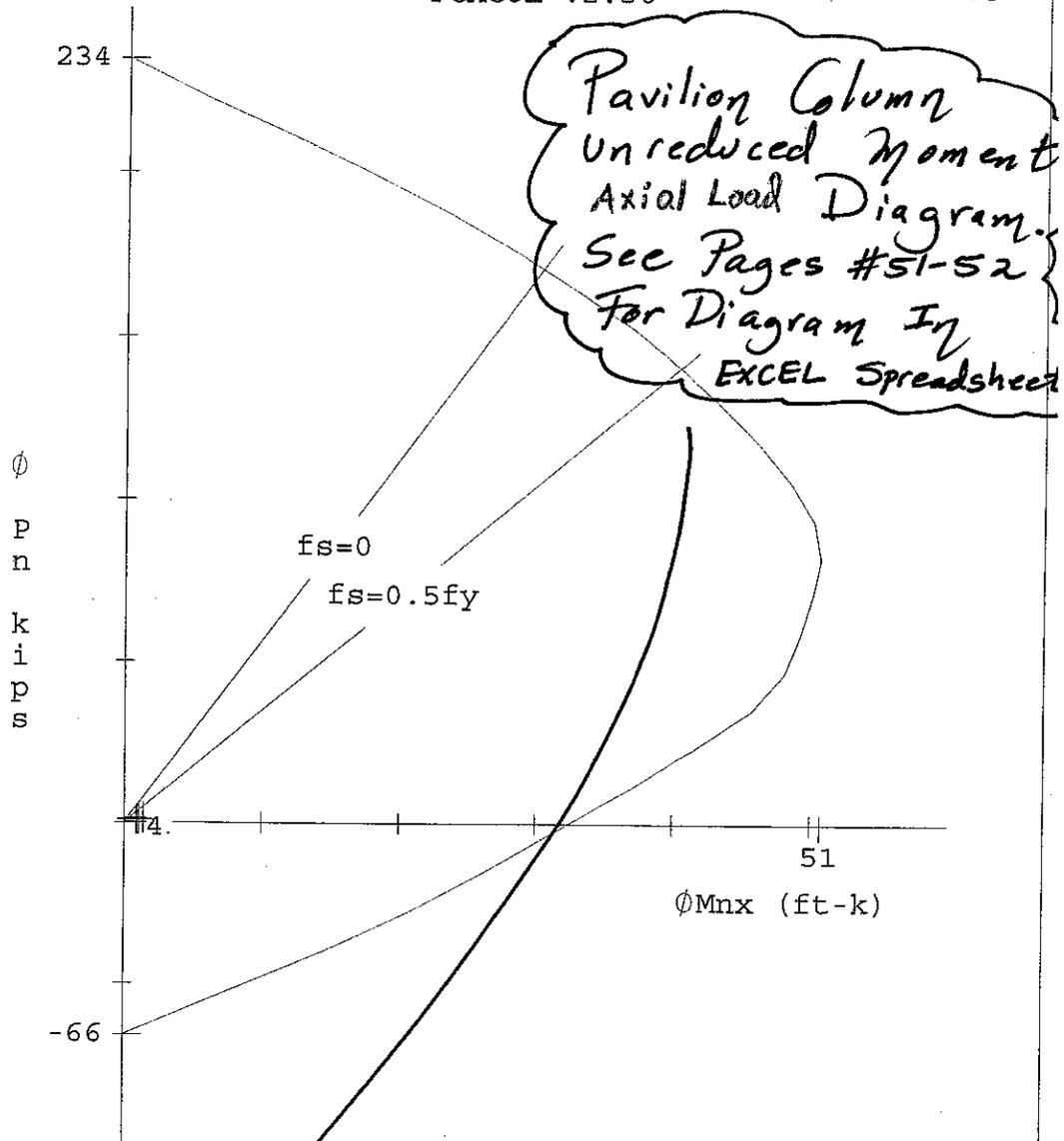
Ix = 1751 in⁴

Iy = 1685 in⁴

Xo = 0.00 in

Yo = 0.00 in

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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

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

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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 fy = 33 ksi
 Ec = 2850 ksi Es = 29000 ksi
 fc = 2.125 ksi erup = 0 in/in
 eu = 0.003 in/in
 Stress Profile: Block 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 in²
 Ix = 1751.43 in⁴ Xo = 0 in
 Iy = 1684.63 in⁴ Yo = 1.99661e-007 in

Reinforcement:

=====

Rebar Database: ASTM

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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, $A_s = 2.00 \text{ in}^2$ 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			

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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

Load Comb	Pc (kip)	----- Braced (X-axis) -----				----- Sway (X-axis) -----		
		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		Moments about Y-axis	
			@ Top (ft-k)	@ Bot (ft-k)	@ Top (ft-k)	@ Bot (ft-k)
1	Dead	1	1	0	0	0
	Live	0	0	0	0	0
	Lat1	0	0	0	0	0

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Pt.	Load Comb	Applied Loads		Computed Strength		Computed/ Applied Ray length
		P (kips)	Mx (ft-k)	P (kips)	Mx (ft-k)	
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!



San Francisco
 (415) 392-3419
 Oakland
 (510) 535-0828

JOB Chabot Tower
 SHEET NO. OF
 CALCULATED BY FC DATE 6/11/2004
 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.

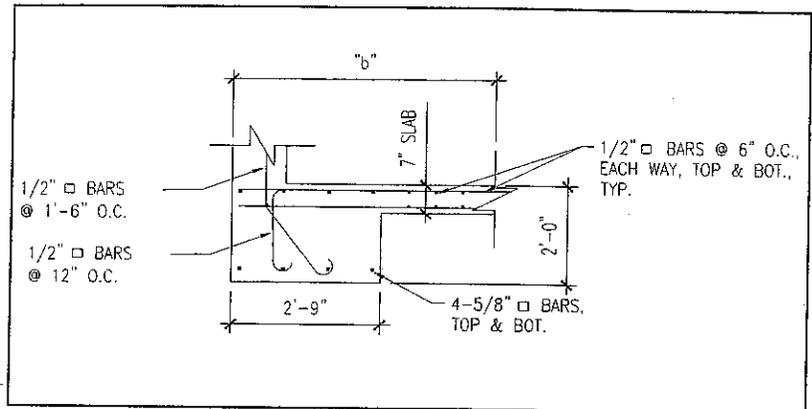


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JOB	Chabot Tower		
SHEET NO.	OF		
CALCULATED BY	FC	DATE	6/11/2004
CHECKED BY		JOB #:	0404

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
 $f_y = 24750$ 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
 $A_{st} = 1.56$ in²
 $A_{sb} = 1.56$ in²

(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 actually is. Because of all these reasons the bending capacity of the members will be reduced by 50%.)

$\phi M_n = (\phi * A_{st} * f_y * (d_t - ((A_{st} * f_y) / (1.7 * f_c * b_t)))) * 0.50 = 385$ k-in = 32 k-ft
 $\mu_u = 126.78$ k-ft
 $\phi = 0.9$ Demand-To-Capacity Ratio(DCR) = $\mu_u / \phi M_n = 3.96$

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

$\phi M_n = (\phi * A_{sb} * f_y * (d_b - ((A_{sb} * f_y) / (1.7 * f_c * b_b)))) * 0.5 = 358$ k-in = 30 k-ft
 $\mu_u = 84.25$ k-ft
 $\phi = 0.9$ Demand-To-Capacity Ratio(DCR) = $\mu_u / \phi M_n = 2.82$

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.)

$\phi V_n = \phi V_c = \phi * (2 * f_c)^{1/2} * b_b * d_b = 51013$ lbs = 51.01 kips
 $\mu_u = 36.39$ k
 $\phi = 0.85$ Demand-To-Capacity Ratio(DCR) = $\mu_u / \phi V_n = 0.71$



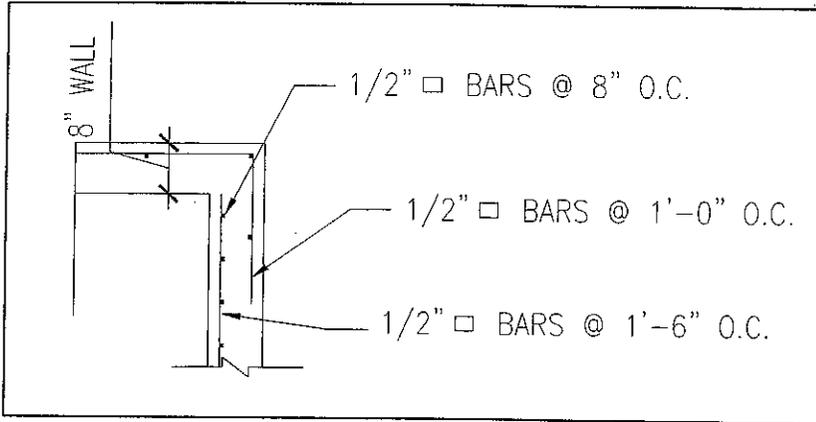
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FILENAME C:\FC-Jobs\0404-Chabot Tower\Excel\RoofStemWall.xls\Stem Wall

Pavilion Roof Stem Wall



Material Properties (Lower Bound):

$f_c = 1875$ psi
 $f_y = 24750$ 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

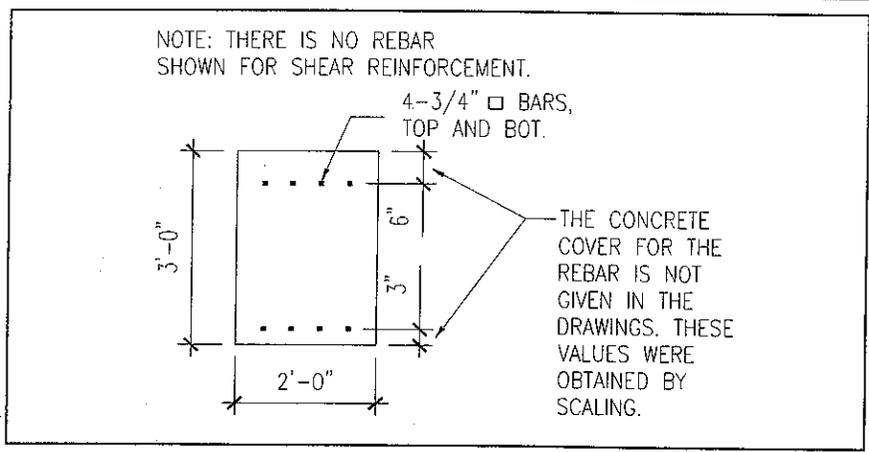
$\phi V_n = \phi V_c = \phi * 2 * (f_c)^{1/2} * b * L = 80090$ lbs = 80.09 kips

$\phi = 0.85$ $V_u = 73.7$ kips

Demand-To-Capacity Ratio (DCR) = $V_u / \phi V_n = 0.92$

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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 in² Asb = 1.77 in²

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 actually is. Because of all these reasons the bending capacity of the members will be reduced by 50%.)

$$\phi M_n = (\phi * A_{st} * f_y * (d_t - ((A_{st} * f_y) / (1.7 * f_c * b)))) * 0.5 = 773 \quad \text{k-in} = 64 \quad \text{k-ft}$$

$$M_u = 43.16 \quad \text{k-ft}$$

$$\phi = 0.9 \quad \text{Demand-To-Capacity Ratio (DCR)} = M_u / \phi M_n = 0.57$$

At Midspan:

$$\phi M_n = (\phi * A_{sb} * f_y * (d_b - ((A_{sb} * f_y) / (1.7 * f_c * b)))) * 0.5 = 852 \quad \text{k-in} = 71 \quad \text{k-ft}$$

$$M_u = 29.06 \quad \text{k-ft}$$

$$\phi = 0.9 \quad \text{Demand-To-Capacity Ratio (DCR)} = M_u / \phi M_n = 0.41$$

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.)

$$\phi V_n = \phi V_c = (\phi * 2 * (f_c)^{1/2} * b * d) = 61200 \quad \text{lbs} = 61.2 \quad \text{kips}$$

$$V_u = 25.54 \quad \text{k}$$

$$\phi = 0.85 \quad \text{Demand-To-Capacity Ratio (DCR)} = V_u / \phi V_n = 0.42$$

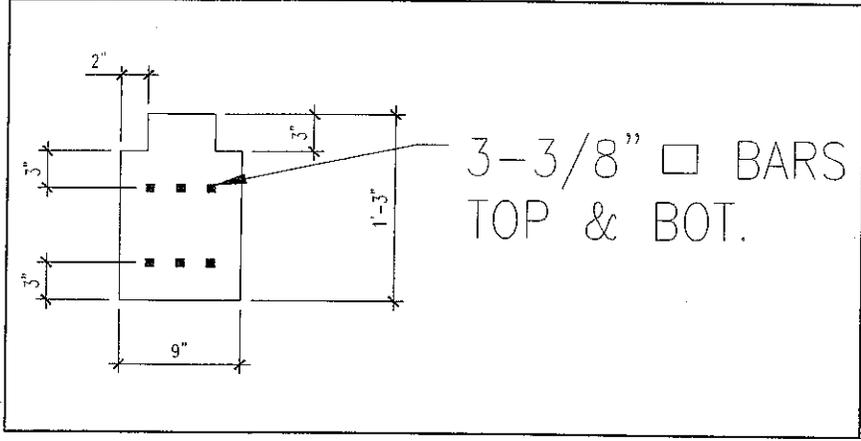


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FILENAME C:\FC-Jobs\0404-Chabot Tower\Excel\InteriorPavilionFloorBeam-1.xls\Rectangular Floor Beam

Pavilion Concrete Beam/Interior Rectangular Floor Beam



Material Properties (Lower Bound):

f_c = 2500 psi
f_y = 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 in²
 $\phi Mn = \phi * Ast * fy * (dt - ((Ast * fy) / (1.7 * fc * b))) = 278$
 $\phi = 0.9$ k-in = 23 k-ft

At Midspan:

Ast = 1.17 in² a = (Ast * fy - 0.85 * fc * 3 * 5) / (0.85 * fc * 9) + 3 = 3.35 in (Depth of conc. block)
 db = 12 in
 Centroid of concrete block Xc = 1.79 in
 $fMn = (\phi * Asb * fy * (db - Xc)) = 355$
 $\phi = 0.9$ k-in = 30 k-ft

Shear Capacity:

Aconc. = 123 in²
 $\phi Vn = \phi Vc = (\phi * 2 * (fc)^{1/2} * Aconc.) = 10455$
 $\phi = 0.85$ lbs = 10.46 kips



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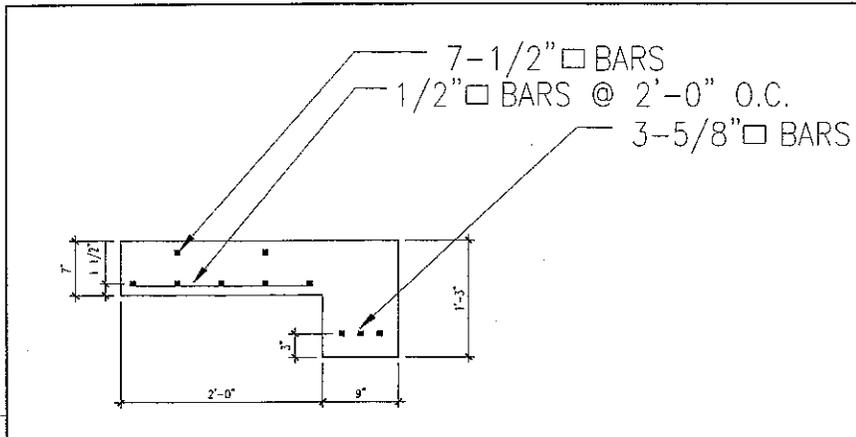
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FILENAME C:\FC-Jobs\0404-Chabot Tower\Excel\InteriorPavilionFloorBeam-2.xls\ "L" Shape Floor Beam

Pavilion Concrete Beam/"L" Shape Floor Beam



Material Properties (Lower Bound):

$f_c = 2500$ psi
 $f_y = 33000$ 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 $A_{st} = 1.75$ in²
 $\phi M_n = \phi * A_{st} * f_y * (dt - ((A_{st} * f_y) / (1.7 * f_c * b))) = 475$ k-in = 40 k-ft
 $\phi = 0.9$

At Midspan:

$b = 9$ in $db = 12$ in $A_{sb} = 1.17$ in²
 $g = ((A_{sb} * f_y) / ((b / 2 * 3) * 0.85 * f_c * 0.5)) = 2.69$ in
 $\phi M_n = \phi * A_{sb} * f_y * (db - g / 3) = 386$ k-in = 32 k-ft
 $\phi = 0.9$

Shear Capacity:

$A_{conc.} = 303$ in²
 $\phi V_n = \phi V_c = (\phi * 2 * (f_c)^{1/2} * A_{conc.}) = 25755$ lbs = 25.76 kips
 $\phi = 0.85$



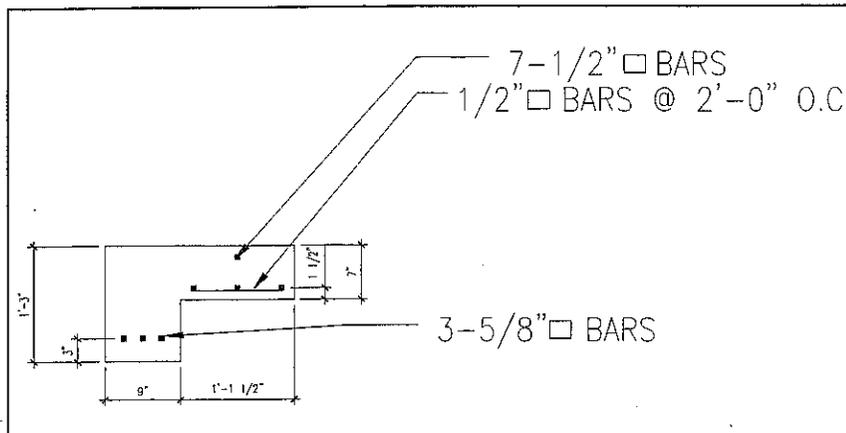
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FILENAME C:\FC-Jobs\0404-Chabot Tower\Excel\Temp\InteriorPavilionFloorBeam-3.xls]L" Shape Floor Beam

Pavilion Concrete Beam/"L" Shape Floor Beam



Material Properties (Lower Bound):

$f'_c = 2500$ psi
 $f_y = 33000$ 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	$A_{st} =$	1	in ²
$\phi Mn = \phi * A_{st} * f_y * (dt - ((A_{st} * f_y) / (1.7 * f'_c * b))) =$				286		$k-in =$	24	k-ft
$\phi = 0.9$								

At Midspan:

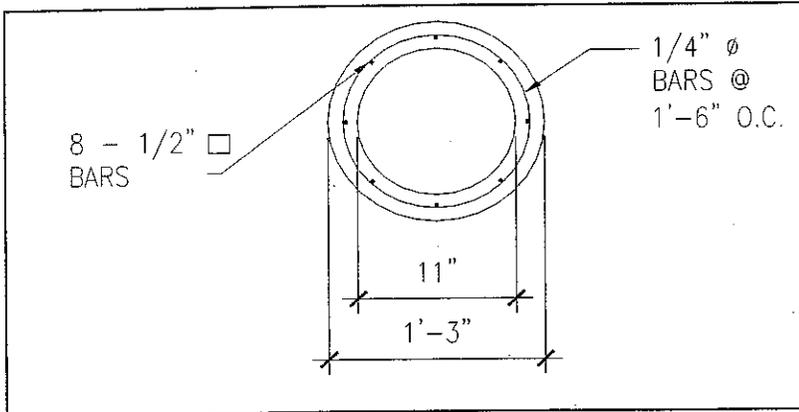
$b =$	9	in	$db =$	12	in	$A_{sb} =$	1.17	in ²
$g = ((A_{sb} * f_y) / ((b / 2 * 3) * 0.85 * f'_c * 0.5)) =$				2.69	in			
$\phi Mn = \phi * A_{sb} * f_y * (db - g / 3) =$				386		$k-in =$	32	k-ft
$\phi = 0.9$								

Shear Capacity:

$A_{conc} =$	229.5	in ²						
$\phi V_n = \phi V_c = (\phi * 2 * (f'_c)^{1/2} * A_{conc}) =$			19507.5	lbs =	19.51	klps		
$\phi = 0.85$								

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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
 $f_y = 33000$ psi
 $E_c = 2850000$ psi
 $E_s = 29000000$ psi

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

Determine if slenderness effects need to be considered:

$L = 10$ ft $K = 2$ $d_o = 15$ in
 $d_i = 11$ in

$A = \Pi(d_o^2 - d_i^2)/4 = 81.68$ in² $I = \Pi(d_o^4 - d_i^4)/64 = 1766.36$ in⁴

$r = (I/A)^{1/2} = 4.65$ 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:

$\phi_b M_n$ (k*ft)	$\phi_c P_n$ (k*ft)	M_n (k*ft)	P_n (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 values w/ Reduction Factors From 2001 CBC. See Sheets 31-37 For PCA Column output.

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

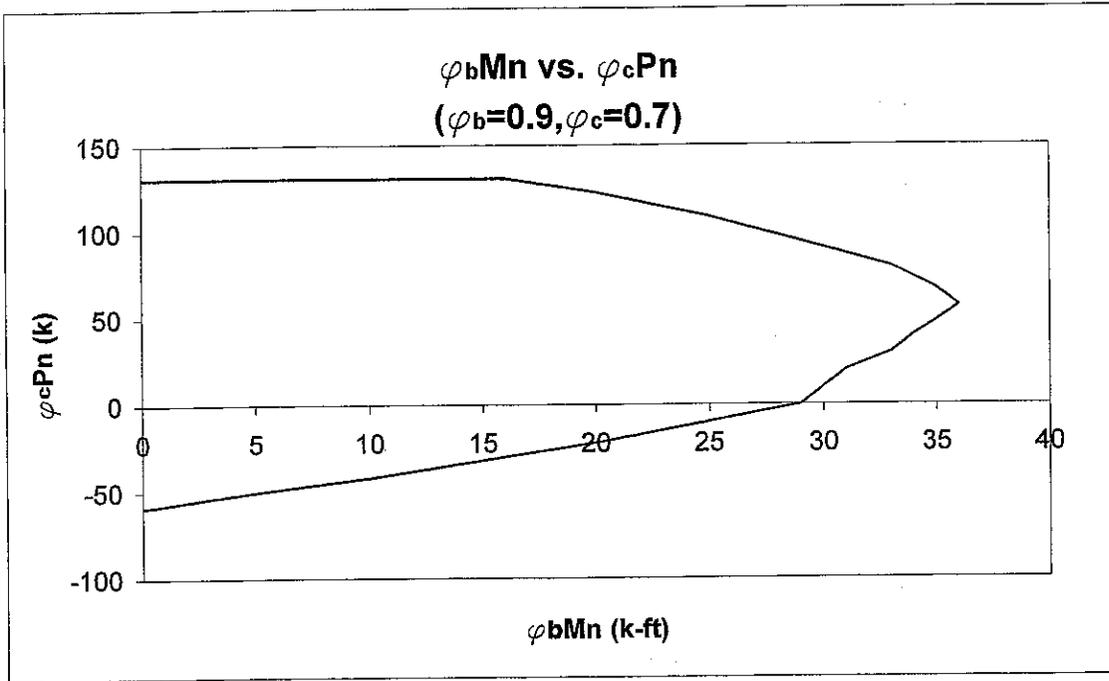
(Note: In order to account for inadequate detailing of the column such as large tie spacing and inadequate development length of the vertical reinf. in the column. The capacities shown will be reduced by 1/3.)

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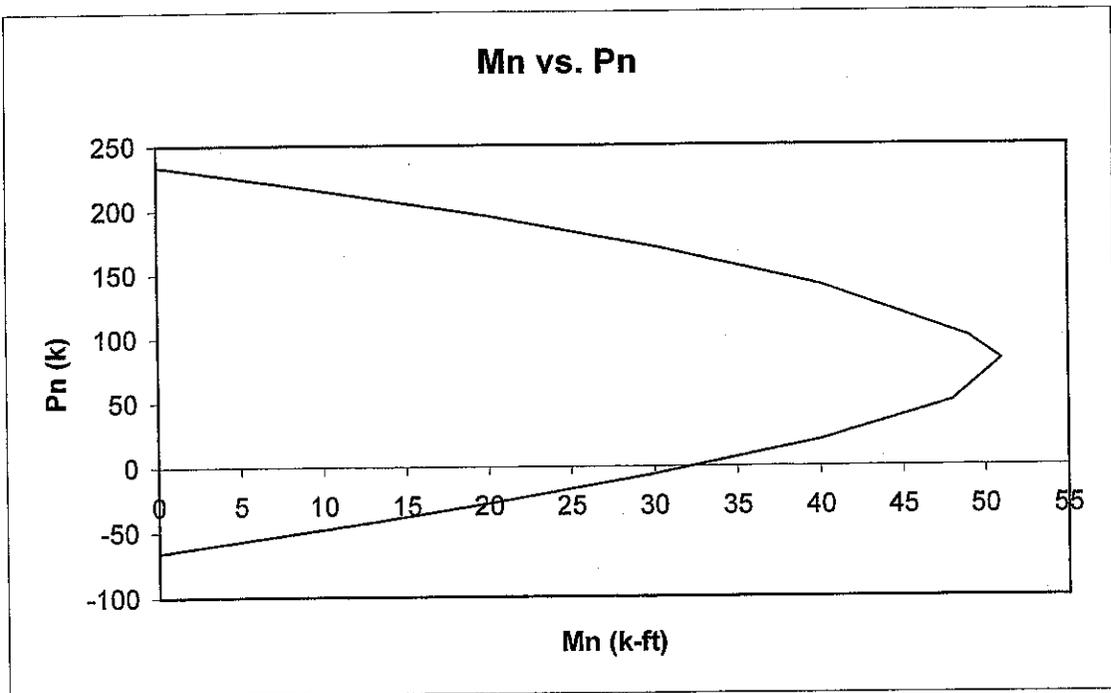
FILENAME C:\FC-Jobs\0404-Chabot Tower\Excel\PavilionColumn.xls\Pavilion Column

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

$P_u = 14.54$ kips $M_u = 54.8$ k-ft
 $\phi P_n = 7$ kips $\phi M_n = 30$ k-ft

Demand-To-Capacity Ratio(DCR) = $P_u/0.667 \cdot P_n + M_u/0.667 \cdot \phi M_n = 5.85$

Case #2 : Maximum Axial Force(Compression)

$P_u = 46.24$ kips $M_u = 49.67$ k-ft
 $\phi P_n = 24$ kips $\phi M_n = 32$ k-ft

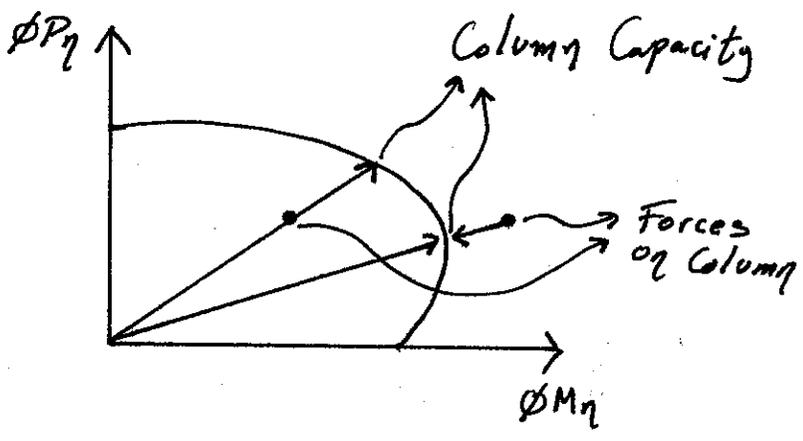
Demand-To-Capacity Ratio(DCR) = $P_u/0.667 \cdot P_n + M_u/0.667 \cdot \phi M_n = 5.22$

Case #3 : Maximum Axial Force(Tension)

$P_u = -21.34$ kips $M_u = 40.6$ k-ft
 $\phi P_n = -12$ kips $\phi M_n = 24$ k-ft

Demand-To-Capacity Ratio(DCR) = $P_u/0.667 \cdot P_n + M_u/0.667 \cdot \phi M_n = 5.20$

Note: The axial and bedding 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.



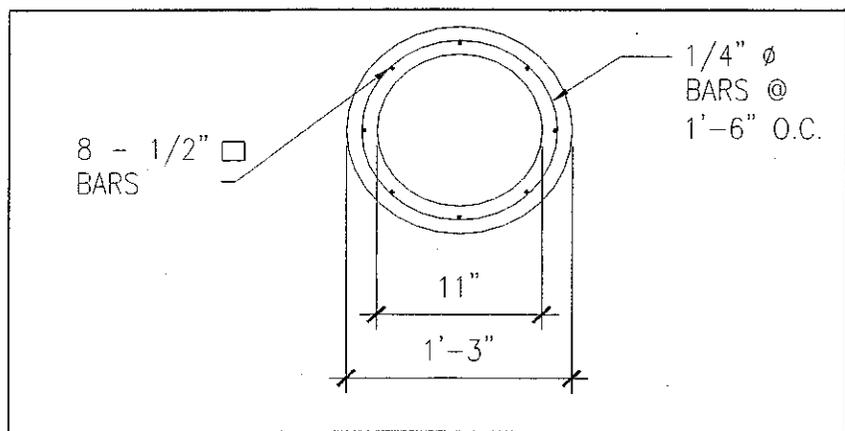


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FILENAME C:\FC-Jobs\0404-Chabot Tower\Excel\PavilionColumn.xls\Pavilion Column

Pavilion Concrete Column/Shear Capacity



Material Properties (Lower Bound):

$f_c = 2500$ psi
 $f_y = 33000$ psi

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

$b = 15$ in $d = 13$ in $A_v = 0.0982$ in²
 $do = 15$ in $di = 11$ in $s = 18$ in
 $A_c = \Pi(do^2 - di^2)/4 = 81.68$ in²

$\phi V_n = \phi V_c + \phi V_s = (0.85 \cdot 2 \cdot (f_c)^{1/2} \cdot A_c + 0.85 \cdot A_v \cdot f_y \cdot d/s)$

$\phi V_n = 8932$ lbs = 8.93 kips $V_u = 10.99$ kips

$\phi = 0.85$ Demand To Capacity Ratio (DCR) = $V_u / \phi V_n = 1.23$

B. INTAKE TOWER



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Weight of Tower For
Seismic Calculations: (Case I)

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

$$\begin{aligned} \text{Weight of Pavilion Columns} &= 18(0.15 \text{ K/ft}^3)(\pi) \left((1.75\text{ft})^2 - (1/12\text{ft})^2 \right) \\ &\quad \times (14) \times (9\text{ft}) \\ &= 42.4 \text{ K} \end{aligned}$$

$$\begin{aligned} \text{Weight of Pavilion Floor} &= 0.15 \text{ K/ft}^3 (3/12\text{ft}) (2(2.83\text{ft}) + \\ &\quad 2.33\text{ft}) (14.71\text{ft}) \\ &+ 0.15 \text{ K/ft}^3 (9/12\text{ft}) (15/12\text{ft}) (2) \\ &\quad \times (14.71\text{ft}) \\ &+ 0.15 \text{ K/ft}^3 (0.375\text{ft}(7/12\text{ft}) + \\ &\quad 8/12\text{ft})(9/12\text{ft}) (2) + (1.875\text{ft})(7/12\text{ft}) \\ &+ 0.15 \text{ K/ft}^3 (3\text{ft})(2\text{ft}) (16.71\text{ft} + \\ &\quad 2.75\text{ft}(2)) \\ &+ 0.15 \text{ K/ft}^3 (4.5\text{ft})(3\text{ft}) (18.33\text{ft}) (2) \\ &= 103.3 \text{ K} \end{aligned}$$

$$\begin{aligned} \text{Weight of Brick} &= 0.12 \text{ K/ft}^3 (1)(1.5\text{ft})(17\text{ft})(12.67\text{ft}) \\ &= 38.77 \text{ K} \end{aligned}$$

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

$$\begin{aligned} \text{Total weight} &= 109.83 \text{ K} + 42.4 \text{ K} + 103.3 \text{ K} + 38.77 \text{ K} + 1,344 \text{ K} \\ &= 1,638.33 \text{ K} \leftarrow \text{"Total weight including Both walls Forming The Tower"} \end{aligned}$$



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Weight of Tower For
Seismic Calculations: (Case II)

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

$$\begin{aligned} \text{Weight of Pavilion Columns} &= 18(0.15 \text{ K/ft}^3)(\pi)((1.75\text{ft})^2 - (1/12\text{ft})^2) \\ &\quad \times (1/4) \times (9\text{ft}) \\ &= 42.4 \text{ K} \end{aligned}$$

$$\begin{aligned} \text{Weight of Pavilion Floor} &= 0.15 \text{ K/ft}^3 (3/12\text{ft})(2(2.83\text{ft}) + \\ &\quad 2.33\text{ft})(14.71\text{ft}) \\ &+ 0.15 \text{ K/ft}^3 (9/12\text{ft})(15/12\text{ft})(2) \\ &\quad \times (14.71\text{ft}) \\ &+ 0.15 \text{ K/ft}^3 (0.375\text{ft})(7/12\text{ft}) + \\ &\quad (8/12\text{ft})(9/12\text{ft})(2) + (1.875\text{ft})(7/12\text{ft}) \\ &+ 0.15 \text{ K/ft}^3 (3\text{ft})(2\text{ft})(16.71\text{ft} + \\ &\quad 2.75\text{ft}(2)) \\ &+ 0.15 \text{ K/ft}^3 (4.5\text{ft})(3\text{ft})(18.33\text{ft})(2) \\ &= 103.3 \text{ K} \end{aligned}$$

$$\begin{aligned} \text{Weight of Brick} &= 0.12 \text{ K/ft}^3 (1)(1.5\text{ft})(17\text{ft})(12.67\text{ft}) \\ &= 38.77 \text{ K} \end{aligned}$$

$$\begin{aligned} \text{Weight of stone} &= 0.16 \text{ K/ft}^3 (1)(27.1\text{ft}) \\ &\quad \times (17\text{ft})(12.67\text{ft}) \\ &= 933.93 \text{ K} \end{aligned}$$

$$\text{Total weight} = 109.83\text{K} + 42.4\text{K} + 103.3\text{K} + 38.77\text{K} + 933.93\text{K}$$

$$= 1,228.23 \text{ K} \leftarrow$$

Total Weight
Including Both walls
Forming The Tower"



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FILENAME C:\FC-Jobs\0404-Chabot Tower\Excel\Ubc97Vb.xls\UBC97-Case 1

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 SOURCE 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		Ω_0	
PERIOD T_B (formula 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	
Nv(Code)	2	Nv(Used)	2	
Ca			0.6	
Cv			0.8	
$T_A = Ct(hn)^{0.7}$			0.35	s
USED T for strength design			0.35	s
$V = \frac{Cv * I * W}{RT}$		strength design	1.7143	W
		drift check	1.7143	W

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

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

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

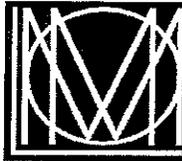
FOR STRENGTH DESIGN

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 1



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ENGINEERS

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

PROJECT Chabot Tower

JOB NO. O404

ENGINEER FC

CHECKED BY OLMM

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			
FAULT TYPE	A		Given Na? No	
NEAR SOURCE DISTANCE(km)	0.5	Km	Given Nv? No	
Z	0.4		If "Yes", Input Data	
Ct	0.02		Na(Given) 1.5	
S	SB		Nv(Given) 2	
R	2		Ωo	
PERIOD T _B (formula 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	
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= $\frac{Cv \cdot I \cdot W}{RT}$		strength design	2.1429	W
		drift check	2.1429	W

V_{max}= $\frac{2.5 \cdot Ca \cdot I \cdot W}{R}$ 1.125 W

V_{min}= $\frac{0.8 \cdot Z \cdot Nv \cdot I \cdot W}{R}$ 0.48 W

V_{min}= $0.11 \cdot Ca \cdot I \cdot W$ 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

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

MATERIAL	Type M	Type S	TYPE M OR TYPE S MORTAR				TYPE N		
	Com-pression ¹	Com-pression ¹	Shear or Tension in Flexure ^{2 3}		Tension in Flexure ⁴		Com-pression ¹	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. 2500-4500 p.s.i. 1500-2500 p.s.i.	250	225	20	10	40	20	200	15	7.5
	175	160	20	10	40	20	140	15	7.5
	125	115	20	10	40	20	100	15	7.5
Solid Concrete Unit Masonry Grade A Grade B	175 125	160 115	12 12	6 6	24 24	12 12	140 100	12 12	6 6
	Grouted Masonry 4500 p.s.i. 2500-4500 p.s.i. 1500-2500 p.s.i.	350	275	25	12.5	50	25		
275		215	25	12.5	50	25			
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 Grade B or 1500-2500 p.s.i. Hollow Units ⁵	140	130	12	6	30	15	110	10	5
	100	90	12	6	30	15	80	10	5
	70	60	12	6	30	15	50	10	5
Stone-Masonry Cast Stone Natural Stone	400	360	8	4	---	---	320	8	4
	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 the TME Report. The Average Values were used for the Report. See Shear and #1.*

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)	GRAouted 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



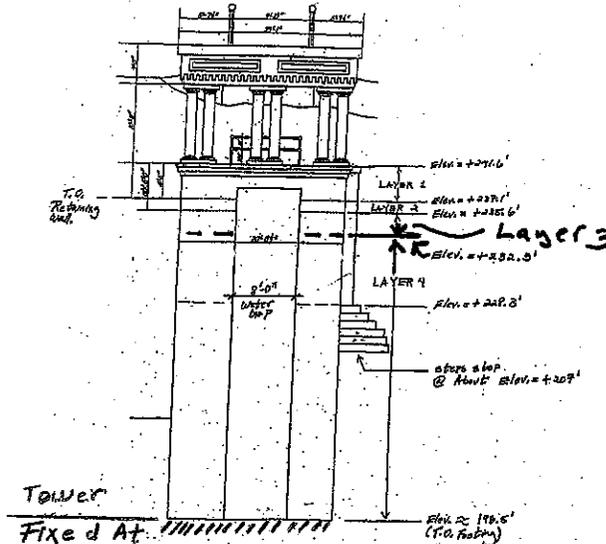
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ENGINEERS

San Francisco
(415) 982-6446
D.B.K.H.B.
(510) 425-0628

JOB	Chabot Tower		
SHEET NO.	OF		
CALCULATED BY	FC	DATE	7/8/2004
CHECKED BY		JOB #:	0404

FILENAME C:\FC-Jobs\0404-Chabot Tower\Excel\Main Tower Forces.xls\Main Tower-Case 1

Main Tower Forces & Stresses - Case 1



Base Shear = V = 1843.1 kips
(Main Tower)

Base

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 For Compressive Stresses	DCR For Tensile Stresses	DCR For Shear Stresses
1	0.02	0.16	0.15
2	0.08	3.07	0.31
3	0.08	7.91	0.70
4	0.07	86.81	1.49

Notes:

- 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.
- The compressive strength values were obtained from the report by Tennebaum-Manehim Engineers dated May 26, 2004.
- The tensile and shear strengths of the concrete were obtained from the 2001 CBC. Tensile Strength = $0.1f_c$ and the Shear Strength = $2\sqrt{f_c}^{1/2}$.
- 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.
- DCR = Demand To Capacity Ratio.

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



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415.682.5449
Oakland
(510) 435.9928

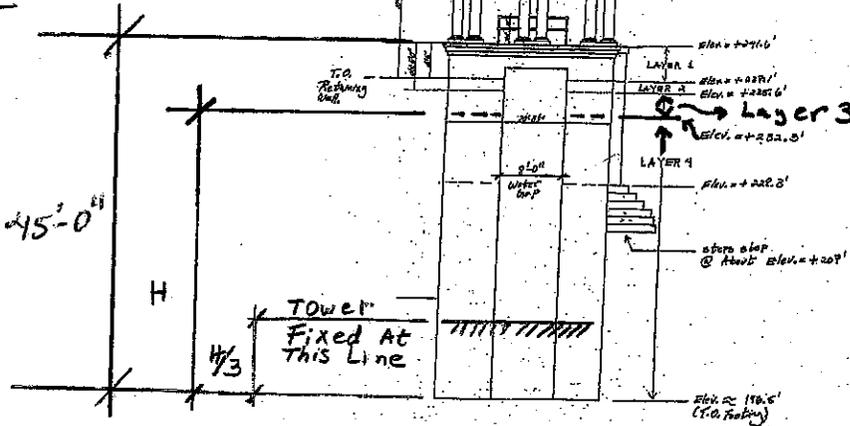
JOB Chabot Tower

SHEET NO.	OF
CALCULATE/FC	DATE 7/8/2004
CHECKED BY	JOB #: 0404

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Main Tower Forces & Stresses - Case II

$H = 35'-9"$
 $45'-0" - \frac{1}{3}(35'-9")$
 $= 33'-11"$



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 Umbraced Length Of The Intake Tower Decreases.

Base Shear = V = 1381.8 kips (Main Tower)

Base Shear = V = 147.4 kips (Pavillon)

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 For Compressive Stresses	DCR For Tensile Stresses	DCR 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	0.24	30.97	1.14

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 = $2 \sqrt{f_c}$.
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 = $P/A - Mc/I$

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**Appendix 3.3: Recent Correspondence between EBMUD and DSOD
Regarding Chabot Dam Seismic Upgrade (8 total)**

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DEPARTMENT OF WATER RESOURCES

1416 NINTH STREET, P.O. BOX 942836
SACRAMENTO, CA 94236-0001
(916) 653-5791

K. Elizabeth
2 Sean/Atta

**JUN 23 2011**

Mr. Xavier Irias, Director of Engineering and Construction
East Bay Municipal Utility District
375 Eleventh Street
Oakland, California 94607-4240

Director of Engineering**JUN 28 2011**

Attention: Mr. Sean Todaro, Associate Engineer
Materials Engineering Section

Chabot Dam, No. 31-5
Alameda County

Dear Mr. Irias:

Your application filed on June 14, 2011, for approval of a geotechnical investigation work plan for the alteration of Chabot Dam, together with the filing fee of \$1,500 based on the estimated project cost of \$50,000, have been received. The proposed work consists of conducting a field exploration program and installing open-standpipe wells in each boring to characterize the geotechnical conditions of the embankment. This information will then be used in the upcoming seismic stability remediation of the dam.

Please review the enclosed "Information Regarding the Consideration of Applications for the Approval of Plans and Specifications for the Construction, Enlargement, Repair, or Alteration of Dams and Reservoirs."

We will notify you by July 14, 2011, as to our progress in reviewing your application, or of additional information which may be required.

If you have any questions, you may contact Design Engineer Richard Armstrong at (916) 227-1300 or Project Engineer Wallace Lam at (916) 227-4626.

Sincerely,

Shan K. Lopez

FOR

David A. Gutierrez, Chief
Division of Safety of Dams

Enclosure

**INFORMATION REGARDING THE CONSIDERATION OF APPLICATIONS FOR
THE APPROVAL OF PLANS AND SPECIFICATIONS FOR THE CONSTRUCTION,
ENLARGEMENT, REPAIR, OR ALTERATION OF DAMS AND RESERVOIRS**

It is required by the California Water Code that this Department approve the application for the alteration of Chabot Dam and Reservoir for safety before construction may commence. Consideration of this application requires a thorough review and independent analysis of the site, plans, specifications, and all other necessary supporting data which form a part of the application.

During the consideration of this application and until it is approved, the Department will be represented by Wallace Lam, Project Engineer, telephone (916) 227-4626, and Design Review Engineer Richard Armstrong, telephone (916) 227-1300. Their office is located at 2200 X Street, Suite 200; the mailing address is Department of Water Resources, Division of Safety of Dams, P.O.Box 942836, Sacramento, CA 94236-0001.

Applications are considered by this Department in the following manner:

1. Upon receipt, the application is administratively checked for validity, correctness of form, and, for construction or enlargement applications, inclusion of the required filing fee. The application is then formally acknowledged and the technical review and analysis commence.
2. Within 30 days of receiving an application, the Department will determine if the application including supporting data is complete and advise the applicant of the finding. The incomplete items will be specified, and the owner will be informed how these items can be completed (Sections 310 and 311, California Code of Regulations, Title 23). One or more site inspections are made by departmental personnel during this period.
3. When all or significant portions of the review and analysis are complete, the applicant is notified of the findings. Supplemental information or supporting data and clarifications may be requested from the applicant at this time.
4. The applicant's civil engineer is requested to make revisions correcting any deficiencies regarding safety and to present revised plans and specifications for further consideration. Plans, specifications, and supporting data are required in duplicate. Usually the applicant and/or civil engineer find it advantageous to meet with departmental representatives in the Division's Sacramento office to discuss the findings and requirements.

All civil engineering work is to bear the seal or stamp of the responsible engineer and shall be signed across the face with the expiration of the certificate shown on or adjacent to the seal. Civil engineering work includes plans, specifications, reports, and documents which are prepared under the Civil Engineering Practice Act.

5. When the Department has determined that the plans and specifications for a new dam construction or the enlargement of an existing dam are satisfactory and that the application can be approved, the applicant's civil engineer is requested to submit two sets of final specifications and three sets of the final plans in the form of blackline ozalid drawings suitable for microfilming and for signature approval. One signed set of plans is returned to the applicant's civil engineer. The process is similar for repair and alteration approvals except two sets of drawings are submitted and no drawings are returned. All drawings must be signed and stamped in accordance with Number 4 above. The approved original application is returned to the owner.
6. Construction must not commence until the applicant has obtained written approval from the Department (Sections 6200 and 6225, Water Code).

DEPARTMENT OF WATER RESOURCES

1416 NINTH STREET, P.O. BOX 942836
SACRAMENTO, CA 94236-0001
(916) 653-5791

Elizabeth Sialek
Sean Todaro



Director of Engineering

JUL 15 2011

RECEIVED

JUL 18 REC'D

**ENGINEERING SERVICES
DIVISION**

JUL 13 2011

Mr. Xavier Irias, Director
Engineering and Construction
East Bay Municipal Utility District
375 Eleventh Street
Oakland, California 94607-4240

Attention: Mr. Sean Todaro, Associate Engineer
Materials Engineering Section

Chabot Dam, No. 31-5
Alameda County

Dear Mr. Irias:

Enclosed is your approved Application No. 31-5, which was filed on June 14, 2011, for approval of plans and specifications for the alteration of Chabot Dam.

Please review the enclosed "Information Regarding Supervision of Construction of Dams and Reservoirs" and provide our Regional Engineer with the required timely notice of start of construction.

Sincerely,

David A. Gutierrez, Chief
Division of Safety of Dams

Enclosures
Certified Mail

**INFORMATION REGARDING SUPERVISION
OF CONSTRUCTION OF DAMS AND RESERVOIRS**

It is required by the California Water Code that this Department supervise the _____ alteration of _____ Chabot _____ Dam and Reservoir for safety. It is also required that the construction work on the dam and reservoir be under the responsible charge of a registered civil engineer representing the owner.

In general, the Department supervises the work by reviewing and approving procedures, decisions, and portions of the work that have been approved by the registered engineer or by his/her authorized representative.

During the work on this dam and reservoir, the Department will be represented by _____ Y-Nhi Enzler _____, Regional Engineer, telephone (916) _____ 227-4604 _____ and _____ Russell Bowlus _____, assigned Field Engineer, telephone (916) _____ 227-4627 _____. Their office is located at 2200 X Street, Suite 200; the mailing address is Department of Water Resources, Division of Safety of Dams, P. O. Box 942836, Sacramento, CA 94236-0001.

The Regional Engineer should be contacted by you or your representative several weeks prior to the start of construction to determine the appropriateness of a pre-construction conference and the schedule for such a meeting if it is deemed necessary.

Supervision of construction by this Department is usually conducted in the following manner:

1. The owner or his/her representative notifies the Regional Engineer approximately one week prior to the start of construction to arrange for the initial construction inspection.
2. Inspections of construction are made by the Field Engineer as deemed necessary to ensure compliance with approved plans and specifications and to ensure that developments which might prove unsafe are corrected.
3. The foundation or abutments are not covered until the Field Engineer has inspected and approved them. This includes the foundations for cutoff trenches, outlet conduit, spillway channel, etc., which will be covered by concrete and/or other materials of the dam.
4. As construction progresses, any changes in work and/or the plans and specifications found necessary must be given prior approval by the Department. Request for such changes are submitted in writing to the Department of Water Resources, Division of Safety of Dams, P. O. Box 942836, Sacramento, CA 94236-0001.
5. Progress of work and/or construction procedures which might result in an unsafe condition will not be permitted; for example, unsatisfactory progress which might result in an unsafe dam in flood season or placement of earth embankment without adequate moisture conditioning of the soil.
6. The Field Engineer will make every attempt to schedule his/her inspections so as to avoid delay in construction. Your cooperation is needed in giving prior notification as far in advance as possible of the time when an inspection will be required.

Owner Copy

APPLICATION FOR APPROVAL OF PLANS AND SPECIFICATIONS FOR THE
REPAIR OR ALTERATION OF A DAM AND RESERVOIR

Original signature required. Please mail
hard copy with signatures to DSOD.

DSOD Office Use Only	
Dam Number	Application Filed
31-5	June 14, 2011
Applicant must NOT fill in the above blanks.	

DEPT. OF WATER RESOURCES
DIV. SAFETY OF DAMS
2011 JUN 14 PM 2:14

For full information concerning the filling out and filing of this form, send for "Statutes and Regulations Pertaining to Supervision of Dams and Reservoirs." This is not an application to appropriate water. To secure the right to appropriate water, application should be made to the State Water Resources Control Board on forms the Board will provide upon request.

I, Elizabeth Z. Bialek of 375 Eleventh Street, Oakland
Name of individual signing application Address

County of Alameda, State of California, hereby make application for the
approval of plans and specifications for the repair alteration of Chabot
Name of dam and reservoir
dam and reservoir.

The owner of the dam and reservoir is East Bay Municipal Utility District
Name of owner
of 375 Eleventh Street, Oakland County of Alameda, State of California
Address

If the owner is a corporation, give name and address of president and secretary:

The applicant is acting for the owner in the legal capacity of _____
Agent, Lessee, Trustee, Engineer, etc.

Location of Dam

1. The dam is in Alameda County, in the NE ¼, Sec. 30, Tp. 2S, R. 2W, M.D. B&M
and is located on San Leandro Creek, tributary to San Leandro Bay
Creek, river or watershed Creek or river

Description of Proposed Work

2. Type of dam Earth
Concrete arch or gravity, earth, rockfill, etc.

3. Description of work contemplated (Use extra sheets or exhibits if necessary.)
Installation of five borings and associated open-standpipe wells, including the construction of an access road to one of the borings.

4. Work will result in: No change in the maximum storage level.
"No change in" or "Lowering"

This form is not to be used if the alteration will increase the water storage elevation of the reservoir as previously operated.

5. Estimated cost of work \$50,000.00

6. Work is to commence by June 15, 2011, and to be completed by August 31, 2011

7. Engineer A.B. Yiadom 8. Contractor To be decided

9. The maps, plans and specifications, and filing fee of \$1,500.00 accompanying this application are a part thereof.

Signed: Elizabeth Z. Bialek
Applicant - sign all copies

this 02 day of June, 2011

APPLICATION FOR APPROVAL OF PLANS AND SPECIFICATIONS FOR THE
REPAIR OR ALTERATION OF A DAM AND RESERVOIR
Chabot

APPROVAL OF APPLICATION

filed June 14, 2011 including the plans and specifications.

This is to certify that the application, including plans and specifications for the repair alteration of
Chabot dam and reservoir, No. 31-5, has been examined and
the same is hereby _____ approved, subject to the following terms and limitations:

1. Construction work shall be commenced on or before _____.
2. Construction work shall be completed on or before _____.



Witness my hand and the seal of the Department of
Water Resources of the State of California

this 13 day of July, 2011



Chief, Division of Safety of Dams

Registered Civil Engineer No. 40734

DEPARTMENT OF WATER RESOURCES

1416 NINTH STREET, P.O. BOX 942836
SACRAMENTO, CA 94236-0001
(916) 653-5791

Elizabeth Sialek
Sean Todaro



Director of Engineering

JUL 15 2011

RECEIVED

JUL 18 REC'D

**ENGINEERING SERVICES
DIVISION**

JUL 13 2011

Mr. Xavier Irias, Director
Engineering and Construction
East Bay Municipal Utility District
375 Eleventh Street
Oakland, California 94607-4240

Attention: Mr. Sean Todaro, Associate Engineer
Materials Engineering Section

Chabot Dam, No. 31-5
Alameda County

Dear Mr. Irias:

Enclosed is your approved Application No. 31-5, which was filed on June 14, 2011, for approval of plans and specifications for the alteration of Chabot Dam.

Please review the enclosed "Information Regarding Supervision of Construction of Dams and Reservoirs" and provide our Regional Engineer with the required timely notice of start of construction.

Sincerely,

David A. Gutierrez, Chief
Division of Safety of Dams

Enclosures
Certified Mail

**INFORMATION REGARDING SUPERVISION
OF CONSTRUCTION OF DAMS AND RESERVOIRS**

It is required by the California Water Code that this Department supervise the _____ alteration of _____ Chabot _____ Dam and Reservoir for safety. It is also required that the construction work on the dam and reservoir be under the responsible charge of a registered civil engineer representing the owner.

In general, the Department supervises the work by reviewing and approving procedures, decisions, and portions of the work that have been approved by the registered engineer or by his/her authorized representative.

During the work on this dam and reservoir, the Department will be represented by _____ Y-Nhi Enzler _____, Regional Engineer, telephone (916) _____ 227-4604 _____ and _____ Russell Bowlus _____, assigned Field Engineer, telephone (916) _____ 227-4627 _____. Their office is located at 2200 X Street, Suite 200; the mailing address is Department of Water Resources, Division of Safety of Dams, P. O. Box 942836, Sacramento, CA 94236-0001.

The Regional Engineer should be contacted by you or your representative several weeks prior to the start of construction to determine the appropriateness of a pre-construction conference and the schedule for such a meeting if it is deemed necessary.

Supervision of construction by this Department is usually conducted in the following manner:

1. The owner or his/her representative notifies the Regional Engineer approximately one week prior to the start of construction to arrange for the initial construction inspection.
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4. As construction progresses, any changes in work and/or the plans and specifications found necessary must be given prior approval by the Department. Request for such changes are submitted in writing to the Department of Water Resources, Division of Safety of Dams, P. O. Box 942836, Sacramento, CA 94236-0001.
5. Progress of work and/or construction procedures which might result in an unsafe condition will not be permitted; for example, unsatisfactory progress which might result in an unsafe dam in flood season or placement of earth embankment without adequate moisture conditioning of the soil.
6. The Field Engineer will make every attempt to schedule his/her inspections so as to avoid delay in construction. Your cooperation is needed in giving prior notification as far in advance as possible of the time when an inspection will be required.

Owner Copy

APPLICATION FOR APPROVAL OF PLANS AND SPECIFICATIONS FOR THE
REPAIR OR ALTERATION OF A DAM AND RESERVOIR

Original signature required. Please mail
hard copy with signatures to DSOD.

DSOD Office Use Only	
Dam Number	Application Filed
31-5	June 14, 2011
Applicant must NOT fill in the above blanks.	

DEPT. OF WATER RESOURCES
DIV. SAFETY OF DAMS
2011 JUN 14 PM 2:14

For full information concerning the filling out and filing of this form, send for "Statutes and Regulations Pertaining to Supervision of Dams and Reservoirs." This is not an application to appropriate water. To secure the right to appropriate water, application should be made to the State Water Resources Control Board on forms the Board will provide upon request.

I, Elizabeth Z. Bialek of 375 Eleventh Street, Oakland
Name of individual signing application Address

County of Alameda, State of California, hereby make application for the
approval of plans and specifications for the repair alteration of Chabot
Name of dam and reservoir
dam and reservoir.

The owner of the dam and reservoir is East Bay Municipal Utility District
Name of owner
of 375 Eleventh Street, Oakland County of Alameda, State of California
Address

If the owner is a corporation, give name and address of president and secretary:

The applicant is acting for the owner in the legal capacity of _____
Agent, Lessee, Trustee, Engineer, etc.

Location of Dam

1. The dam is in Alameda County, in the NE ¼, Sec. 30, Tp. 2S, R. 2W, M.D. B&M
and is located on San Leandro Creek, tributary to San Leandro Bay
Creek, river or watershed Creek or river

Description of Proposed Work

2. Type of dam Earth
Concrete arch or gravity, earth, rockfill, etc.

3. Description of work contemplated (Use extra sheets or exhibits if necessary.)

Installation of five borings and associated open-standpipe wells, including the construction of an access road to one of the borings.

4. Work will result in: No change in the maximum storage level.
"No change in" or "Lowering"

This form is not to be used if the alteration will increase the water storage elevation of the reservoir as previously operated.

5. Estimated cost of work \$50,000.00

6. Work is to commence by June 15, 2011, and to be completed by August 31, 2011

7. Engineer A.B. Yiadom 8. Contractor To be decided

9. The maps, plans and specifications, and filing fee of \$1,500.00 accompanying this application are a part thereof.

Signed: Elizabeth Z. Bialek
Applicant - sign all copies

this 02 day of June, 2011

APPLICATION FOR APPROVAL OF PLANS AND SPECIFICATIONS FOR THE
REPAIR OR ALTERATION OF A DAM AND RESERVOIR
Chabot

APPROVAL OF APPLICATION

filed June 14, 2011 including the plans and specifications.

This is to certify that the application, including plans and specifications for the repair alteration of
Chabot dam and reservoir, No. 31-5, has been examined and
the same is hereby _____ approved, subject to the following terms and limitations:

1. Construction work shall be commenced on or before _____.
2. Construction work shall be completed on or before _____.



Witness my hand and the seal of the Department of
Water Resources of the State of California

this 13 day of July, 2011


Chief, Division of Safety of Dams

Registered Civil Engineer No. 40734

DEPARTMENT OF WATER RESOURCES

1416 NINTH STREET, P.O. BOX 942836
SACRAMENTO, CA 94236-0001
(916) 653-5791



FEB 16 2012

Mr. Xavier Irias, Director
Engineering and Construction
East Bay Municipal Utility District
Post Office Box 24055
Oakland, California 94623-1055

Attention: Ms. Elizabeth Z. Bialek, Manager
Engineering Services

Chabot Dam, No. 31-5
Alameda County

Dear Mr. Irias:

On December 20, 2011, Area Engineer Russell Bowlus inspected Chabot Dam and verified completion of all work associated with the alteration application filed on June 14, 2011, and approved on July 13, 2011. The alteration consisted of five sonic borings for geotechnical investigation.

We have reviewed the report and logs submitted by the District for this work. Based on the information provided, we agree that the alterations were satisfactorily completed.

To complete the administrative requirements associated with the alteration application, please submit the following items:

1. A cost affidavit. Two affidavit forms are enclosed.
2. A breakdown of construction costs, including engineering.

If you have any questions or need additional information, you may contact Area Engineer Russell Bowlus at (916) 227-4627 or Regional Engineer Y-Nhi D. Enzler at (916) 227-4604.

Sincerely,

Michael Wagner For

David A. Gutierrez, Chief
Division of Safety of Dams

DEPARTMENT OF WATER RESOURCES

1416 NINTH STREET, P.O. BOX 942836
SACRAMENTO, CA 94236-0001
(916) 653-5791



APR 26 2012

Mr. Xavier Irias, Director
Engineering and Construction
East Bay Municipal Utility District
Post Office Box 24055
Oakland, California 94623-1055

Attention: Ms. Elizabeth Z. Bialek, Manager
Engineering Services

Chabot Dam, No. 31-5
Alameda County

Dear Mr. Irias:

This is in reply to Mr. Sean Todaro's March 2, 2012, letter transmitting an affidavit of actual costs, a breakdown of costs, and a further fee check in the amount of \$980.00 for the alteration of Chabot Dam. The alteration consisted of five sonic borings for geotechnical investigation.

We have reviewed the cost information and accept \$82,677 as the total actual cost of the alteration. This completes the administrative requirements associated with the alteration application filed on June 14, 2011, and approved on July 13, 2011.

If you have any questions or need additional information, you may contact Area Engineer Russell Bowlus at (916) 227-4627 or Regional Engineer Y-Nhi D. Enzler at (916) 227-4604.

Sincerely,

Michael Waggoner
FW

David A. Gutierrez, Chief
Division of Safety of Dams

DEPARTMENT OF WATER RESOURCES

1416 NINTH STREET, P.O. BOX 942836
SACRAMENTO, CA 94236-0001
(916) 653-5791

→ ATTA



JUN - 7 2012

Ms. Elizabeth Z. Bialek, Manager
Engineering Services
East Bay Municipal Utility District
Post Office Box 24055
Oakland, California 94623

Attention: Mr. Sean Todaro, Associate Engineer
Materials Engineering Section

Chabot Dam, No. 31-5
Alameda County

Dear Ms. Bialek:

This is in reply to your letter dated November 30, 2011, presenting the latest interpretation of the material boundaries within Chabot Dam and additional slope stability analyses to support the proposed design for the seismic rehabilitation of the dam.

We have completed our independent evaluation of the seismic stability of the dam, taking into consideration the updated geotechnical information and the nature and extent of the various soil zones within the embankment. Based on our analysis, the downstream sluiced fill materials are judged to be potentially liquefiable during the postulated safety evaluation earthquake. In addition, the dam, which is comprised mainly of the wagon fill material, will likely undergo moderate deformation which may damage the upstream slope.

Your proposal to improve the seismic performance of the dam consists of removing a portion of the sluiced fill material on the downstream slope and recompacting this same material to modern compaction standards. It is generally prudent to remove or remediate all potentially liquefiable soils as deemed possible to achieve the best performance of the dam during a seismic event. Otherwise, some damage to the dam can be expected.

Since the dam is operated with a minimum 23 feet of total freeboard, your proposal will be acceptable if robust filter and drainage features are provided between the recompacted material and the excavated face of the dam. Also, we will require the District to lower the reservoir and perform needed emergency repairs following any significant earthquake event with noted damage to the dam.

Your proposed schedule to submit the repair application for approval of plans and specifications after the CEQA process is complete is also acceptable. Our specific comments regarding the proposed design are enclosed.

Ms. Elizabeth Z. Bialek
JUN - 7 2012
Page 2

If you have any questions or need additional information, you may contact Design Engineer Richard Armstrong at (916) 227-1300 or Project Engineer Wallace Lam at (916) 227-4626.

Sincerely,

A handwritten signature in black ink, appearing to read "D. A. Gutierrez". The signature is stylized with a large initial "D" and a long horizontal stroke at the end.

David A. Gutierrez, Chief
Division of Safety of Dams

Enclosure

California Natural Resources Agency
Department of Water Resources
Division of Safety of Dams
Page 1 of 1

Enclosure to letter to Ms. Elizabeth Z. Bialek, Manager, Engineering Services
Comments regarding EBMUD's Conceptual Design of Seismic Rehabilitation.

1. As mentioned, robust filter and drainage features are required between the recompacted material and the excavated face of the dam. Maintaining the recompacted sluiced fill and the remaining portion of the downstream channel fill in an unsaturated condition is essential for satisfactory performance of the dam during the postulated safety evaluation earthquake.
2. The reservoir will need to be lowered to a safe elevation during excavation and construction of the downstream embankment. Your proposal to maintain a maximum reservoir elevation at 223 feet with approximately 9,000 acre-feet of storage may not be prudent. In order to minimize the risk during construction, a lower reservoir level needs to be considered and included in the planning study in compliance with the CEQA process.
3. The proposed dewatering system should be installed well in advance of construction and sufficiently monitored with instrumentation prior to the onset of the downstream slope excavation in order to maintain a stable excavated slope during construction.
4. The proposed modification of the existing 30-inch steel pipe should be completed and tested for adequacy prior to the embankment excavation to ensure the reliability of the outlet works in case of an emergency. The required drawdown rate is 10 percent of the maximum storage depth in seven days.
5. Geological mapping of excavated foundation surfaces is required to confirm design assumptions. The specification should allow for adjustment to the proposed excavation when actual field conditions are different from the available design information.

November 30, 2011

David A. Gutierrez, Chief
Division of Safety of Dams
2200 X Street
Sacramento, CA 95818

SUBJECT: Chabot Dam No. 31-5
Cross Section and Slope Stability

Dear Mr. Gutierrez:

As requested by your staff, the District has performed a supplemental subsurface investigation and analyses of the Chabot Dam embankment slopes. The investigation, which benefits from the direct input of your staff, provides more definitive information about the nature and extent of the various soils within the embankment. This letter presents a summary of the investigation, updated stability analyses and a proposed dam upgrade. More details on the results of the field investigation, including boring logs, are provided in the attached memorandum from URS, dated November 7, 2011.

New analyses were performed, building on the past analyses presented in the 2005 URS report *Dynamic Stability Analysis of Chabot Dam*. The cross section shown in Figure 1 includes the data from the field investigation program added to the interpretive cross section from URS 2005 Figure 7-3, which was the maximum cross section used to develop a model of the material zonation in the dam in the 2005 report. To address the concerns of your staff regarding the exact nature of the upstream soils, a sensitivity analysis was performed to determine if there would be any significant difference if the soils were assumed to be sluiced fill. To model this, the cross section now shows this material change on the upstream side. Note that this is similar to URS 2005 Figure 11-37, the maximum section for limit-equilibrium slope stability analyses, where reservoir silt is shown overlying the upstream slope to evaluate the effects of uncertainties in the characteristics of these materials. As far as the downstream side, the cross section was also modified to reflect the results of the recent investigation. Specifically, the boundary of the downstream sluiced fill has been adjusted to conform to the results of the new borings, including WI-69, WI-71, and WI-72, which are shown on the section. This cross section is based on actual soil data and is consistent with the historical records that we have recently obtained (see attached drawings from 1892, and the historical and descriptive summary of existing works from the August 1923 *Report on Development of Additional Water Supply from the San Leandro Creek Catchment Area*). The material distinction is also described in more detail on Page 3 of the attached URS report.

For the sensitivity analysis, the post-earthquake slope stability of the upstream cross section was analyzed using UTEXAS3 as described in Sections 9 and 11 of the URS 2005 report, using the material properties summarized in Tables 7-2, 9-1, and 9-2 with the maximum strength degradations as described in Section 11. The idealized geometry used for slope stability analyses is shown in Figure 2.

The post-earthquake upstream slope stability is shown in Figure 3. Blocks 1, 2, 3 were previously analyzed in the URS 2005 report, which summarizes the safety factors on Table 11-1 and the yield accelerations on Table 11-2. Block 1 is the critical surface. The safety factors and yield accelerations of Blocks 1 and 3 from the current evaluation are identical to those in the 2005 report. Although the Block 2 safety factor is lowered slightly, from 1.8 to 1.7 and the yield acceleration from 0.15 g to 0.13 g, the surface remains stable regardless of the characterization of the materials. Two additional surfaces, Blocks 4 and 5, were also analyzed and shown to be stable. Furthermore, it should be noted that URS had evaluated the effects of uncertainties in 2005 by assuming that the materials overlying the upstream slope were reservoir silt rather than wagon fill (Table 11-4 and Figure 11-37). The results of this analysis had shown that the displacement of the crest was 1.2 feet and that the earthquake-induced crest settlement was minimal.

As far as the downstream slope, the supplementary investigation provided more detail on the locations of the wagon fill versus sluiced fill as shown in Figure 1. Based on these findings and based on the current upstream stability evaluation, we propose to rehabilitate the portion of the downstream slope which has potentially liquefiable materials. These materials would be improved by excavating the downstream face of the dam on a 2:1 (horizontal: vertical) slope from the bench at Elevation 210 feet to bedrock, and replacing the excavated material with new compacted earth fill. While the excavation is in progress, the phreatic surface would be lowered below the sluice fill. The proposed slope during construction is shown in Figure 4, along with the results of the stability analysis for the temporary condition. The results show that the required safety factor of 1.3 is exceeded for all surfaces.

Finally, the downstream post-earthquake pseudo-static slope stability was analyzed for the reconstructed embankment, using maximum strength degradations as described in the URS 2005 report. The new earth fill was assigned the same properties as the modern fill that was placed in 1980, as both materials would be mostly clayey sands and gravels compacted to modern standards. The portion of the sluiced fill left in place would be confined by the new earth fill, and the geometry is shown together with the slope stability results on Figure 5. Blocks 1 and 3 were previously analyzed in the 2005 report, and the current evaluation shows that the safety factor and yield accelerations are improved on these blocks. Blocks 5 and 6 replace Blocks 2 and 4 from the 2005 report. The results show that the proposed rehabilitated dam has acceptable slope stability characteristics.

Mr. David A. Gutierrez
November 30, 2011
Page 3 of 3

Following your approval of the current evaluation, the District would develop the rehabilitation concept and begin the CEQA process. We expect to submit the Application for Approval of Plans and Specifications for the Repair or Alteration of a Dam and Reservoir at the end of the CEQA process, approximately 18 months after your approval of the current evaluation. If you have questions or comments, please contact Sean Todaro at (510) 287-0756.

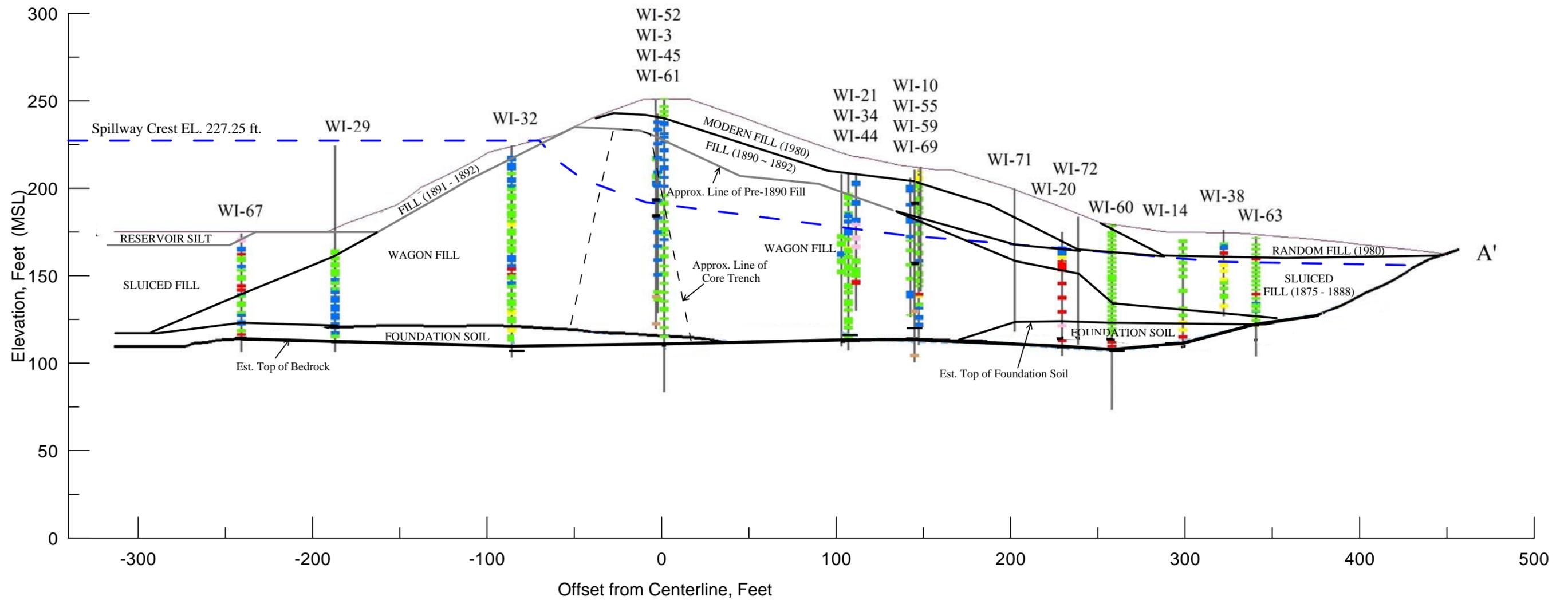
Sincerely,



ELIZABETH Z. BIALEK
Manager of Engineering Services

EZB:SMT:smt

Enclosures



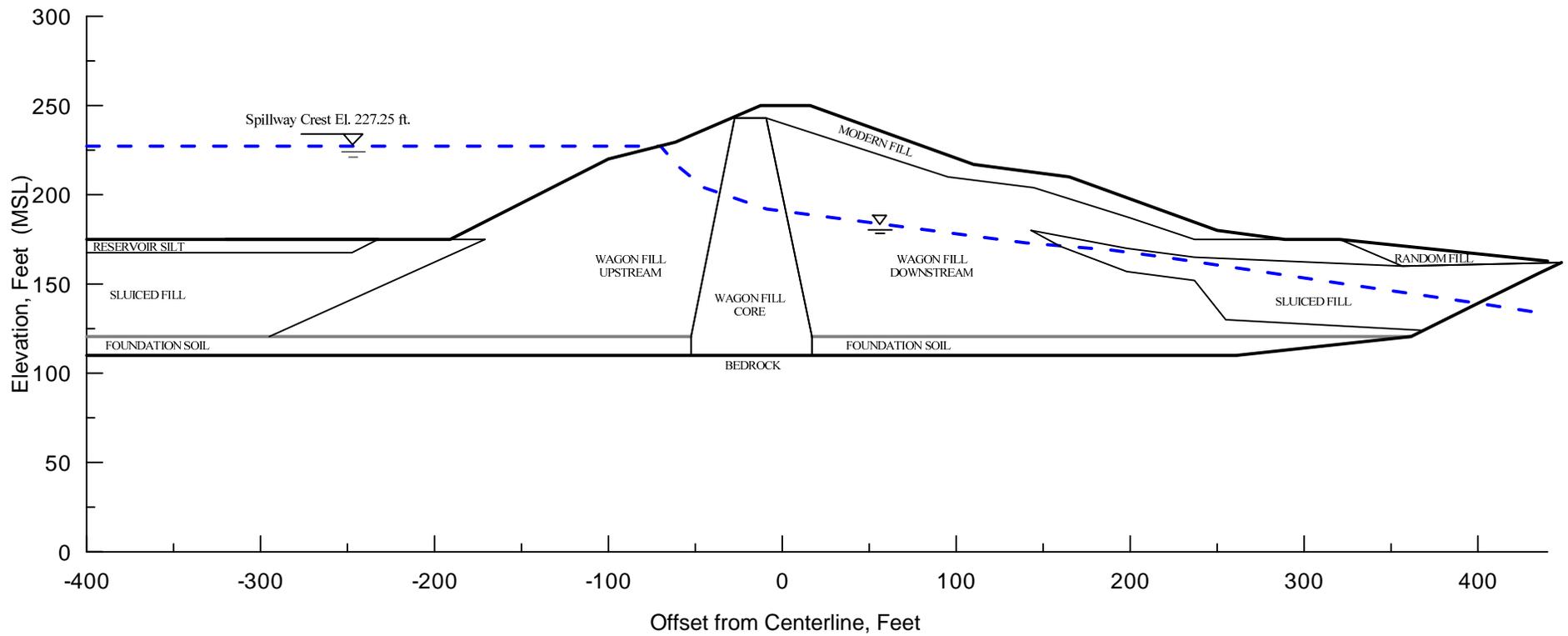
Legend

- Clays
(CH; CL; CL-OL; CL/GC; CL/GP; CL/SC; CL/SC/GC; CL/SP; CL/CH; CL-ML)
- Silts
(MH; MH/CH; ML; ML/SM; ML/CH)
- Sands and Gravels
(GP; GP/GW; GW; GW-GC; GP-GC; SP; SP-SC; SP-SM; SP/SW)
- Silty Sands
(SM; SM/GM; SM/SC)
- Clayey Sands
(GC; SC/GC; SC/CL; SC; SC/Fm; SC-SM)
- ← Piezometer reading (grey if dry)
- ← Piezometer tip
- ← Boring extent (extent below pre-dam topography in blue)
- ← Top of bedrock

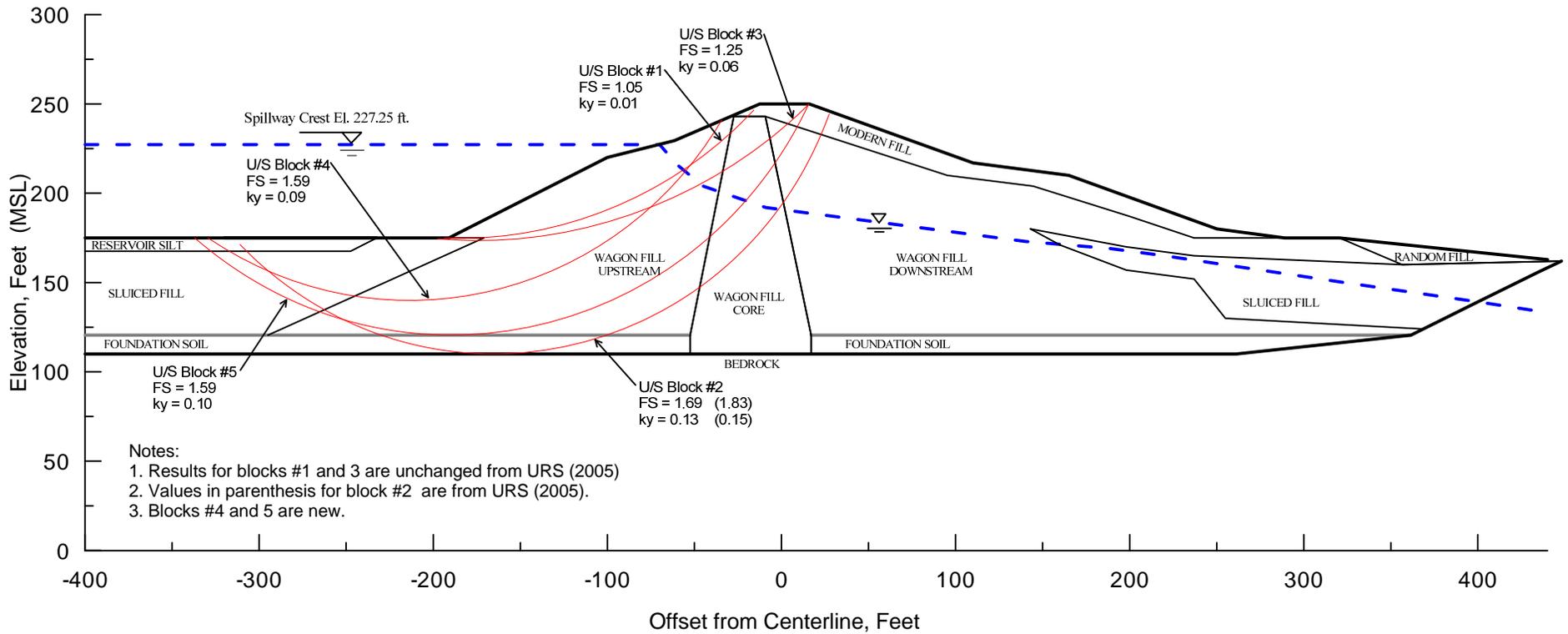
Notes:

1. This figure is an update of Figure 7-1 of URS 2005 report. Changes were made to include the additional borings (WI-68 through WI-72) performed on the downstream slope and toe in August 2011, and to reflect the revised interpretation of sluiced fill overlying upstream wagon fill as provided by EBMUD on November 10, 2011.

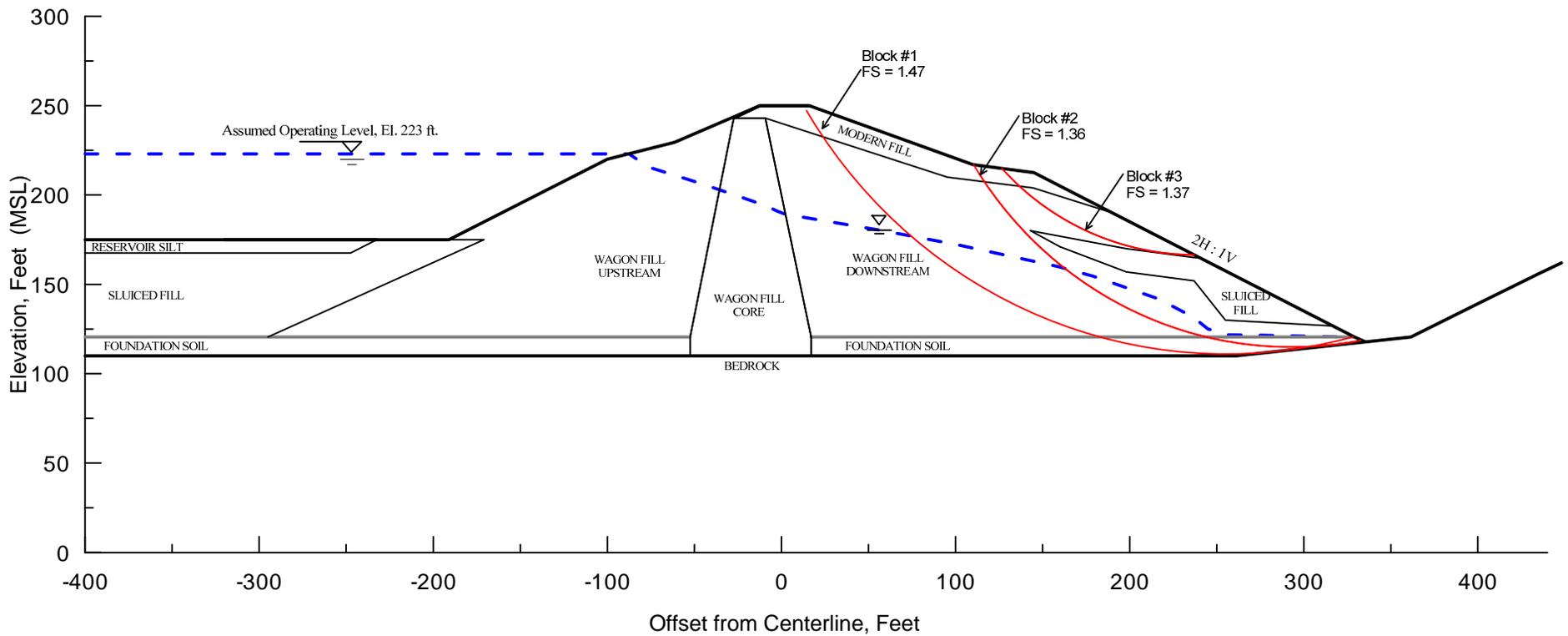
Project No. 26818113	Chabot Dam Conceptual Remediation	Updated Interpretive Cross Section A - A'	FIGURE 1
URS			



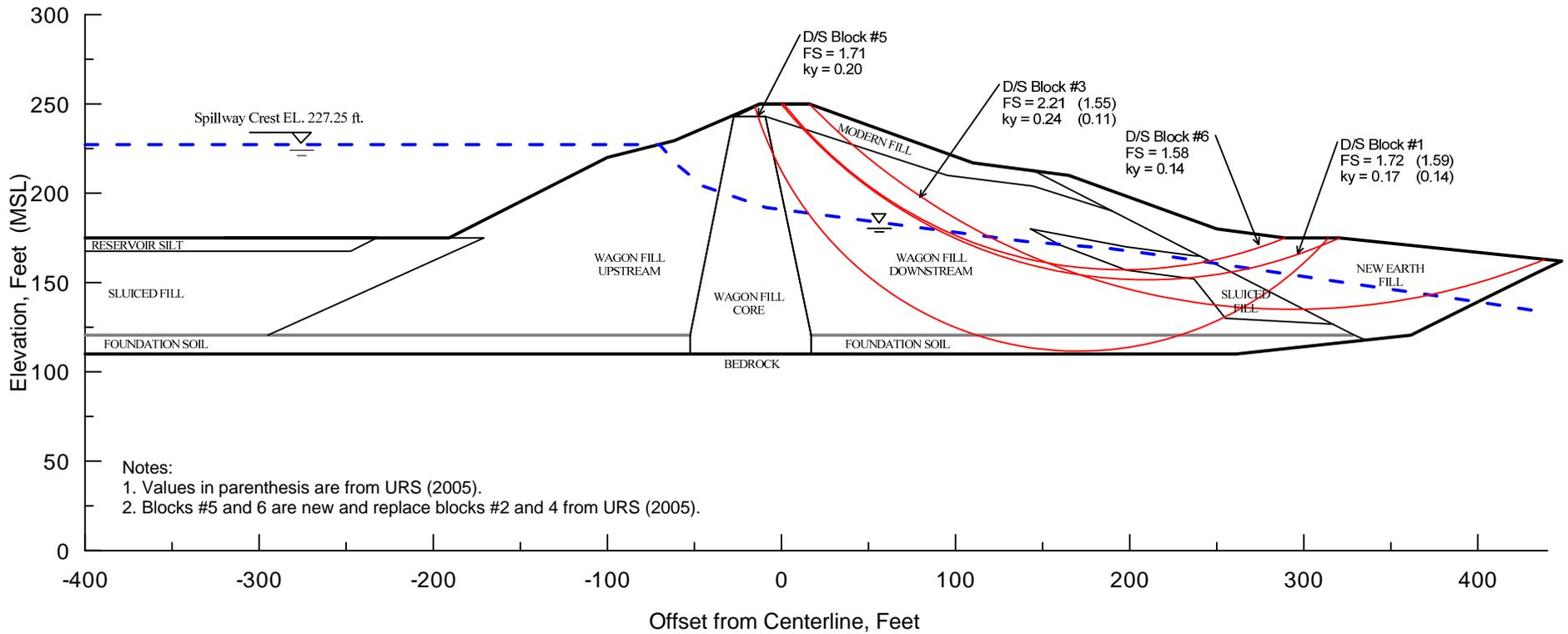
Project No. 26818113	Chabot Dam Conceptual Remediation	Updated Idealized Cross Section A-A'	Figure
URS			2



Project No. 26818113	Chabot Dam Conceptual Remediation	Post-Earthquake Stability Upstream Slope Updated Cross Section A-A'	Figure
URS			3



Project No. 26818113	Chabot Dam Conceptual Remediation	Downstream Slope Stability Under Construction Conditions Updated Cross Section A-A'	Figure 4
URS			



Project No. 26818113	Chabot Dam Conceptual Remediation	Post-Earthquake Stability Downstream Slope Remediated Cross Section A-A'	Figure
URS			5



EAST BAY MUNICIPAL UTILITY DISTRICT

Materials Engineering Section
375 Eleventh Street / P.O. Box 24055, MS# 610
Oakland, California 94623

Date: March 2, 2012

Fax (510) 287-1345

LETTER OF TRANSMITTAL

To: Mr. David Gutierrez, Chief
Division of Safety of Dams
P. O. Box 942836
Sacramento, CA 94236-0001

Project: Chabot Dam Seismic Upgrades

WE ARE SENDING YOU:

Attached Under separate cover

VIA:

Mail Federal Express Courier

Copies	Description
1	Check in the amount of \$980.00 for the balance of fees due
1	Cost affidavit for well construction at Chabot Dam
1	Cost breakdown for well construction at Chabot Dam

As requested For your review and comment For your approval
 For your use Approved as noted Returned with comments for correction

If you have any questions, please call Sean Todaro at (510) 287-0756.

Sincerely,

Sean Todaro

cc:

EAST BAY MUNICIPAL UTILITY DISTRICT

S2032B

ACCOUNTS PAYABLE
 P.O. BOX 24055
 OAKLAND, CA 94623-1055

XCG14EZ

EBMUD REFERENCE	P. O. NUMBER	INVOICE NUMBER	INVOICE DATE	INVOICE AMOUNT
MMIS1470458		DSOD.APP.FEE	02/23/12	980.00
CHECK NUMBER 00002389622			TOTAL	980.00

8510035429 / 11-14-1675 ©2011 - 0221

REMOVE DOCUMENT ALONG THIS PERFORATION

THIS DOCUMENT IS PRINTED IN TWO COLORS. DO NOT ACCEPT UNLESS BLUE AND GREEN ARE PRESENT.

EAST BAY MUNICIPAL UTILITY DISTRICT
 Accounts Payable
 P.O. Box 24055
 Oakland, CA 94623-1055

Wells Fargo Bank
 San Francisco, CA

11-24/1210

Check No.
 00002389622

Date: 03/02/12

Pay Amount: *****\$980.00

PAY Nine hundred eighty and 00/100 Dollars

Void After 180 Days

TO THE ORDER OF **STATE OF CALIFORNIA**
 DIVISION OF SAFETY DAMS
 PO BOX 942836
 SACRAMENTO, CA 94236

Standa Hendrix

⑈ 2389622⑈ ⑆ 121000248⑆ 4121064679⑈

STATE OF CALIFORNIA
THE RESOURCES AGENCY
DEPARTMENT OF WATER RESOURCES
Division of Safety of Dams

State of California
Department of Water Resources
Division of Safety of Dams
P. O. Box 942836
Sacramento, CA. 94236-0001

I, Elizabeth Bialek am the Engineering Manager of ERMUD
(owner) (owner, or authorized representative of owner)
of the Chabot Dam and Reservoir
(name of dam and reservoir)

The total cost of construction for work involving the dam is \$ 82,677.

Enclosed is an itemized list of the final costs.

I hereby declare under penalty of perjury that the above statement is true and correct.



(Signature)

23 February 2012

(Date)

Engineering Manager

(Title)

**DSOD APPLICATION FEE COST BREAKDOWN
CHABOT DAM WELL CONSTRUCTION**

1.0 Project Cost for Permit Application

\$13,927 Engineering Cost
\$68,750 Construction Contract
\$82,677 Total project cost for permit application

2.0 Permit Application Fee

\$2,480 Fee based on Application Fee Calculator (3% of total project cost)
\$1,500 Submitted with Application

\$980 Fee Currently Due

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**Appendix 3.4: Correspondence between EBMUD and East Bay Regional
Parks District (EBRPD) Regarding Chabot Dam Seismic Upgrade**

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May 25, 2012

Glenn Gilchrist, PE
Civil Engineer
East Bay Regional Park District
2950 Peralta Oaks Court
P. O. Box 5381
Oakland, CA 94605-0381

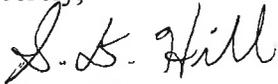
Ref. Request to Lower the Lake Chabot Water Level for Construction

Dear Glenn:

I am in receipt of your letter dated May 23, 2012 requesting that the water level at Lake Chabot be lowered to an elevation of 220 feet or lower by July 1 and 218 feet or lower by August 1, 2012.

We can meet that target dates and elevations with the stipulation that the water level will only come up again after it starts to rain, and that EBMUD will not release Upper San Leandro reservoir water to restore Lake Chabot reservoir to the normal operating level. Depending on precipitation and runoff the return to normal operating levels could take a year or more.

Sincerely,



Scott D. Hill
Manager of Watershed and Recreation

SDH/sdh

CC: J. Hurlburt
S. Todaro

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**Appendix 3.5: EBMUD Internal Memorandum(s) Regarding Chabot Dam
Seismic Upgrade (2 total)**

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EAST BAY MUNICIPAL UTILITY DISTRICT

DATE: March 8, 2012

MEMO TO: Board of Directors

THROUGH: Alexander R. Coate, General Manager *ARC*

FROM: Xavier J. Irias, Director of Engineering and Construction *XJI*

SUBJECT: Chabot Dam Seismic Upgrade Project

INTRODUCTION

In 2005, the District prepared a report on the dynamic stability analysis of Chabot Dam, which concluded that under the maximum credible earthquake there would be localized instability in the downstream toe of the dam and that the dam crest could settle 1.5 to 3.5 feet. Although this presents no risk to public safety, the expected minor slope deformations would require timely repair after an earthquake. In addition, the existing tower would likely fail in a brittle fashion, potentially damaging the outlet pipeline. This could impact the ability to drain the reservoir, which is a safety concern.

In light of these findings, upgrades to the dam and tower are required at this time. Staff evaluated various options and developed a recommended project. The recommended project is to remove and replace soil near the toe of the dam and complete a seismic retrofit of the tower outlet structure. Since Chabot Dam is regulated by the California Division of Safety of Dams (DSOD), the proposed work at the dam was reviewed by DSOD. The extent of necessary repairs was approved by DSOD in February 2012. This project will be discussed at the Planning Committee on March 13, 2012.

DISCUSSION

Purpose, Location, and Stakeholders

Chabot Reservoir, originally constructed in 1874, provides approximately 10,400 acre feet (3.4 billion gallons) of water for emergency storage. It is also used extensively for recreation: East Bay Regional Park District operates Lake Chabot Regional Park, and the City of San Leandro operates Chabot Park on the property (see Figure 1). Lake Chabot Regional Park is used for fishing, boating, hiking, bicycle riding, horseback riding, and picnicking; Chabot Park's amenities include picnic areas, play equipment, volleyball courts, restrooms and an amphitheater. The seismic upgrade project site is located in Wards 6 and 7, and in Oakland, San Leandro, and Castro Valley.

Need for Project

Repairs at the dam are necessary if it is to remain in service. DSOD will not allow the District to take no action at this time or permanently operate the reservoir at a reduced water level because of the risk posed by the outlet tower. The reservoir provides many benefits to the community, including local emergency water storage and public recreation. It also allows for more flexibility in operating the Upper San Leandro reservoir upstream. In addition, keeping the reservoir in service will avoid the biological impacts from removing the entire lake ecosystem, which would require significant environmental mitigation. Therefore, repair of the dam and tower are recommended.

Project Scope

Various technical alternatives have already been evaluated based on technical feasibility. At this time, for the dam repair work, conventional earth replacement is recommended based on constructability, cost effectiveness and a preliminary assessment of impacts. During construction, materials that are vulnerable to strength loss during an earthquake would be removed and stockpiled, then replaced and compacted to improve the strength and seismic performance of the dam.

To efficiently allow for the required earthwork, temporary stockpiles may be constructed on-site or off-site. Based on a preliminary investigation of viable stockpile sites, Figure 1 shows possible on-site stockpiles and possible off-site locations include Dunsmuir Reservoir in Oakland or San Leandro Rock Quarry in Castro Valley, pending discussions with the quarry owner. These alternatives will be further developed during the planning phase of the project. Stakeholder input will be considered so that site plans developed during the environmental documentation process will balance the project's technical requirements with community concerns.

In addition to the dam strengthening, repair or replacement of the tower is required for dam safety. Among various technical alternatives, the currently preferred option is to retrofit the tower outlet piping and reinforce the existing pavilion structure. This is the most cost-effective alternative that provides a safe post-earthquake reservoir outlet and preserves the historical pavilion on top of the structure. No public access to the pavilion is allowed as it has been an attractive nuisance. However, because of its landmark status, preservation of the structure is being considered. The tower alternative will also be considered further during the project's California Environmental Quality Act process. Tower photographs are shown in Figure 2 and the preferred upgrade is shown in Figure 3.

NEXT STEPS AND BUDGET

Next steps include starting agency and public outreach in April 2012. The planning phase will involve the preparation of an Environmental Impact Report, which will be developed in the summer of 2012 through the fall of 2013. Design will follow and be completed in early 2014,

Board of Directors

March 8, 2012

Page 3

and construction will begin after bid and award of the project in 2014. The project budget is approximately \$15 million for the dam and approximately \$2 million for the tower.

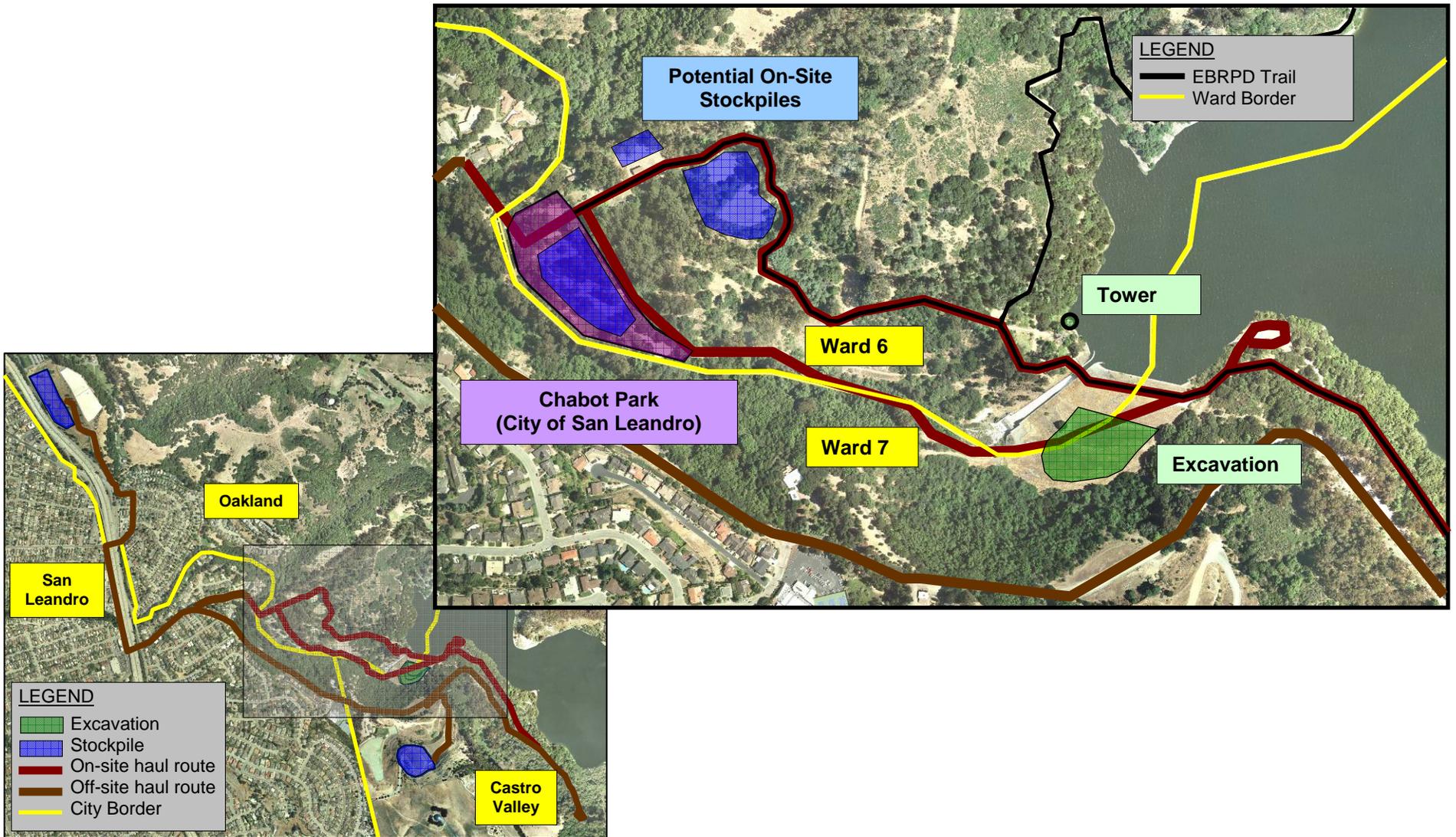
ARC:XJI:smt

I:\SEC\03-13-12 Planning Items\E&C Chabot Dam Seismic Upgrade Project.doc

Attachments: Figure 1 – Preliminary Project Layout

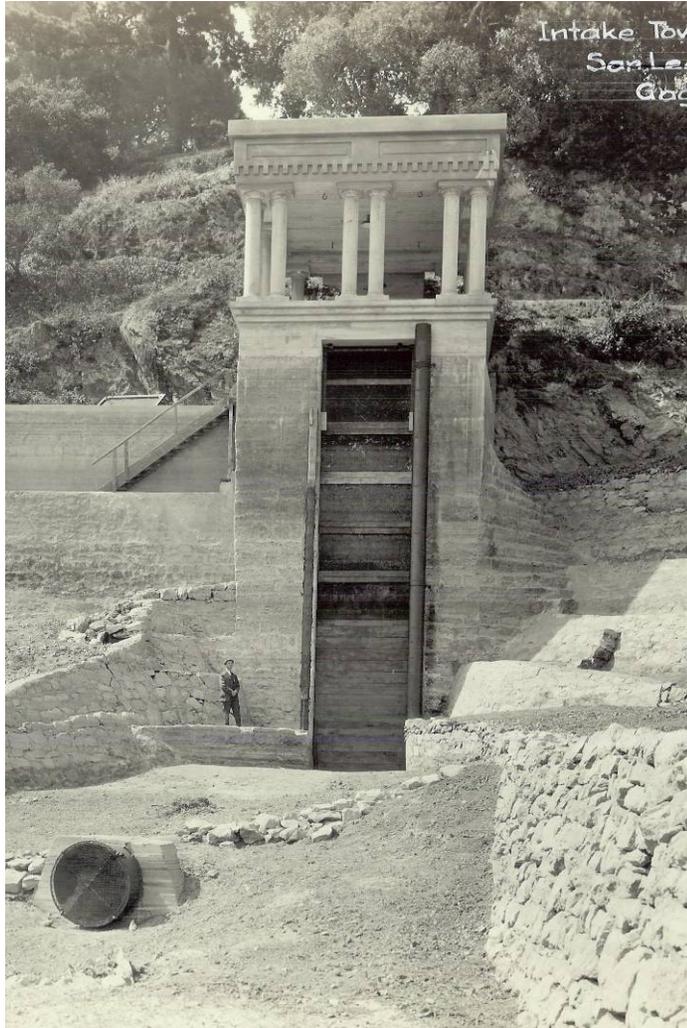
Figure 2 – Tower Photographs

Figure 3 – Tower Retrofit

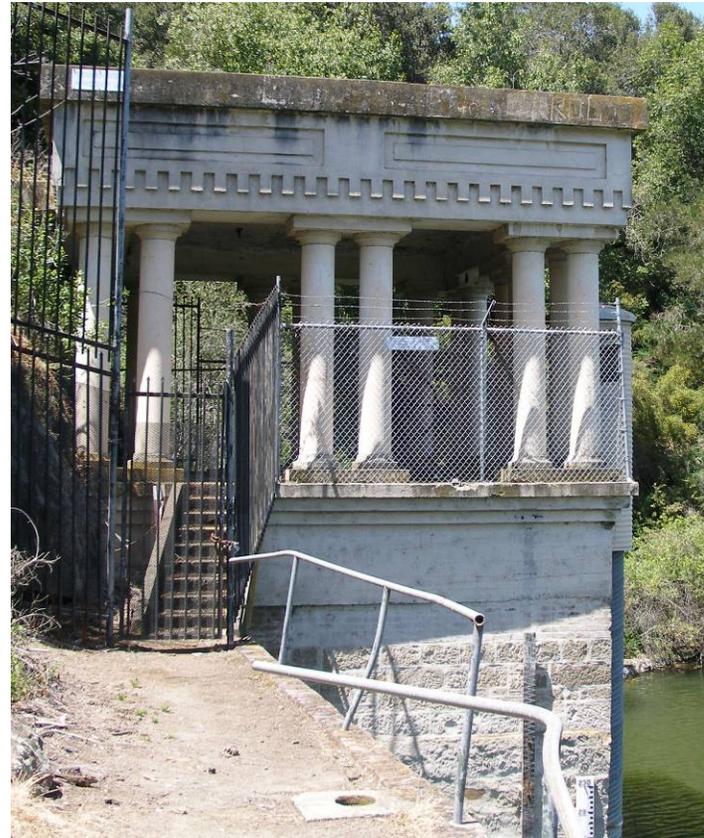


PRELIMINARY PROJECT LAYOUT

FIGURE 1

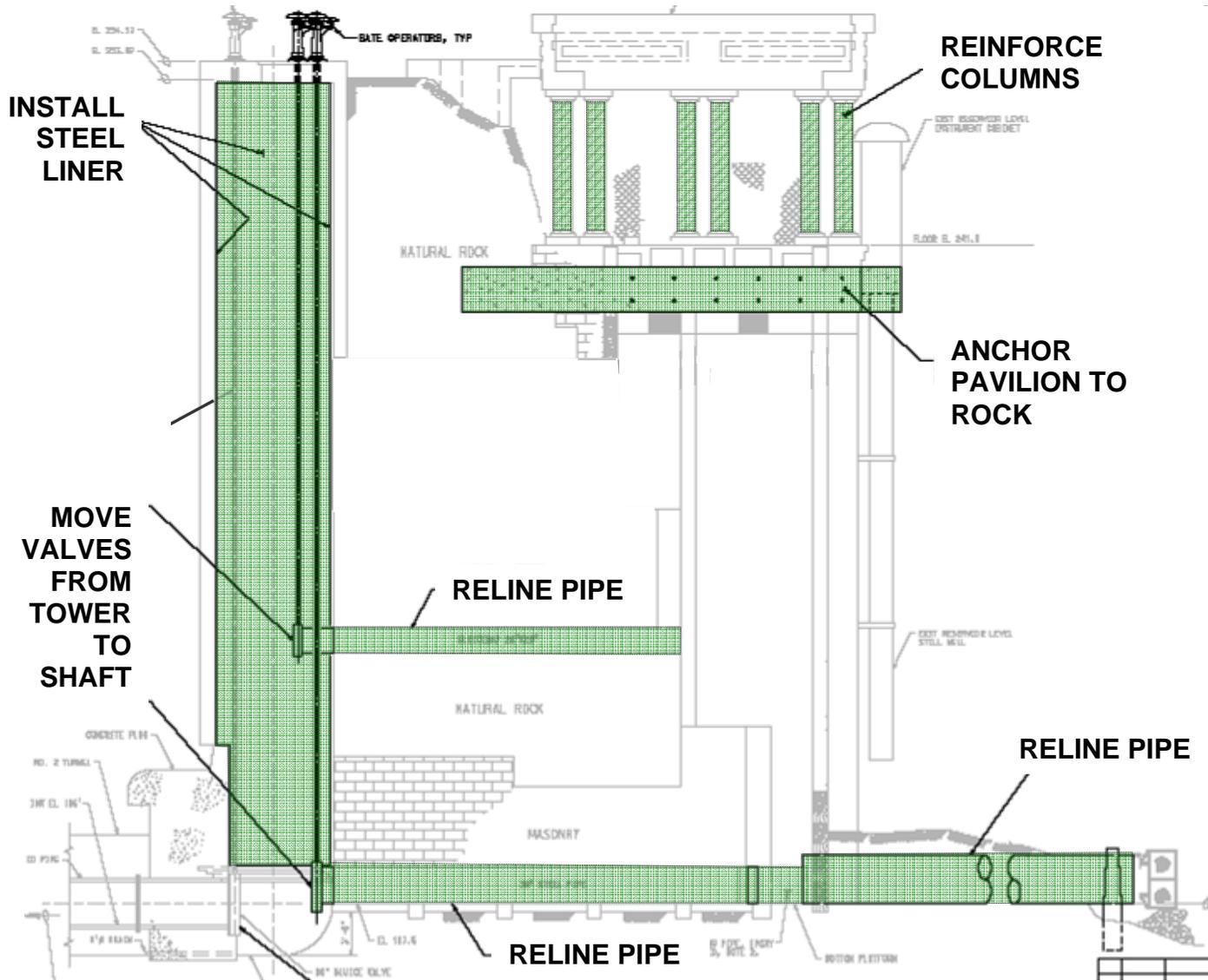


Tower and Pavilion in 1920s



Pavilion in 2011

TOWER PHOTOGRAPHS



TOWER RETROFIT

FIGURE 3

EAST BAY MUNICIPAL UTILITY DISTRICT

DATE: September 6, 2012

MEMO TO: Board of Directors

THROUGH: Alexander R. Coate, General Manager *ARC*

FROM: Xavier J. Irias, Director of Engineering and Construction *XJI*

SUBJECT: Chabot Dam Seismic Upgrade Project

SUMMARY

The purpose of this memo is to provide an update on the Chabot Dam Seismic Upgrade Project. The goal of the project is to improve the seismic stability of the dam and the seismic reliability of the outlet as required by the California Division of Safety of Dams. A presentation was provided to the Planning Committee on March 13, 2012. At this time, informational outreach meetings are beginning and the California Environmental Quality Act (CEQA) process will begin in October.

DISCUSSION

Public Outreach

To date, three informational briefings on the project have been given. The first was with the East Bay Regional Parks District (EBRPD) on June 21, 2012, which was attended by six EBRPD staff members, including the Park Supervisor, the Lakes Unit Manager, and the Community & Media Relations Manager. The second was to the EBRPD/EBMUD Liaison Committee on June 27, 2012, which was attended by members of the EBRPD and EBMUD's Board of Directors and select management staff. The third was with the City of San Leandro on July 12, 2012, and was attended by the Deputy City Manager, the Assistant City Manager, and the City Engineer.

At each of the presentations, EBMUD staff explained the need for the project and that the construction footprint and temporary closures would be similar to those during the 1980 Chabot Dam and Spillway Modifications Project. EBRPD's staff was most concerned with temporary trail closures at Lake Chabot Regional Park, and the City of San Leandro's staff was most concerned with temporary closure of Chabot Park and neighborhood traffic.

Two additional briefings and one community meeting are currently scheduled. The briefings are scheduled for the San Leandro City Council meeting on September 17, 2012 and the Friends of San Leandro Creek meeting on September 18, 2012. EBMUD is holding a community meeting on September 20, 2012 at the main San Leandro Public Library. Additional public outreach meetings will be scheduled as the formal CEQA process begins in October 2012.

CEQA Status

On June 11, 2012, District staff issued a Request for Proposals to prepare the Environmental Impact Report (EIR). A consultant will be selected to conduct constructability studies to refine the project alternatives prior to initiating the EIR and will also complete the EIR, working closely with EBMUD staff. Pursuant to CEQA, the EIR will identify potentially significant environmental issues, including specific environmental impacts and necessary mitigation measures. Authorization of the consulting agreement is scheduled for consideration by the Board of Directors on October 23, 2012.

NEXT STEPS

The scheduled milestone dates for the EIR are as follows:

- Award of CEQA Consultant Contract: October 2012.
- Issue Notice of Preparation: April 2013.
- Publish Draft EIR: December 2013.
- Consideration by the Board of Directors for approval of the EIR and project: May 2014.

Design is anticipated to start in the spring of 2014. Construction is planned to begin in the spring of 2015 and be completed in the fall of 2015.

ARC:EZB:smt

I:\SEC\09-06-12 Infos\E&C Chabot Dam Seismic Upgrade Project Update.doc

**Appendix 3.6: Contracting Documentation between EBMUD and AECOM,
Inc. for CEQA services, October 2012**

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October 25, 2012

Marcia Tobin, Principal,
AECOM
150 Chestnut Street
San Francisco, CA 94111

Re: Notice to Proceed with the Chabot Dam Seismic Upgrade Project – Preparation of
an Environmental Impact Report

Dear Ms. Tobin:

Enclosed is a signed copy of the Consulting Agreement for the Chabot Dam Seismic
Upgrade Project –Preparation of an Environmental Impact Report.

In accordance with Article 3 of the Consulting Agreement, East Bay Municipal
Utility District hereby authorizes AECOM to proceed with the scope of services
identified in Exhibit A, with the exception of the Dewatering Methods Review subtask
under Task 2.

If you have any questions, please contact me at (510) 287-1053.

Sincerely,

A handwritten signature in blue ink that reads 'Gwen A. Alie'.

Gwen A. Alie
CEQA Project Manager, Pressure Zone Planning

GAA:sb
sb12_101g.doc

Enclosure

cc: Marcia Tobin, AECOM

Marcia Tobin, Principal,
October 25, 2012
Page 2

bcc: B. Maggiore
S. Todaro
G. Alie
File
Chron

**CONSULTING AGREEMENT
FOR
EAST BAY MUNICIPAL UTILITY DISTRICT**

Chabot Dam Seismic Upgrade Project

THIS Agreement is made and entered into this 25 day of October, 2012, by and between **EAST BAY MUNICIPAL UTILITY DISTRICT**, a public entity, hereinafter called "DISTRICT," and **AECOM Technical Services, Inc.**, hereinafter called "CONSULTANT."

WITNESSETH

WHEREAS, DISTRICT requires consulting services for preparation of environmental documents; and

WHEREAS, CONSULTANT has submitted a proposal to provide consulting services for preparation of an Environmental Impact Report (EIR) for the Chabot Dam Seismic Upgrade Project including preparation of constructability studies to refine project alternatives; preparation of technical reports and an EIR for eleven subject areas including Aesthetics, Air Quality, Greenhouse Gas Emission, Biological Resources, Geology/Soils, Cultural Resources, Hazards/Hazardous Materials, Hydrology/Water Quality, Noise and Vibration, Recreation and Transportation/Traffic; related project management; presentations at public meetings and to the Board of Directors; and optional tasks of providing support during the Planning, Design and Construction Phase for the Chabot Dam Seismic Upgrade Project; and

Whereas CONSULTANT represents that it has the experience, licenses, qualifications, staff and expertise to perform said services in a professional and competent manner; and

WHEREAS, DISTRICT Board of Directors has authorized the contract by Motion Number 134-12;

NOW, THEREFORE, it is mutually agreed by DISTRICT and CONSULTANT that for the considerations hereinafter set forth, CONSULTANT shall provide said services to DISTRICT, as set forth in greater detail herein.

ARTICLE 1 - SCOPE OF WORK

- 1.1 CONSULTANT agrees to furnish services set forth in Exhibit A, Scope of Services, attached hereto and incorporated herein. The services authorized under this Agreement shall also include all reports, manuals, plans, and specifications as set forth in Exhibit A.

- 1.2 CONSULTANT's work products shall be completed and submitted in accordance with DISTRICT's standards specified, and according to the schedule listed, in Exhibit A. The completion dates specified herein may be modified by mutual agreement between DISTRICT and CONSULTANT provided that DISTRICT's Project Manager notifies CONSULTANT of modified completion dates by letter. CONSULTANT agrees to diligently perform the services to be provided under this Agreement. In the performance of this Agreement, time is of the essence.
- 1.3 It is understood and agreed that CONSULTANT has the professional skills necessary to perform the work agreed to be performed under this Agreement, that DISTRICT relies upon the professional skills of CONSULTANT to do and perform CONSULTANT's work in a skillful and professional manner, and CONSULTANT thus agrees to so perform the work. CONSULTANT represents that it has all the necessary licenses to perform the work and shall maintain them during the term of this Agreement. CONSULTANT agrees that the work performed under this Agreement shall follow practices usual and customary to the environmental planning profession and that CONSULTANT is the engineer in responsible charge of the work for all activities performed under this Agreement. Acceptance by DISTRICT of the work performed under this Agreement does not operate as a release of CONSULTANT from such professional responsibility for the work performed.
- 1.4 CONSULTANT agrees to maintain in confidence and not disclose to any person or entity, without DISTRICT's prior written consent, any trade secret or confidential information, knowledge or data relating to the products, process, or operation of DISTRICT. CONSULTANT further agrees to maintain in confidence and not to disclose to any person or entity, any data, information, technology, or material developed or obtained by CONSULTANT during the term of this Agreement. The covenants contained in this paragraph shall survive the termination of this Agreement for whatever cause.
- 1.5 The originals of all computations, drawings, designs, graphics, studies, reports, manuals, photographs, videotapes, data, computer files, and other documents prepared or caused to be prepared by CONSULTANT or its subconsultants in connection with these services shall be delivered to and shall become the exclusive property of DISTRICT. DISTRICT is licensed to utilize these documents for DISTRICT applications on other projects or extensions of this project, at its own risk. CONSULTANT and its subconsultants may retain and use copies of such documents, with written approval of DISTRICT.
- 1.6 CONSULTANT is an independent contractor and not an employee of DISTRICT. CONSULTANT expressly warrants that it will not represent that it is an employee or servant of DISTRICT.
- 1.7 CONSULTANT is retained to render professional services only and all payments made are compensation solely for such services as it may render and recommendations it may make in carrying out the work.

- 1.8 It is further understood and agreed by the parties hereto that CONSULTANT in the performance of its obligations hereunder is subject to the control or direction of DISTRICT as to the designation of tasks to be performed, the results to be accomplished by the services hereunder agreed to be rendered and performed, and not the means, methods, or sequence used by the CONSULTANT for accomplishing the results.
- 1.9 If, in the performance of this agreement, any third persons are employed by CONSULTANT, such person shall be entirely and exclusively under the direction, supervision, and control of CONSULTANT. All terms of employment, including hours, wages, working conditions, discipline, hiring, and discharging, or any other terms of employment or requirements of law, shall be determined by CONSULTANT, and DISTRICT shall have no right or authority over such persons or the terms of such employment.
- 1.10 It is further understood and agreed that as an independent contractor and not an employee of DISTRICT, neither the CONSULTANT nor CONSULTANT's assigned personnel shall have any entitlement as a DISTRICT employee, right to act on behalf of DISTRICT in any capacity whatsoever as agent, nor to bind DISTRICT to any obligation whatsoever. CONSULTANT shall not be covered by DISTRICT's worker's compensation insurance; nor shall CONSULTANT be entitled to compensated sick leave, vacation leave, retirement entitlement, participation in group health, dental, life or other insurance programs, or entitled to other fringe benefits payable by DISTRICT to employees of DISTRICT.

ARTICLE 2 - COMPENSATION

- 2.1 For the Scope of Services described in Exhibit A, DISTRICT agrees to pay CONSULTANT actual costs incurred, subject to a Maximum Cost Ceiling of \$944,435, plus a Professional Fee (prorata dollar profit). The Professional Fee shall be subject to a Professional Fee Ceiling of \$59,598. Total compensation under the Agreement shall not exceed a Maximum Agreement Ceiling of \$1,004,033. Compensation for services shall be in accordance with the method and amounts described in Exhibit B, attached hereto and incorporated herein. CONSULTANT acknowledges that construction work on public works projects is subject to prevailing wage rates and includes work performed during the design and preconstruction phases of construction including, but not limited to, inspection and land surveying work. CONSULTANT certifies that the proposed cost and pricing data used herein reflect the payment of prevailing wage rates where applicable and are complete, current, and accurate.
- 2.2 In case of changes affecting project scope resulting from new findings, unanticipated conditions, or other conflicts or discrepancies, CONSULTANT shall promptly notify DISTRICT of the identified changes and advise DISTRICT of the recommended solution. Work shall not be performed on such changes without prior written authorization of DISTRICT.

ARTICLE 3 - NOTICE TO PROCEED

- 3.1 This Agreement shall become effective upon execution of the second signature. CONSULTANT shall commence work upon receipt of DISTRICT's Notice to Proceed, which shall be in the form of a letter signed by DISTRICT's Project Manager. DISTRICT's Notice to Proceed will authorize the Contracted Services described in Exhibit A with ceiling prices described in ARTICLE 2 - COMPENSATION. No work shall commence until the Notice to Proceed is issued.
- 3.2 DISTRICT may at its option issue a Notice to Proceed for some or all of the Optional Services tasks described in Exhibit A. Compensation for Optional Services shall be in accordance with the method and amounts described in Exhibit B.

ARTICLE 4 - TERMINATION

- 4.1 This Agreement may be terminated by DISTRICT immediately for cause or upon 10 days written notice, without cause, during the performance of the work.
- 4.2 If this Agreement is terminated CONSULTANT shall be entitled to compensation for services satisfactorily performed to the effective date of termination; provided however, that DISTRICT may condition payment of such compensation upon CONSULTANT's delivery to DISTRICT of any and all documents, photographs, computer software, videotapes, and other materials provided to CONSULTANT or prepared by CONSULTANT for DISTRICT in connection with this Agreement. Payment by DISTRICT for the services satisfactorily performed to the effective date of termination, shall be the sole and exclusive remedy to which CONSULTANT is entitled in the event of termination of the Agreement and CONSULTANT shall be entitled to no other compensation or damages and expressly waives same. Termination under this Article 4 shall not relieve CONSULTANT of any warranty obligations or the obligations under Articles 1.4 and 7.1.
- 4.4 If this Agreement is terminated, payment of the Professional Fee shall be in proportion to the percentage of work that DISTRICT judges satisfactorily performed up to the effective date of termination. The Professional Fee shall be prorated based upon a ratio of the actual Direct Labor and Indirect Costs expended to date divided by the Cost Ceiling.

ARTICLE 5 - PROJECT MANAGERS

- 5.1 DISTRICT designates Gwen Alie as its Project Manager, who shall be responsible for administering and interpreting the terms and conditions of this Agreement, for matters relating to CONSULTANT's performance under this Agreement, and for liaison and coordination between DISTRICT and CONSULTANT. CONSULTANT may be requested to assist in such coordinating activities as necessary as part of the services. In the event DISTRICT wishes to make a change in the DISTRICT's representative, DISTRICT will notify CONSULTANT of the change in writing.

- 5.2 CONSULTANT designates Marcia Tobin as its Project Manager, who shall have immediate responsibility for the performance of the work and for all matters relating to performance under this Agreement. Any change in CONSULTANT designated personnel or subconsultant shall be subject to approval by the DISTRICT Project Manager

ARTICLE 6 - CONTRACT EQUITY PROGRAM COMPLIANCE

- 6.1 CONSULTANT expressly agrees that this Agreement is subject to DISTRICT's Contract Equity Program ("CEP"). CONSULTANT is familiar with the DISTRICT's CEP and Equal Opportunity Guidelines, and has read and understood all of the program requirements. CONSULTANT understands and agrees to comply with the CEP and all requirements therein, including each of the Good Faith Efforts. CONSULTANT further understands and agrees that non-compliance with the CEP requirements may result in termination of this Agreement.
- 6.2 Designated CEP compliance for the duration of this Agreement is listed in Exhibit C, which is attached hereto and incorporated herein. CONSULTANT shall maintain records of the total amount actually paid to each subconsultant. Any change of CONSULTANT'S listed subconsultants shall be subject to approval by the DISTRICT'S Project Manager.

ARTICLE 7 - INDEMNIFICATION AND INSURANCE

7.1 Indemnification

CONSULTANT expressly agrees to defend, indemnify, and hold harmless DISTRICT and its Directors, officers, agents and employees from and against any and all loss, liability, expense, claims, suits, and damages, including attorneys' fees, arising out of or pertaining or relating to CONSULTANT's, its associates', employees', subconsultants', or other agents' negligence, recklessness, or willful misconduct, in the operation and/or performance under this Agreement.

7.2 Insurance Requirements

CONSULTANT shall take out and maintain during the life of the Agreement all the insurance required in this ARTICLE, and shall submit certificates for review and approval by DISTRICT. The Notice to Proceed shall not be issued, and CONSULTANT shall not commence work until such insurance has been approved by DISTRICT. The certificates shall be on forms provided by DISTRICT. Acceptance of the certificates shall not relieve CONSULTANT of any of the insurance requirements, nor decrease the liability of CONSULTANT. DISTRICT reserves the right to require CONSULTANT to provide insurance policies for review by DISTRICT.

7.3 Workers Compensation Insurance

CONSULTANT shall take out and maintain during the life of the Agreement, Workers Compensation Insurance, for all of its employees on the project. In lieu of evidence of Workers Compensation Insurance, DISTRICT will accept a Self-Insured Certificate from the State of California. CONSULTANT shall require any subconsultant to provide it with evidence of Workers Compensation Insurance.

7.4 Commercial General Liability Insurance

CONSULTANT shall take out and maintain during the life of the Agreement Automobile and General Liability Insurance that provides protection from claims which may arise from operations or performance under this Agreement. If CONSULTANT elects to self-insure (self-fund) any liability exposure during the contract period above \$50,000, CONSULTANT is required to notify the DISTRICT immediately. Any request to self-insure must first be approved by the DISTRICT before the changed terms are accepted. CONSULTANT shall require any subconsultant to provide the appropriate amount of liability insurance coverages.

The amounts of insurance coverages shall be the following:

\$2,000,000/Occurrence, Bodily Injury, Property Damage – Automobile.

\$2,000,000/Occurrence, Bodily Injury, Property Damage - General Liability.

The following coverages or endorsements must be included in the policy(ies):

1. DISTRICT and its Directors, officers, and employees are additional insureds in the policy(ies) as to the work being performed under this Agreement.
2. The coverage is primary and non-contributory to any other insurance carried by DISTRICT.
3. The policy(ies) cover(s) contractual liability for the assumption of liability through the indemnity in this Agreement.
4. The policy(ies) is(are) written on an occurrence basis.
5. The policy(ies) cover(s) broad form property damage liability.
6. The policy(ies) cover(s) personal injury (libel, slander, and trespass) liability.
7. The policy(ies) cover(s) products and completed operations.
8. The policy(ies) cover(s) use of non-owned automobiles and equipment.

9. The policy(ies) shall not be canceled nor materially altered unless 30 days' written notice is given to DISTRICT.

7.5 Professional Liability Insurance

CONSULTANT shall take out and maintain during the life of the Agreement, professional liability insurance (Errors and Omissions) with a minimum of \$ 2,000,000 of liability coverage. A deductible may be acceptable upon approval of the DISTRICT. The policy shall provide 30 days advance written notice to DISTRICT for cancellation or reduction in coverage.

ARTICLE 8 - NOTICES

Any notice which DISTRICT may desire or is required at any time to give or serve CONSULTANT may be delivered personally, or be sent by United States mail, postage prepaid, addressed to:

AECOM Technical Services, Inc.
Attention: Marcia Tobin, Project Manager
150 Chestnut Street
San Francisco, CA 94111-1004

After November 1, 2012, please use the following address:

AECOM Technical Services, Inc.
Attention: Marcia Tobin, Project Manager
300 California Street, Suite 400
San Francisco, CA 94104

or at such other address as shall have been last furnished in writing by CONSULTANT to DISTRICT.

Any notice which CONSULTANT may desire or is required at any time to give or serve upon DISTRICT may be delivered personally at EBMUD, 375 - 11th Street, Oakland, CA 94607-4240, or be sent by United States mail, postage prepaid, addressed to:

East Bay Municipal Utility District, M/S701
Bill E. Maggiore, Senior Civil Engineer
P.O. Box 24055
Oakland, CA 94623-1055

or at such other address as shall have been last furnished in writing by DISTRICT to CONSULTANT.

Such personal delivery or mailing in such manner shall constitute a good, sufficient and lawful notice and service thereof in all such cases.

ARTICLE 9 - MISCELLANEOUS

- 9.1 This Agreement represents the entire understanding of DISTRICT and CONSULTANT as to those matters contained herein. No prior oral or written understanding shall be of any force or effect with respect to those matters covered hereunder. This Agreement may only be modified by amendment in writing signed by each party.
- 9.2 This Agreement is to be binding on the successors and assigns of the parties hereto. The services called for herein are deemed unique and CONSULTANT shall not assign, transfer or otherwise substitute its interest in this Agreement or any of its obligations hereunder without the prior written consent of DISTRICT.
- 9.3 Should any part of this Agreement be declared by a final decision by a court or tribunal of competent jurisdiction to be unconstitutional, invalid or beyond the authority of either party to enter into or carry out, such decision shall not affect the validity of the remainder of this Agreement, which shall continue in full force and effect, provided that the remainder of this Agreement can be interpreted to give effect to the intentions of the parties.
- 9.4 Multiple copies of this Agreement may be executed by the parties and the parties agree that the Agreement on file at the DISTRICT is the version of the Agreement that shall take precedence should any differences exist among counterparts of the Agreement.
- 9.5 This Agreement and all matters relating to it shall be governed by the laws of the State of California.
- 9.6 The DISTRICT's waiver of the performance of any covenant, condition, obligation, representation, warranty or promise in this agreement shall not invalidate this Agreement or be deemed a waiver of any other covenant, condition, obligation, representation, warranty or promise. The DISTRICT's waiver of the time for performing any act or condition hereunder does not constitute a waiver of the act or condition itself.
- 9.7 There shall be no discrimination against any person or group of persons, on account of race, color, religious creed, national origin, ancestry, gender including gender identity or expression, age, marital or domestic partnership status, mental or physical disability, medical condition, genetic information, or sexual orientation in the performance of this contract. CONSULTANT shall not establish or permit any such practice(s) of discrimination with reference to the contract or any part thereof. CONSULTANTS determined to be in violation of this section shall be deemed to be in material breach of this Agreement.
- 9.8 CONSULTANT affirms that it does not have any financial interest or conflict of interest that would prevent CONSULTANT from providing unbiased, impartial service to the DISTRICT under this Agreement.

ARTICLE 10 - TERMS

Unless terminated pursuant to Article 4 herein, this Agreement shall expire when all tasks have been completed and final payment has been made by DISTRICT. The terms of this Agreement may be amended only in writing signed by both parties.

IN WITNESS WHEREOF, the parties hereto each herewith subscribe the same in duplicate.

EAST BAY MUNICIPAL UTILITY DISTRICT

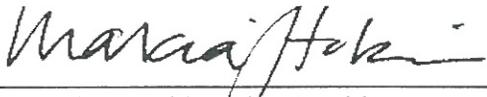
By: 
Xavier J. Irias - Director of Engineering and Construction

Date 10-25-12

Approved As To Form

By: 
for the Office of the General Counsel

AECOM TECHNICAL SERVICES, INC.

By: 
Marcia J. Tobin, Vice President

Date 10/16/2012

EXHIBIT A

East Bay Municipal Utility District Chabot Dam Seismic Upgrade Project

SCOPE OF SERVICES

I. CONSULTANT SERVICES

CONSULTANT (AECOM) shall provide the following:

SCOPE OF WORK

The objectives of the Chabot Dam Seismic Upgrade Project are to improve the embankment soils on the downstream side of Chabot Dam to withstand shaking generated by the maximum credible earthquake on the Hayward Fault without significant strength loss, to limit permanent deformation or settlement at the dam crest to acceptable levels, to prevent damage to the outlet works from the design level earthquake, and to continue reservoir and outlet works operation during construction. East Bay Municipal Utility District (EBMUD) is considering two construction options: a) conventional earthwork and b) ground improvement.

The following Scope of Work describes the work that the AECOM team will undertake to complete constructability studies, refine project alternatives and prepare an Environmental Impact Report (EIR). The EIR will address potentially significant environmental issues associated with project development and construction including conducting analyses to identify specific environmental impacts and mitigation measures necessary to reduce the environmental impacts to less than significant where feasible, pursuant to the California Environmental Quality Act, (CEQA). Both the conventional earthwork and ground improvement construction options need to be fully evaluated in the EIR.

Task 1. Project Initiation

Attend Project Kickoff Meeting

At the kick-off meeting, discussion will focus on the project objectives, key project components and issues, data needs, and contract schedule. The AECOM team will discuss questions and required clarifications on the information package that EBMUD will provide prior to the kick-off meeting. The meeting objective is to confirm EBMUD's expectations and deliverables for the project.

Attend Site Visit

To gain a clear understanding of the field conditions of the project components (outlet works, alternative haul routes, and stockpile locations) and adjacent neighborhood characteristics, particularly along the alternative haul routes, the lead technical specialists will attend the site visit with EBMUD staff.

Prepare Project Work Plan and Schedule

AECOM will prepare a project work plan and schedule detailing all tasks and deliverables for EBMUD's review and approval. AECOM will schedule a meeting with EBMUD and key AECOM team members to confirm the work plan, schedule and approach on key issues.

Review of Project Materials

The AECOM team will discuss questions, required clarifications, and additional data needs with the EBMUD team at the Project Kickoff meeting. If needed, particularly for the technical specialists in geotechnical issues, biology, cultural resources, recreation, and transportation, the AECOM team will conduct an additional site visit to gain a clear understanding of the project site conditions and neighborhood characteristics.

***Deliverable** - Project Work Plan and Schedule Provide: searchable PDF, Microsoft schedule and four hard copies.*

Task 2. Constructability and Environmental Review of Alternative Haul Routes and Stockpiles, Dewatering and Outlet Works Construction

Objectives

To determine the technical feasibility, cost, constructability, and environmental aspects of:

- The alternative haul routes and stockpiles for temporary storage of excavated embankment soils.
- Dewatering methods to ensure safe excavation slopes.
- The alternative upgrades to the outlet works.
- The seismic upgrade methods (conventional earthwork, cement deep soil mixing, or a combination of the two).

To determine the distinguishing environmental aspects of each project alternative, the environmental team will review all issues on the CEQA checklist with a primary focus on biological resources, traffic, noise and recreation. Based on the outcomes of Task 2, EBMUD will refine and select the preferred project alternatives for each construction option. Each alternative may include multiple haul routes and stockpile

locations. EBMUD's project description for the selected project alternatives will be used by AECOM to update the Initial Study (Task 3), as well as form the basis for evaluation of environmental issues in the Technical Reports (Task 5) and preparation of the EIR (Tasks 6 -8).

Alternative Haul Routes and Stockpiles Constructability Review

This review will consider construction costs, schedule requirements, and distinguishing environmental issues in order to clearly outline the benefits and challenges of each alternative. The AECOM team will develop the key evaluation criteria for the review, conduct a site visit, define the scope of improvements and upgrades to the existing roads along the proposed haul routes and stockpile areas, and prepare the preliminary construction cost estimate in sufficient detail to provide cost comparison between alternatives, including costs of permitting and mitigation of each alternative. The purpose of this review is to allow EBMUD to understand the practicality of each haul route and stockpile area and to determine if any can be screened out prior to the formal CEQA analysis.

Dewatering Methods Review

This review will use well test results provided by EBMUD to assess the feasibility of different dewatering techniques (e.g., wellpoints, wellpoint vacuum, jet eductors, etc.) to allow for safe excavation slopes. The review will consider the technical feasibility, cost, required dewatering duration, and risks associated with the preferred method.

Deliverable – *Dewatering methods review report describing the feasible dewatering techniques, cost, dewatering duration, and associated risks.*

Outlet Works Constructability Review

The construction process alternatives for the outlet works replacement/relining will be reviewed and expanded upon to determine the feasibility of completing construction of the outlet works in the same year as the earthwork for the dam (2015), or if the outlet works construction needs to be done one year prior to the earthwork for the dam (2014). The outcome of this review will allow EBMUD to determine the construction schedule for the project.

Deliverable - *Constructability review report describing the constructability review process, the evaluation criteria used, and a comparison of the alternatives. The report will contain a matrix that outlines the constraints and benefits of each of the alternatives to facilitate decision-making.*

Provide - *Three hard copies and one CD that contains the file in Word and PDF formats.*

Task 3. Finalize Initial Study

Utilizing knowledge gained from review of EBMUD's background material on the project and the project description provided by EBMUD, AECOM will modify and finalize EBMUD's Draft Initial Study, including the Findings of Significance.

AECOM anticipates that the following CEQA issue areas can be scoped out of the Initial Study and not evaluated further in the EIR: agriculture and forestry, mineral resources, public services, utilities/service systems, population/housing and land-use. AECOM will meet with EBMUD to confirm the environmental issues with the potential for significant impact that will be analyzed in the Technical Reports and subsequent EIR.

***Deliverable** - Final Initial Study, allowing one week for EBMUD review and approval.*

***Provide** - Three hard copies and one CD that contains the file in Word and PDF formats.*

Task 4. Notice of Preparation of EIR

AECOM will prepare an Notice of Preparation (NOP) to comply with all informational requirements of CEQA as set forth in the State CEQA Guidelines. The NOP (prepared under Task 3 above) will be distributed by EBMUD. AECOM will work with EBMUD to identify public agencies and other interested parties for the distribution list. AECOM will discuss with EBMUD staff the desirability of scheduling an agency scoping meeting. If requested, preparation and attendance at a public scoping meeting will be included as an optional task.

***Deliverable** - Notice of Preparation and Mailing List for distribution by EBMUD.*

***Provide** - Four hard copies and one CD that contains the file in Word and PDF format.*

Task 5. Prepare Technical Environmental Reports

Each of the environmental topics identified in the Initial Study as having the potential for significant impact will be evaluated in detail, for the project alternatives selected by EBMUD as a result of Task 2. The topic sections include aesthetics, air quality, biological resources, cultural resources, geology/soil, greenhouse gas emissions, and hazards/hazardous materials, hydrology/water quality, noise, recreation, and transportation/traffic. For each topic, the relevant regulatory requirements and standards will be described, as well as the existing baseline physical conditions of the project site and surroundings from a regional and site-specific perspective. The State Planning regulation that exempts EBMUD from specified local government regulations for specified water facilities (California Government Code 53091 (d)) shall be considered in each Technical Report.

The section on Impacts and Mitigation Measures will identify the thresholds of significance, and the impact evaluation including the direct and indirect impacts of the

project on the environment, for the selected project alternatives. Thresholds of significance, impacts and mitigation measures for each alternative should be clearly differentiated, as applicable, to guide decision making on the project that will ultimately be approved for construction. Impact evaluation will focus on the short-term; long-term environmental impacts are not anticipated once project construction is completed. For those impacts considered to be significant or potentially significant, mitigation measures will be proposed in close coordination with EBMUD staff.

As this evaluation will serve as a foundation for the Technical Environmental Reports and the EIR, one or more meetings will be held with EBMUD to review and discuss questions and clarify direction, particularly regarding the thresholds of significance, and initial assessment of impacts and potential mitigation measures for each selected project alternative.

Deliverable(s) - Meeting minutes.

AECOM will prepare technical reports which consist of stand alone, objective, and detailed evaluations of each of the environmental issues identified for EIR analysis in the Initial Study, for the selected project alternatives. Each technical report will incorporate the environmental setting, impacts and mitigation measures. The AECOM team will coordinate with EBMUD staff as each report is developed. The analysis from these technical reports will form the basis of the Administrative Draft EIR.

Aesthetics

This report will address the anticipated visual effects from the soil excavation, temporary stockpiling of the excavated soils at stockpile locations (to be determined under Task 2) and related tree/vegetation removal, replacement/relining of outlet works pipeline, and demolition of the Pavilion above the outlet works.

The existing visual environment will be described. Impacts to scenic vistas and other scenic resources will be determined based on the field visit and review of the Alameda General Plan. Key viewpoints will be identified and photographed. Visual changes anticipated from the project will be described in relation to the existing visual conditions.

For analysis of pavilion alternatives, up to four photo simulations will be created from photos of the site. If a faux pavilion at a different location is created, it will be based on a photograph of the existing pavilion. The simulations will be used to evaluate potential impacts, and to facilitate an understanding and discussion of each pavilion alternative. The simulations will ultimately guide selection of which pavilion alternative is selected by EBMUD.

Mitigation measures will be applied to the proposed project where feasible and applicable to avoid potentially significant impacts. Where necessary, the AECOM

team will identify additional approaches and features (e.g., built, landscape, and natural) as mitigation measures for visual impacts.

Air Quality

This report will evaluate impacts to air quality from construction of the project to sensitive receptors. This will include evaluating the list of construction equipment, operating hours and days, calculating emissions using a standard model approved by the Bay Area Quality Management District (BAAQMD), and comparing emissions to the lead agency's CEQA significant thresholds for Alameda County. The BAAQMD is the public agency entrusted with regulating stationary sources of air pollution in Alameda County and surrounding counties. Thresholds were established in 1999 and later updated in 2010; however, the new thresholds were set aside by court order. This issue will be discussed with EBMUD to confirm the preferred methodology. Several on and off-site locations may be impacted by project construction, including roads, stockpiles, and the excavation area. This topic will be addressed in the report and air quality emissions will be evaluated for all locations.

Biological Resources

Based on an initial review of available information, AECOM understands that sensitive biological resources at the project site may include wetlands and other potentially jurisdictional waters of the U.S., native trees subject to permitting under the City of Oakland and County of Alameda tree ordinances, and potential for occurrence of several special-status species, including California red-legged frog, Alameda whipsnake, California tiger salamander, central California coast steelhead distinct population segment (DPS), and a suite of early- and late-blooming special-status plant species.

To supplement this information, the AECOM team will review the EBMUD Draft Initial Study for the project, California Department of Fish and Game's Natural Diversity Database (CNDDDB), California Native Plant Society's (CNPS's) Online Inventory of Rare and Endangered Plants of California, the EBMUD Habitat Conservation Plan (EBMUD HCP) and Watershed Master Plan (EBMUD WMP), and applicable city and county tree ordinances.

Biological reconnaissance surveys will be conducted of the project work areas, including vegetation mapping and habitat assessments for special-status species, focused special-status plant surveys over two intervals (once in early spring and once in late spring/early summer) and wetland delineations. The AECOM team will also conduct a tree inventory to assess impacts of tree removal for various alternatives and mitigation requirements to comply with local tree ordinances. To assess potential for impacts to California red-legged frog, AECOM anticipates that a Site Assessment per U.S. Fish and Wildlife Services (USFWS) 2007 guidelines would be sufficient and no further protocol surveys would be needed.

The wetland delineation report, tree inventory, and California red-legged frog Site Assessment would be prepared as stand-alone reports and attached as appendices to the Biological Resources Technical Report, and summarized in the technical report and EIR Biological Resources chapter. These appendices would then be used directly for further necessary permitting processes for the project (as discussed under the Optional Tasks in this scope of work).

In the Biological Resources Technical Report, potential impacts of the project will be quantified including habitat type acreages (including wetlands and waters of the US affected); the potential presence and likely effects to any special-status species; and the quantity, type and size of trees subject to local tree ordinances that would be removed under different alternatives. The AECOM team will provide clear and detailed analysis and mitigation for all potentially significant direct, indirect, and cumulative impacts of the proposed project on biological resources, designing mitigations in conformance with EBMUD and resource agency requirements.

Geology/Soils

This report will describe the geological, soils, and seismic setting of the project site using existing information and site specific geotechnical data and seismicity data obtained by EBMUD and summarized in technical memoranda and/or reports prepared by or for EBMUD staff. Based on this information, the impact analysis will consider the seismic and soil stability effects of the upgraded dam structure. Geologic hazards on the project site will be described using applicable publications by the U.S. Geologic Survey (USGS) and other agencies in addition to the technical memoranda and/or reports prepared by or for EBMUD. This section will also analyze whether the proposed project would expose people or structures to potential substantial adverse effects involving rupture of a known earthquake fault/strong seismic ground shaking/seismic-related ground failure/landslides, result in substantial soil erosion, located on a unstable geologic unit, located on expansive soil, or substantially change the topography of the site.

Design recommendations proposed as part of the project to control potential adverse effects in the context of relevant State, City and other applicable regulations related to soils, seismic safety, groundshaking, liquefaction, and other ground failures will be identified. Cumulative geology and soils effects of the proposed project in consideration with other projects will be discussed. Mitigation measures to reduce or avoid potential adverse geology and soils effects will be identified.

Cultural Resources

AECOM will conduct cultural resources investigations and prepare a Cultural Resources Technical Report in accordance with standards for the identification of CEQA historical resources. In order to adequately identify CEQA historical resources, AECOM will coordinate with Native American representatives for their input, conduct a records search at the Northwest Information Center, conduct research

and field investigations to identify historic and archaeological resources in the project area. AECOM architectural historians will evaluate the California Register of Historical Resources (CRHR) eligibility of up to two historical resources (the Chabot Dam and related structures). For the purposes of this scope and cost estimate, it is assumed that no new archaeological resources will be identified or recorded in the project area and that it will not be necessary to evaluate any new archaeological resources.

Using the results of pre-field research and field surveys, AECOM archaeologists and architectural historians will prepare a stand-alone Cultural Resources Technical Report consistent with CEQA standards. The report will provide an overview of the prehistoric, ethnographic, and historic-era developments of the project area and the surrounding region, outline AECOM's research methodology, and discuss the findings of the research, inventory, and evaluation efforts. The Cultural Resources Technical Report will be summarized in the EIR. Project impacts will be analyzed and incorporated into the EIR, along with the development of mitigation measures, if necessary.

Greenhouse Gas Emissions

A general summary of Federal, State, and regional climate change and greenhouse gas-related regulations applicable to the proposed project will be described, referring to the Air Quality technical study for background information on regional climate and meteorology. The scientific background behind greenhouse gas emissions (GHG) and climate change will be provided as well as the current levels of emissions within Alameda County and the State of California using the Air Resources Board (ARB) GHG Emissions Inventory.

Pursuant to the State Office of Planning and Research (OPR) 2010 CEQA Amendments, AECOM will quantify direct (e.g., area and mobile sources) and indirect (e.g., electricity, water, waste) GHG emissions associated with the existing and proposed project's land uses. Methods and emission factors used to quantify construction and operational GHG emissions will be consistent with those recommended by BAAQMD and ARB. Construction-related GHG emissions will be quantified using methods similar to those for Air Quality. CalEEMod will also be used to quantify direct and indirect GHG emissions associated with existing and proposed operational activities.

Although carbon dioxide (CO₂) is not the only GHG, the focus of the analysis will be on CO₂ generation. A majority of available data on GHG generation is CO₂ based, and CO₂ can be used as a suitable indicator of overall GHG emissions for land use development projects. However, when available, methane (CH₄) and nitrous oxide (N₂O) emissions will be incorporated in the total GHG emissions to calculate metric tons of carbon dioxide equivalent (MT CO₂e).

Based on the OPR's first GHG significance criterion, the analysis will also evaluate whether the modeled level of GHG emissions generated by the proposed project's construction and net operational activities (i.e., proposed minus existing emissions) constitutes a substantial contribution to the significant adverse cumulative impact of global climate change. Significance thresholds from various regulatory agencies throughout California, including BAAQMD, will be used to provide context for the proposed project's GHG emissions. The section will also qualitatively discuss any potential adverse impacts to the proposed project from climate change and vice versa. If significant impacts are identified, mitigation measures that clearly identify timing, responsibility, and performance standards will be prepared for any adverse effects.

Hazards/Hazardous Materials

The project area will be evaluated for hazards and hazardous materials through review of probable past site uses and their possible impact on the current environmental status of the site, a site visit to visually identify areas of possible contaminated soil or surface water, improperly stored hazardous materials, possible sources of PCBs, and possible risks of contamination from activities at the site and adjacent properties. Available local regulatory files, databases, and historical documents will be reviewed to identify known hazardous waste sites, landfills, and leaking underground storage tanks.

Hydrology and Water Quality

The hydrology and water quality technical report will focus on four main water resource topics including:

1. Short-term temporary construction-related effects on hydrology and water quality;
2. Long-term operations-related water quality;
3. Permanent changes to stormwater drainage and/or flooding; and
4. Cumulative onsite and offsite hydrology and water quality impacts.

The environmental setting will briefly describe existing drainage and water quality conditions of the project study area based on existing information. Applicable federal, state, and local regulations for stormwater drainage, grading and erosion control, and other water quality protection procedures will be described.

The impact analysis of hydrologic and water quality resource topics will generally be qualitative based on existing information; quantitative information and analyses will be provided where information is available. Specific water resources issues that are anticipated to be of interest to responsible agencies and other stakeholders will be addressed in the impact analysis and include, but are not limited to stormwater drainage runoff effects to San Leandro Creek and communities located downstream of the property and offsite water quality effects associated with construction.

Noise and Vibration

A description of the existing noise environment on and near the Chabot Dam project site, stockpile locations, and haul routes based on existing documentation and reconnaissance-level data will be described. Nearby existing, noise-sensitive receptors (e.g., residential uses adjacent to the off-site haul routes) and existing noise sources (e.g., vehicular traffic on nearby roads like Lake Chabot Road, Estudillo Avenue, MacArthur Boulevard, Benedict Drive, and construction activities) will be identified and discussed. Prior to undertaking the actual analysis, AECOM will meet with EBMUD to discuss potential approaches/methodologies that may be utilized. AECOM will also meet with EBMUD to discuss potential mitigation measures once impacts have been identified.

The noise analysis will include a quantitative description of the existing ambient noise environment within the vicinity of the project site and at the nearest noise sensitive receptors. AECOM will use noise measurements from the Cities of Oakland and San Leandro and Alameda County general plans, noise ordinances, and other relevant studies. Additional noise measurements will not be prepared for the EIR. The “Inverse Square Law of Noise” formula will be used to estimate noise levels at the nearest sensitive receptors from construction equipment which generally translates to a 6 dBA reduction in noise levels for each doubling of distance from the noise source. Existing site conditions and potential shielding that may be afforded from existing structures, topography, vegetation, and natural and man-made berms/barriers will be documented (in one site visit).

Relevant background information, including noise fundamentals, descriptors, and applicable federal, state, and local regulatory framework, will also be described, including the noise/land use compatibility chart from the Alameda County General Plan. Depending on the stockpile locations and on-site and off-site haul routes determined, the General Plans from the Cities of Oakland and San Leandro will also be consulted for noise/land use compatibility.

The analysis will include an assessment of potential short-term, construction-related noise impacts (e.g., on-site heavy-duty equipment, and off-site haul truck traffic) with respect to nearby noise-sensitive receptors and other uses and their relative exposure (considering the effects of shielding and distance). Project-generated noise levels at these receptors will be determined using the reference noise measurement along with standard noise propagation projections. The proximity of the construction activities could adversely affect neighboring residences along the stockpile and haul routes. This potential temporary effect requires strategies to reduce construction noise, including evaluating alternative truck haul routes. Long-term noise impacts are not anticipated as this is a construction project which would be completed upon the seismic upgrade of Chabot Dam.

The technical report will also include an assessment the potential exposure of sensitive receptors to or generation of ground borne vibration from project

implementation (e.g., use of heavy-duty construction equipment). This assessment will include a quantitative description of existing vibration-sensitive receptors (sensitive land uses and structures) compared to expected vibration from construction activities. Mitigation measures will be proposed, as necessary, as part of the technical report.

Recreation

The Project may involve the use of portions of Chabot Park or nearby areas for temporary stockpiling of soil excavated from the dam, and use of roads and pathways in and adjacent to the park for haul roads to and from the dam. In addition, parts of existing trails within Lake Chabot Regional Park in the vicinity of the dam may be used for haul routes, and new haul route roads may be constructed to link the trail segments that will be used as haul routes. AECOM will evaluate the potential for the project to temporarily impact recreational uses and facilities within Chabot Park and Lake Chabot Regional Park during construction, and to cause longer-term impacts to recreation in those areas due to ground disturbance and removal of vegetation.

Primarily based on information from East Bay Regional Parks District and the City of San Leandro on park usage, including volume and use patterns, AECOM will prepare the environmental setting discussion, describing the approximate numbers of recreation visitors of various use-types potentially affected by the project. If adequate information is not available from East Bay Regional Parks District and the City, then an optional field reconnaissance study may be recommended (see more under Optional Task 13).

It is possible that the proposed stockpile locations and haul routes would have impacts on recreation access and activities during the construction period, including potential extended closures of Chabot Park and portions of Chabot Park and Lake Chabot Regional Park trails. AECOM will evaluate the potential impacts on recreation in terms of the locations, facilities, activities and approximate numbers of recreationists likely to be affected by each alternative, taking into account the recreation use information obtained during the field reconnaissance (on-site observations), and the specific details of the proposed stockpile locations and haul routes that are carried forward in the EIR.

AECOM will also evaluate potential impacts to shoreline angling and other shoreline uses and facilities at Lake Chabot, which may occur due to the necessity to reduce the reservoir surface elevation during construction. AECOM will develop a suite of potential mitigation measures to minimize the identified recreation impacts for significant and unavoidable adverse impacts to recreation lands, facilities and uses.

Transportation/Traffic

Construction-related traffic/transportation analysis will include local neighborhood impacts (school, residential safety and lane closures) as well as intersection level of service impacts to existing roadway systems that feed into the area surrounding the project site. Existing traffic conditions will be evaluated by collecting a.m. and p.m. peak hour traffic turning movement counts for up to seven (7) intersections, 24-hour traffic counts at up to three locations, and park-visitor counts for up to five days. Once construction details such as schedule, estimated construction traffic and parking demand are developed, a matrix will be developed that summarizes the estimated construction traffic volume by type of vehicles, by segment and duration. The pros and cons of the alternative haul routes will be evaluated as well as potential impacts on public trails.

The AECOM team will work with EBMUD to develop a preliminary Traffic Control Plan, identifying points of conflict with non-construction activities, and developing alternatives for potential detours, lane closures, and parking displacement. The report will present potential transportation impacts (including increased traffic safety hazards and reduction in roadway capacity) and identify appropriate mitigation measures.

***Deliverables** - Draft and Final versions of the Technical Reports. These will be delivered to EBMUD on a rolling (staggered) schedule and a minimum of two weeks will be allotted for EBMUD review of each report. The schedule for review of technical reports will be submitted to EBMUD for approval, prior to initiation of Task 5. Task 5 shall be completed no later than June 27, 2013. Time is of the essence for this entire Agreement, including but not limited to all deliverables and schedules (See Section 1.2. The project schedule, standards and deadlines are extremely important).*

***Provide** - Draft Reports – one hard copy and one CD that contains the files in Word format for each technical report.*

Final Reports – four hard copies and one CD that contains the files in Word format and searchable PDF format.

Renderings, photos and charts will be submitted in their native format.

Task 6. Prepare Draft EIR and Draft Mitigation and Monitoring Reporting Plan

The AECOM team will prepare an annotated outline of the EIR that identifies individual sections to be prepared, impacts to be addressed, thresholds of significance and methodologies, likely impact conclusions and mitigation strategies, cumulative impact analysis scenario, and alternatives to and of the project.

The Administrative Draft EIR will follow the outline as approved by EBMUD. It will include analysis of the issue areas identified in the IS/NOP as having potentially significant impacts for selected project alternatives, and will include an Introduction, Executive Summary, Project Description, Environmental Setting, Impacts and Mitigation

Measures, analysis of a reasonable range of alternatives, and CEQA-mandated sections such as cumulative impacts, potential growth-inducing impacts of the proposed project and any significant and unavoidable environmental effects. The State Planning Code regulation that exempts EBMUD from specified local government regulations for specified water facilities (California Government Code 53091(d)) shall be clearly described in the EIR as relevant (Executive Summary, and Regulatory Setting for each environmental issue).

The AECOM team will also prepare a draft Mitigation Monitoring Reporting Plan (MMRP) that identifies each full text impact and mitigation measure that must be implemented for each selected project alternative, the agency responsible for assuring compliance, the method of implementation or particular monitoring actions, and the timing of the mitigation. AECOM Project Manager and/or Deputy Project Manager and others as appropriate will attend the EBMUD internal stakeholders meeting where the draft MMRP will be discussed.

Deliverables - *Annotated outline of the EIR, Administrative Draft EIR, and draft Mitigation Monitoring Plan. Attend one meeting.*

Provide - *Five hard copies in three-ring binders and one CD that contains the EIR files in Word format and searchable PDF format.*

Draft EIR

Preparation of the Draft EIR includes preparation of an Initial Draft Screen-Check EIR, and a Draft EIR for Public Review.

The purpose of the Screen-check EIR is to provide EBMUD staff with a final review opportunity immediately prior to printing and releasing the EIR for public review. The Screen-check review is assumed to be an iterative process requiring close coordination with EBMUD staff. To expedite the preparation of the Draft EIR, one or two meetings may be held with EBMUD staff to review EBMUD comments and determine revisions.

For the Draft EIR, AECOM will resolve any final issues from EBMUD's review of the Screen-check Draft EIR to prepare the Draft EIR for public review. EBMUD will be responsible for distribution of the Draft EIR.

AECOM will prepare the Notice of Completion for EBMUD to submit to the State Clearinghouse and the Notice of Availability of the Draft EIR for EBMUD to distribute to the public.

Deliverables - *Initial Draft of the EIR (three hard copies and one CD that contains the EIR files in Word format).
Screencheck EIR (one hard copy and one CD that contains the EIR files in Word format).*

The Draft EIR including two sets of CDs; one that contains electronic files of the Draft EIR, graphics, appendices, and all supporting documentation for the EIR in Word and one with the electronic files in a searchable PDF format that is ready for two-sided copying and three hard copies that include appendices in three-ring binders. A PDF(s) for posting on the web (each not to exceed 25 MG).

Task 7. Public Meetings

AECOM team members will participate in up to three evening public meetings on the Draft EIR during the 45-day public review period. AECOM will prepare and make a presentation of the Draft EIR, and minutes for any meeting attended.

Task 8. Prepare Final EIR and Final MMRP

Response to Draft EIR Comments

AECOM will review the comments received during the public review period on the Draft EIR and identify discrete comments requiring a response. EBMUD will review AECOM's effort to ensure that the most comprehensive approach has been utilized. Individual responses will be prepared for each substantive comment identified. A master response(s) may be prepared if numerous substantive comments are the same or similar. Changes to the Draft EIR text or graphics due to responses to comments will be included as a separate section of the Administrative Final EIR. Two meetings are included in this task, as needed, to review comments and approach to responses.

Deliverable - *Response to Comments document (CD with RTP in Word format for editing).*

Administrative Draft Final EIR

AECOM will prepare the administrative final EIR which will include: an introductory chapter; enumerated comment letters and public hearing comments on the Draft EIR; responses to all comments on the Draft EIR; and a listing of revisions to the Draft EIR. Draft EIR Text will be shown in track changes format to incorporate the comments and responses into the Draft EIR.

Deliverable - *Administrative Draft Final EIR (five hard copies in three-ring binders and one CD that contains the EIR files in Word and searchable PDF).*

Final EIR

AECOM will incorporate one, consolidated set of EBMUD staff comments on the Administrative Draft Final EIR, which together with the Draft EIR, will constitute the Final EIR.

***Deliverable** - Final EIR consisting of (two sets of CDs - one that contains electronic files of the Final EIR, graphics, appendices, and all supporting documentation for the EIR in Word and one with the electronic files in a searchable PDF format that is ready for two-sided copying and three hard copies that include appendices in three-ring binders.)*

A PDF(s) for posting on the web; each not to exceed 25 MG.

Final MMRP

AECOM will finalize the MMRP, and submit it concurrently with the Final EIR to EBMUD.

***Deliverable** - MMRP (one unbound copy ready for double-sided production and one CD with files in original format and searchable PDF format.*

Task 9. Findings Resolution and Statement of Overriding Considerations

AECOM will prepare a Resolution of Findings and Statement of Overriding Considerations (if significant unavoidable impacts are identified), in coordination with EBMUD's Office of General Counsel.

***Deliverable** - Electronic Word file*

Task 10. Presentation to EBMUD Board of Directors

AECOM staff will participate in one public hearing before the EBMUD Board of Directors and prepare a presentation of the EIR findings. AECOM will attend the meeting as a subject matter expert.

Task 11. File Notice of Determination

AECOM will prepare the Notice of Determination (NOD) in coordination with EBMUD's Office of General Counsel. EBMUD will file the NOD with the County Clerk and State and would pay any associated fees.

***Deliverable** - Electronic Word file*

Task 12. Project Management

The AECOM Project Director/Project Manager, Marcia Tobin, will oversee the team, directing preparation of the environmental analyses, and managing quality control of the environmental document. Richard Hunn will provide strategic CEQA guidance and direction to the team. Susan Yogi and Marie Galvin will work on the EIR sections, integrating the work of each of the individual authors. Robert Kirby will lead the Alternatives Haul Route Constructability Review, and Gustavo Arboleda will lead the

Outlet Works Constructability Review. Bi-weekly progress meetings will be held with EBMUD's project manager by phone or in person.

OPTIONAL SERVICES

Task 13. Additional Environmental Services

These services could require additional effort by AECOM:

- To address any comments over the 100 individual comments assumed in AECOM's cost estimate to prepare the Response To Comments for the EIR.
- To conduct Recreation Field Reconnaissance.

Adequate evaluation of Project impacts on recreation uses will require information on the types, volume and frequency of uses occurring in Chabot Park and on the affected trails at various times. Park managers will be interviewed and available information gathered on these topics. To gather more detailed information AECOM will perform on-site observation of recreation use to obtain the information as part of the Field Reconnaissance conducted during the Constructability and Environmental Review. Observation will note and describe the activities occurring and obtaining approximate counts of the number of park visitors in total and the number of participants in specific activities within the park and on relevant trail segments. The intent is to obtain a defensible and objective representation of recreation activity potentially impacted by the defined project area, not a statistically significant sample of overall park and trail use. Observation would occur during the months of April through October, the period that construction activities would be occurring; this could be extended to additional months if the construction schedule is extended and if significant recreation use is present. Observation would primarily occur on weekends and holidays, when use would be expected to be greatest, but would also occur on weekdays to provide comparative data. The Field Reconnaissance schedule would include 1 weekend day or holiday per month and 3 weekdays interspersed during the April – October period of 2013 (approximately a 10-days total, with each visit lasting about 4 hours). Information on the number of participants in special events that may be affected by the Project, such as the Lake Chabot Trail Challenge running event, would be obtained from event organizers.

- To conduct additional Noise and Vibration Analysis.

If EBMUD determines a need for additional analytical effort to support the conclusions of the noise analysis, noise measurements may be taken. These would be completed in accordance with American National Standards Institute (ANSI) standards.

Task 14. Permitting Support during the Project Design Phase

The AECOM team is prepared to support EBMUD with the preparation of permits and authorizations including U.S. Army Corp of Engineers (USACE) Clean Water Act (CWA) Section 404 Nationwide Permit (NWP), San Francisco Bay Regional Water Quality Control Board (RWQCB) CWA Section 401 Water Quality Certification, California Fish and Game (CDFG) Code Section 1602 Lake or Streambed Alteration Agreement, USFWS federal Endangered Species Act (ESA) Section 7 consultation, City of Oakland Code of Ordinances Section 12.36.040 Tree Removal Permit and an Alameda County Tree Ordinance Section 12.11.150 Site Specific Permit for tree removal within the county right-of-way.

AECOM will complete a Joint Aquatic Resources Permit Application (JARPA) for the USACE CWA Section 404 NWP, RWQCB CWA Section 401 Water Quality Certification, and CDFG Code Section 1602 Notification of Lake or Streambed Alteration. Additionally, AECOM will complete a Biological Assessment to support ESA Section 7 consultation with the USFWS, as well as all required associated materials and documentation. AECOM assumes that EBMUD will provide project design drawings sufficient to meet the requirements of all permit applications and that EBMUD will provide all required permit application fees. This scope of work assumes that if regulatory agencies require mitigation for project impacts EBMUD will participate in an agency approved mitigation/conservation bank and preparation of a mitigation monitoring plan is not included in this scope of work.

AECOM assumes that no project-specific Biological Opinion will be required and that the project mitigation approaches will conform to the EBMUD April 2008 Low Effect East Bay HCP. AECOM assumes that no California Endangered Species Act (CESA) Section 2081 Incidental Take Permit or CESA Section 2080.1 Consistency Determination will be required for the project; however, additional support related to compliance with CESA may be provided at additional cost, as needed.

Because the project will involve land disturbance of greater than an acre and discharge groundwater into San Leandro Creek, compliance with the National Pollution Discharge Elimination System (NPDES) General Permit for Storm Water Discharges Associated with Construction Activities and Land Disturbance Activities (Order No. 2010-0014-DWQ [NPDES CGP]) will be required. AECOM assumes that EBMUD or its authorized contractors will prepare a Notice of Intent (NOI) for compliance with the NPDES CGP; however, AECOM can assist with the NOI at additional cost, as needed.

The preparation of permit applications and associated materials can occur concurrently with Task 5.

Task 15. Mitigation and Monitoring Support during the Project Construction Phase

AECOM will provide a biological monitor to assure compliance with environmental protection measures, commitments, and mitigation conditions contained in the EIR, the

adopted MMRP, permit conditions, and regulatory approvals. The biological monitor would be on site as needed to observe construction activities and implementation of mitigation measures in the vicinity of and would prepare monitoring reports for submittal to EBMUD and the appropriate resource agencies.

II. PROJECT SCHEDULE

Time is of the essence for this entire Agreement including but not limited to all deliverables by AECOM. See Section 1.2.

Notice to Proceed	10/24/12
Finalize Constructability/Environmental/Outlet Works Report	02/28/13
Finalize Initial Study	03/20/13
Issue Notice of Preparation	04/17/13
Technical Environmental Reports Final	06/27/13
Finalize Draft EIR	10/08/13
Draft EIR Public Meetings (up to 3)	11/5/13 – 11/7/13
Final EIR Completed	03/03/14
Findings Resolution and Statement of Overriding Considerations	03/10/14
Presentation to EBMUD Board of Directors	03/18/14
File Notice of Determination	03/19/14

EXHIBIT B

East Bay Municipal Utility District Chabot Dam Seismic Upgrade Project

COMPENSATION

Compensation for services provided in Exhibit A, SCOPE OF SERVICES, shall be in accordance with the methods and specific amounts described in this Exhibit.

1. DISTRICT shall pay CONSULTANT only the actual costs incurred, subject to the agreed cost ceiling. CONSULTANT certifies that the cost and pricing information used herein are complete, current and accurate. CONSULTANT acknowledges that it will expend public funds and hereby agrees to use every appropriate method to contain its fees and minimize costs under this Agreement.
2. Compensation for CONSULTANT services authorized shall be on a cost reimbursement basis and include Direct Labor, Indirect Costs, Subconsultant Services, Other Direct Costs, and a Professional Fee. Costs to be paid comprise the following:

2.1 Direct Labor

Direct labor costs shall be the total number of hours worked on the job by each employee times the actual hourly rate for such employee's labor. Hours worked shall be rounded-up to the nearest quarter-hour (0.25) increment. Labor costs for principals shall be based upon the actual hourly rate of pay for those individuals. Labor rates shall be based on a normal 8-hour day, 40-hour week. DISTRICT will pay all personnel at their regular rate including any work performed on overtime or on holidays or weekends.

2.2 Indirect Costs

DISTRICT shall pay CONSULTANT an overhead expense equal to one-hundred ninety (190) percent of labor costs incurred by CONSULTANT. CONSULTANT acknowledges and agrees that this overhead compensation is in lieu of itemized payments for indirect and overhead expenses which includes, but is not limited to:

- Clerical, word processing and/or accounting work.
- Vehicle usage and mileage between CONSULTANT's office and DISTRICT service area.
- Parking (DISTRICT does NOT provide parking to CONSULTANT in the DISTRICT Administration Building, located at 375 11th Street, Oakland, California. CONSULTANT shall be responsible for parking elsewhere).

- Postage, or for certified or registered mail. Extraordinary postage, overnight delivery, or messenger delivery charges must be approved in advance.
- Routine copying costs for in-house copying.
- Local telephone charges, including cellular phone, modem and telecopier/FAX charges.
- Office space lease.
- Office supplies.
- Computer equipment.
- Computer usage charges.
- Books, publications and periodicals.
- Insurance.
- Miscellaneous hand tools or equipment rental.
- Safety training, seminars or continuing education.
- Utilities.
- Meals, transportation or other charges.
- Inadequately described or miscellaneous expenses.

The above items are illustrative, rather than exhaustive.

2.3 Subconsultant Services

Subconsultant services shall be billed at cost (plus a five (5) percent markup).

2.4. Other Direct Costs

Other Direct Costs shall be approved by DISTRICT in advance in writing, and shall be billed at cost, without markup. These costs include, but are not limited to the following:

1. Automobile expenses at fifty-five and one-half (55.5) cents per mile when CONSULTANT is required to travel outside of the DISTRICT's service area. Mileage will NOT be reimbursed for rental car expenses, where the rental agreement specifies unlimited mileage.
2. DISTRICT will pay for necessary and reasonable travel expenses provided the travel is approved in advance by DISTRICT Project Manager, and providing that:
 - Each expense is separately identified (air fare, hotel, rental car) with an amount and date incurred. Confirming documents may be requested.
 - Charged mileage for vehicle mileage shall not exceed the current allowable Internal Revenue Service rate.
 - Air travel is coach or economy rate for refundable tickets. Business and first class rates will not be reimbursed.

- Lodging accommodations are moderately priced.
- Meal charges are reasonable. (Reimbursement for meals will only be made in conjunction with out-of-town travel.)
- Taxis or shuttles are used rather than rental cars whenever cost effective.
- Rental cars are intermediate or compact class only.

2.5 Professional Fee

As a portion of the total compensation to be paid to CONSULTANT, DISTRICT shall pay the Professional Fee, subject to the agreed Professional Fee Ceiling of \$59,040 as specified in Exhibit B-1, as profit for services rendered by CONSULTANT covered by this Agreement. CONSULTANT shall earn the Professional Fee based on a ten (10) percent markup of CONSULTANT's Direct Labor and Indirect Costs billed and approved.

2.6 Budget Amounts

	Contracted Services	Optional Services	Maximum Services*
Cost Ceiling	\$721,628	\$222,807	\$944,435
Professional Fee Ceiling	\$ 38,168	\$ 21,431	\$ 59,598
Agreement Ceiling	\$759,795	\$244,238	\$1,004,033

* *Maximum Services is the sum of Contracted and Optional Services.*

The Cost Ceiling shown above is based upon the cost estimate and labor hours attached hereto as Exhibit B-1 and Exhibit B-2. Costs described above, comprising Direct Labor, Indirect Costs, Subconsultant Services, Other Direct Costs, and Professional Fee, shall be payable up to the Agreement Ceiling as specified herein.

2.7 Billing and Payment

CONSULTANT shall invoice DISTRICT monthly for the actual costs incurred and a prorated Professional Fee for work performed during the previous month. Actual costs shall include Direct Labor, Indirect Costs, Subconsultant Services, and Other Direct Costs as specified herein. Actual costs shall be invoiced by task as described in Exhibit A. Invoices shall set forth a description of the actual costs incurred and the services performed, the date the services were performed and the amount of time spent rounded to the nearest quarterly hour increment (.25) on each date services were performed and by whom. Supporting documentation for the invoice shall be organized to clearly identify the task charged and shall be supported by such copies of invoices, payroll records, and other documents as may be required by DISTRICT to authenticate invoiced costs. Copies of all invoices from any subconsultant(s) and outside service(s) shall be attached.

DISTRICT shall pay CONSULTANT within thirty (30) days, upon receipt of a proper CONSULTANT invoice, provided that all invoices are accompanied by sufficient cost documentation, and DISTRICT Form P-47 (Subcontractor Payment Report - CEP Participation), to allow the determination of the reasonableness and accuracy of said invoice.

A ceiling price is in effect for the entire Scope of Services. If the authorized Agreement Ceiling, including the authorized Professional Fee Ceiling, is reached, CONSULTANT shall complete the agreed-upon work for the authorized Agreement Ceiling. Labor hours may be reallocated within the tasks without renegotiation of the Agreement with written approval from the DISTRICT Project Manager in such a manner so as not to exceed the Agreement ceiling price. In no event shall the Cost Ceiling of the Agreement or the Professional Fee Ceiling be increased unless there is a written amendment of this Agreement.

2.8 Budget Status Reports

For the duration of this Agreement, the CONSULTANT shall provide DISTRICT with monthly budget status reports that include, in tabular or graphical format, for each report period: (1) the original cumulative projected cash flows for the duration of the project (prepared at the start of the project), (2) the actual cash flows for the work completed to date, (3) the current projected cash flows to complete the project, and (4) the earned value (the amount of work actually completed to date compared to the budget expended). Current projected cash flows shall be based on all CONSULTANT and subconsultant time sheets up to a date within 3 weeks of the date of the budget status report.

EXHIBIT B-1

East Bay Municipal Utility District
Chabot Dam Seismic Upgrade Project

Name Firm Title	AECOM										Ninyo and Moore	
	AECOM Direct Labor Costs	AECOM OH (Indirect Costs)	Other Direct Costs	Professional Fee (10%)	Total AECOM Cost	Sustain Environmental (includes 5% markup)	CHS Consulting (includes 5% markup)	Burleson Consulting (includes 5% markup)	Underwater Resources (includes 5% markup)	WRE (includes 5% markup)		Terra Engineers
1. Project Initiation												
Subtotal Hours					224	6	15	12	14	18	76	8
Cost	\$ 10,967	\$ 20,838		\$ 3,181	\$ 34,986	\$ 864	\$ 3,370	\$ 1,829	\$ 1,987	\$ 4,079	\$ 14,513	\$ 1,261
2. Constructability Review and Environmental Review of Alternative Haul Routes and Outlet Works Construction												
Subtotal Hours					246	0	168	0	42	200	344	0
Cost	\$ 13,114	\$ 24,916	\$ -	\$ 3,803	\$ 41,832	\$ -	\$ 23,335	\$ -	\$ 5,961	\$ 33,257	\$ 67,999	\$ -
3. Finalize Initial Study												
Subtotal Hours					62	0	6	0	0	0	0	0
Cost	\$ 3,200	\$ 6,080		\$ 928	\$ 10,209	\$ -	\$ 1,184					
4. Notice of Preparation of EIR												
Subtotal Hours					42	0	0	0	0	0	0	0
Cost	\$ 2,005	\$ 3,809		\$ 561	\$ 6,394	\$ -	\$ -					
5. Prepare Technical Environmental Reports												
Subtotal Hours					1,140	36	118	192	0	60	110	138
Cost	\$ 42,720	\$ 81,167		\$ 12,389	\$ 136,275	\$ 4,713	\$ 17,507	\$ 20,700	\$ -	\$ 10,758	\$ 16,979	\$ 17,262
6. Prepare Draft EIR and Draft Mitigation and Monitoring Reporting Plan (MMRP)												
Subtotal Hours					509	0	101	125	0	0	33	84
Cost	\$ 24,576	\$ 46,694	\$ -	\$ 7,127	\$ 78,396	\$ -	\$ 14,524	\$ 13,081	\$ -	\$ -	\$ 7,798	\$ 10,814
7. Public Meetings												
Subtotal Hours					88	0	12	0	0	8	8	0
Cost	\$ 4,757	\$ 9,038	\$ -	\$ 1,380	\$ 15,175	\$ -	\$ 4,007	\$ -	\$ -	\$ 1,813	\$ 1,890	\$ -
8. Prepare Final EIR and Final MMRP												
Subtotal Hours					253	0	80	40	8	16	22	12
Cost	\$ 12,166	\$ 23,115	\$ -	\$ 3,528	\$ 38,809	\$ -	\$ 10,332	\$ 4,649	\$ 1,136	\$ 3,058	\$ 5,198	\$ 1,512
9. Findings Resolution and Statement of Overriding Considerations												
Subtotal Hours					13	0	0	0	0	0	0	0
Cost	\$ 707	\$ 1,343	\$ -	\$ 205	\$ 2,254	\$ -	\$ -	\$ -	\$ -	\$ -	\$ -	\$ -
10. Presentation to EBMUD Board of Directors												
Subtotal Hours					32	0	0	0	0	0	0	0
Cost	\$ 1,832	\$ 3,480	\$ -	\$ 531	\$ 5,843	\$ -	\$ -	\$ -	\$ -	\$ -	\$ -	\$ -
11. File Notice of Determination												
Subtotal Hours					11	0	0	0	0	0	0	0
Cost	\$ 568	\$ 1,079	\$ -	\$ 165	\$ 1,811	\$ -	\$ -	\$ -	\$ -	\$ -	\$ -	\$ -
12. Project Management												
Subtotal Hours					282	0	48	0	0	0	0	0
Cost	\$ 15,003	\$ 28,505	\$ 4,743	\$ 4,351	\$ 52,802	\$ -	\$ 7,837	\$ -	\$ -	\$ -	\$ -	\$ -
Total Labor Hours					2,902	42	548	369	64	302	577	242
Total	\$ 131,612	\$ 250,063	\$ 4,743	\$ 38,168	\$ 424,586	\$ 5,577	\$ 82,096	\$ 40,259	\$ 9,084	\$ 52,965	\$ 114,378	\$ 30,850
Total Costs by Subconsultant without 5% Markup												
					\$ 5,311		\$ 78,187	\$ 38,342	\$ 8,652	\$ 50,443	\$ 105,318	\$ 29,381

EXHIBIT B-1

**East Bay Municipal Utility District
Chabot Dam Seismic Upgrade Project
Optional Tasks**

Optional Tasks

	Direct Labor	OH	Other Direct Costs	Professional Fee	Total
13. Additional Environmental Services					
Subtotal Hours					355
Cost	\$ 15,894	\$ 30,199	\$ 1,000	\$ 4,609	\$ 51,702
14. Permitting Support during the Project Design Phase					
Subtotal Hours					780
Cost	\$ 30,549	\$ 58,043	\$ 2,500	\$ 8,859	\$ 99,951
15. Mitigation and Monitoring Support during the Project					
Subtotal Hours					1,040
Cost	\$ 27,456	\$ 52,166	\$ 5,000	\$ 7,962	\$ 92,585
Total Hours					2,175
Total Cost	\$ 73,899	\$ 140,408	\$ 8,500	\$ 21,431	\$ 244,238

EXHIBIT C

**East Bay Municipal Utility District
Chabot Dam Seismic Upgrade Project**

CEP COMPLIANCE

	Amount	Percent of Total Fee \$759,795
WBE		
Sustain Environmental	\$ 5,311	0.7%
Terra Engineers	\$105,318	13.9%
	\$110,029	14.6%
MBE		
CHS Consulting	\$ 78,187	10.6%
Burleson Consulting	\$ 38,342	5.2%
WRE	\$ 50,443	6.8%
Ninyo & Moore	\$ 29,381	4.0%
	\$196,352	26.6%

Based on a Contracted Ceiling Amount of \$759,795.