**Pacific Gas and Electric Company**

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September 13, 2007

Mr. Takeshi Yamashita, Regional Engineer
San Francisco Regional Office
Federal Energy Regulatory Commission
901 Market Street, Suite 350
San Francisco, CA 94103-1778

Re: Project No. 2105-CA, Upper N.F. Feather River
Lake Almanor Dam, NATDAM NO. CA00327

Dear Mr. Yamashita:

As follow up to our letter dated July 11, 2007, this letter is to provide an update and proposed resolution for addressing the seismic analysis of the outlet tower at Canyon (Lake Almanor) Dam.

As noted in our July 11, 2007 letter, PG&E has concluded the inlet gates will not be damaged and the tower will remain functional after the design earthquake event without modification except for the need to strengthen the roof diaphragm of the tower structure.

Enclosed are three copies each of the following reports for the seismic analyses of the outlet tower:

1. "Lake Almanor Intake Tower, Plumas County – California: Seismic Analysis with Base Rocking", prepared by Button Engineering, dated July 2007.

2. "Lake Almanor Intake Tower, Seismic Evaluation: Pushover Analysis", prepared by Anatech Corp., dated August 2007.

In addition we are also including three copies of a letter from our board of consultants on the seismic qualification of the tower, entitled "The Board of Consultants to Pacific Gas and Electric Company on the Lake Almanor Intake Tower, Almanor, CA", dated August 26, 2007. Our board of consultants consists of professors Mete Sozen, Anil Chopra and Skip Hendron who are experts in the field of civil engineering and are internationally known in the seismic analysis and design of earthquake resistant structures.

The board of consultants' comment letter is based on the review of the two above analysis reports. The qualification criteria for the inlet tower are based on the satisfactory operational performance of the inlet gates when subjected to the required simulated earthquake event. The design earthquake was based on the $50^{th}$ percentile of MCE at the site as approved previously by FERC.

It is the opinion of the board that the seismic analysis as described in the above reports is an accurate and appropriate approach to this highly non linear structure due to discontinuity of the rebar at the base at Elev. 4422 ft. The operating deck of the valve house is at Elev. 4415 ft. Lake Almanor intake tower is a lightly reinforced concrete structure originally built in 1913 to Elev. 4465 and later raised to Elev. 4544 ft in 1927.

The SAP2000 analysis described in Report 1 above is a time history analysis in which the fluid was explicitly modeled with three dimensional finite elements. The SAP2000 model accurately represents the fluid structure interaction under the seismic ground motions. Uplift at the base is allowed because of lack of the rebar continuity into the mass concrete below Elev. 4422 ft. The material non linearity was considered using recommendations specified by FEMA 356 by adjusting the Young's modulus.

Using SAP2000 model, three time history analyses were performed to predict an upper bound lateral displacement (target displacement) at the valve house deck level. Three separate analysis runs were made under different combinations of independent design earthquake time histories to obtain the target displacement. Using this procedure, a target displacement of 5.2 inches was determined at the operating deck level with a maximum uplift of less than 2 inches at the base of the buttresses. The analysis also indicated that most of the target displacement at the deck level was contributed by the rigid body motion (RBM) caused by rocking. This RBM keeps the valve blade and its stem more or less in a straight line with little deformation in the valve system.

It was also determined that the seismic forces (shears and moments) are greatly reduced in the tower under the design earthquake if the uplift is allowed at the base. Our original finite element models assumed a continuity of the rebar at the base of the tower until a close examination showed otherwise. Therefore the present model which has been used in the two analysis reports are more realistic than the previous models/analyses that we submitted to FERC.

The Sap2000 model was considered adequate for displacements but not for the stresses as it lacks the material non linearity. To predict the stresses and the deformations more accurately around the gate openings, a push over analysis was performed using a non linear ABAQUS model as described in Report 2 above. The ABAQUS model includes material non linearity and a compressive strength of 3000 psi was used for the old 1913 concrete. The rebar was modeled accurately in the ABAQUS model with truss elements and the concrete was modeled with solid elements.

The ABAQUS model was also allowed to uplift at the base and the model was pushed using a static push-over load as well as an inertial loading for comparison purposes. The results of these two independent loadings were close to each other and the deformations in the gate openings remained negligible.

The ABAQUS model allowed a redistribution of the forces, and the stresses remained within the specified compressive and tensile stresses. The deformations around the gate openings were plotted and closely examined to assure the operability of the gates after the earthquake. The maximum deformations in the gate openings are less than 0.05 inches under the full target displacement of 5.2 and 4.9 inches the strong and weak axes of the tower, respectively. The gates will be able to operate under the anticipated seismic deformations because the gate blades are ½ inch smaller than the gate openings and can easily absorb 0.05 inch deformations.

The comments in the board of consultants' letter addressed the performance of the Lake Almanor outlet tower under the design earthquake loading. Based on the results contained in the two analysis report, it is the opinion of the board that the inlet gates will not be damaged and the tower will remain functional after the design earthquake event provided the roof diaphragm is strengthen.

As previously suggested in our letter of July 11, 2007, a joint meeting with FERC and DSOD can be scheduled to review the basis of our analyses and conclusions if needed. I will contact you in early November to coordinate selection of a meeting date should you deem necessary.

If you have any questions or need additional information, please call me at (415)-973-3076, or Dr. Mohammad Aslam at 415-973-3322.

Sincerely,

Wing Lee

Wing H. Lee
Facility Safety Coordinator

Enclosure

THE BOARD OF CONSULTANTS TO THE PACIFIC GAS & ELECTRIC COMPANY
ON THE LAKE ALMANOR INTAKE TOWER, ALMANOR, CA


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26 August 2007

Dr. Mohammad Aslam
Pacific Gas & Electric Company
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SUMMARY

Lake Almanor Intake Tower is a lightly reinforced concrete structure with its base at elevation 4422 and the top of the concrete portion at elevation 4515 (Fig. 1 and 2). It serves to regulate the level of Lake Almanor in Plumas County, CA.

This report documents the opinion of the Board on the stability and the structural integrity of the Lake Almanor Intake Tower based on two engineering reports. The first report, by Button Engineering of Austin, TX, focused on the limits of dynamic response of the Lake Almanor Intake Tower for strong ground motions defined by Pacific Gas & Electric based on a detailed study of the seismicity of the Lake Almanor region (Fig.8-10). The second report, by ANATECH of San Diego, CA, investigated the effects of the lateral-displacement maxima established in the first study on the stability and structural integrity of the Intake Tower.

For the proportions of the Intake Tower and the characteristics of the specified ground motions, we find the bounds to the rocking motion (a maximum lateral drift of 5.2 in. at elevation 4515 corresponding to a drift ratio of 0.5%) determined by Button Engineering to be credible. The analysis by ANATECH as well as the one by Button demonstrated that the structure will respond to the rocking essentially as a rigid body. The calculated changes in the gate-opening dimensions did not exceed 0.05 in. (original length 118 in.) in the diagonal and 0.015 in. (original length 108 in.) in the vertical directions.

From the results of the studies by Button Engineering and ANATECH, we conclude that the tower will be stable and the gates will not lose their functionality in the event of any one of the prescribed design earthquakes.

**Lake Almanor Dam Intake Tower**

The Almanor Dam intake tower (on Lake Almanor, CA) serves to regulate the level of the lake and to release water into the north fork of the Feather River to support the fish habitat. The tower comprises a reinforced concrete segment that is seated on a concrete base excavated in rock at elevation 4422 and extends up for 93 ft to elevation 4515 and an operating house that rests on the reinforced concrete segment with the ridge of its roof at elevation 4544 (Fig.1).

The reinforced concrete tower structure has nine 24-in. thick buttresses extending out from a cylindrical core (Fig.2) with buttresses at 40 deg. to one another. The dimensions edge-to-edge of the buttresses are approximately 46 ft on axis X, the "strong" axis, and 40 ft on axis Y, the "weak" axis. The cylindrical core varies in thickness with height as described in Fig.2 through 6. The inside diameter is constant at 16 ft. Construction of the tower was initiated in 1914 when the concrete segment was built to elevation 4465. It was extended up in 1927 to reach 4515 ft with the control house adding another 29 ft of height (Fig.1).

The tower has three 4-ft wide gates with their bases at elevation 4422. The height of each gate opening is 9 ft at the outer surface of the tower core and 6 ft at the inner surface. As shown in Fig. 7, they are located between buttresses B2 and B1, B1 and B9, and B9 and B8. These gates are to remain functional after the design earthquake.

Based on the two studies of the Lake Almanor Intake Tower listed below, we considered the stability of the tower and the level of distortion of the gate openings.

1. Martin Button, "Lake Almanor Intake Tower, Plumas County California: Seismic Analysis with Base Rocking," Button Engineering, 4701 Shoal Creek Boulevard, Austin, TX 78756, July 2007.
2. Daniel Parker, "Lake Almanor Intake Tower Seismic Evaluation: Pushover Analysis," Report ANA-R-07-0719, August 2007, ANATECH Corporation, 5435 Oberlin Drive, San Diego, CA 92121.

The first study reports dynamic-response analyses of the Intake Tower. A three-dimensional finite-element model of the Intake Tower is used. The model uses linear elements to represent the concrete and includes reservoir effects. It has gap elements that respond only in compression at elevation 4422 to permit rocking of the tower structure.

The second study was carried out to investigate the integrity of the concrete structure and to determine the possible changes in the dimensions of the gate openings with the tower displaced to the maximum limit determined by the dynamic-response analyses by Button. It involved three different static analyses of a three-dimensional nonlinear model of the tower with a discontinuity at elevation 4422: (1) Lateral load applied at elev. 4453 in direction X, (2) lateral load applied at elev. 4453 in direction Y, and (3) linearly increasing acceleration in direction X applied at base.

The first two analyses approximated the ratio of base moment to base shear in the analyses by Button Engineering while developing lateral displacements equal to those obtained by Button. The third analysis was made to confirm the essentially rigid-body response of the intake tower, observed in the first two analyses as well as in those by Button, subjected to inertia forces throughout its height.

**Earthquake Ground Motions**

In 1966, Pacific Gas and Electric Company completed a detailed review of the seismicity of the Lake Almanor Dam region. Based on that review, two different sets of base excitation were specified for evaluation of the seismic response of the intake tower: a uniaxial motion with a peak acceleration of 0.64g and a biaxial motion having components designated N75E and N15W with peak accelerations comparable to the uniaxial motion. Characteristics of these motions are illustrated in Fig. 8-10 reproduced from the report by Button Engineering.

**Estimated Tower Displacements**

Button Engineering considered four cases of ground motion:

Case 1: Uniaxial motion with $A_{max}$ = 0.64g applied along axis X (strong axis) of the Intake Tower.

Case 2: Uniaxial motion with $A_{max}$ = 0.64g applied along axis Y (weak axis) of the Intake Tower.

Case 3: Biaxial Motion with N75E applied along axis X (strong axis) and N15W applied along axis Y (weak axis).

Case 4: Biaxial Motion with N15W applied along axis X (strong axis) and N75E applied along axis Y (weak axis).

The dynamic-response results obtained by Button Engineering demonstrated that the Intake Tower will rock but will remain stable in all of the four cases considered. The displacement-demand maxima at elevation 4515 are summarized in Table 1.

The maximum response displacement calculated at elevation 4515 was 5.2 in. (Case 3) corresponding to a drift ratio of approximately 0.5%. Almost 90% of the lateral displacement calculated was due to rotation of the tower as a rigid body.

The reinforced concrete segment of the Intake Tower has a minimum base dimension of 40 ft and a height of 93 ft. The specified ground displacement is smaller than 2% of the minimum base dimension. In our opinion, the response limits determined by Button Engineering are reasonable and confirm that, for the ground motions specified, there is no likelihood of the Intake Tower becoming unstable.

**Distortion of The Gates**

To obtain a measure of the distortion of the gates, we refer to the study by ANATECH. In two of the analyses included in this study, the tower was displaced by a force applied at 31 ft above elevation 4422 in order to approximate the base moment/shear ratios calculated in the Button Engineering study. The structural model was nonlinear with the 1914 concrete assumed to have a compressive strength of 3000 psi. The concrete cast in 1927 was assumed to have a compressive strength of 4000 psi.

The ANATECH study established that the changes in the dimensions of the gate openings will be of trivial magnitude at the maximum lateral displacement anticipated for the Intake Tower. The maximum change in the diagonal was found not to exceed 0.05 in (nominal original length 118 in.) and the maximum change in the vertical was found to be 0.015 in. (nominal original length 108 in.)

There is no reason to expect loss of function of the gate mechanisms related to distortion of the gate openings in the event of the specified ground motions.

**Calculated Concrete Strains**

Contours referring to vertical strains calculated at a drift of 5.1 in. at elev. 4515 (drift ratio of approx. 0.5 %) are shown in Fig. 11 for motion along axis X and in Fig. 12 for motion along axis Y. The distributions along the height of the compressive strain calculated at the extreme fiber are shown in Fig. 13 and 14. (All four figures are reproduced from the ANATECH Report). At that drift, the structure is supported over a short length at the extremities of buttresses that are in compression. As would be expected, high vertical strains are calculated at these points accompanied by high strain gradients in both the axial and transverse directions. In the rest of the structure, the calculated strains are quite low.

The maximum calculated concrete strains in the vertical direction were 0.006 (at a drift of 5.2 in. in the X or strong direction) and 0.008 (at a drift of 5.1 in. in the Y or weak direction.) In a region with strong strain gradients and bearing on material that has smaller stress, calculated strains of this magnitude do not threaten the integrity of the structure. .

**Concluding Remarks**

The critical issues in the performance of the Lake Almanor Tower are the stability of the tower and the functioning of the gates after the design earthquake. The study by Button Engineering has demonstrated that the tower will remain stable and its lateral drift, related almost entirely to rocking essentially as a rigid body, at an elevation of 4515 will be 5.2 in. (a drift ratio of approximately 0.5 %). The calculated changes in the gate-opening dimensions did not exceed 0.05 in. (original length 118 in.) in the diagonal and 0.015 in. (original length 108 in.) in the vertical directions.

The results of the two studies combine to lead to the conclusion that the Lake Almanor Intake Tower will be functional after any one of the design earthquakes. The analyses by Button Engineering and ANATECH indicate that the tower will remain stable and the gate distortions will be negligible for the ground-motion demands considered. We find the analyses well reasoned and their conclusion acceptable.

TABLE 1 Dynamic Response Determined by Button Engineering

| | | Case 1 | Case 2 | Case 3 | | Case 4 | |
|---|---|---|---|---|---|---|---|
| | | Axis X | Axis Y | Axis X | Axis Y | Axis X | Axis Y |
| Drift at 4515 | in. | 2.7 | 4.1 | 5.2 | 4.9 | 3.7 | 3.7 |
| Drift Ratio | % | 0.24 | 0.37 | 0.47 | 0.44 | 0.33 | 0.33 |

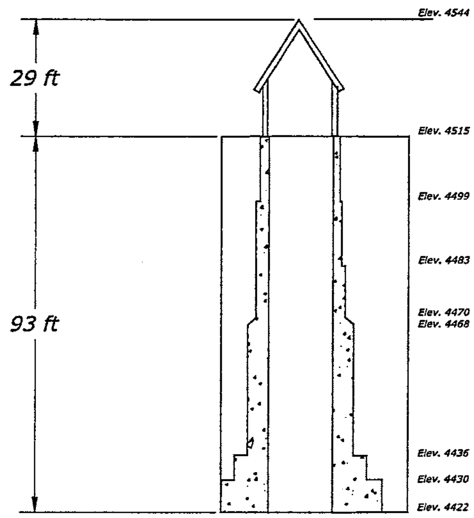



Figure 1 Section of Lake Almanor Intake Tower

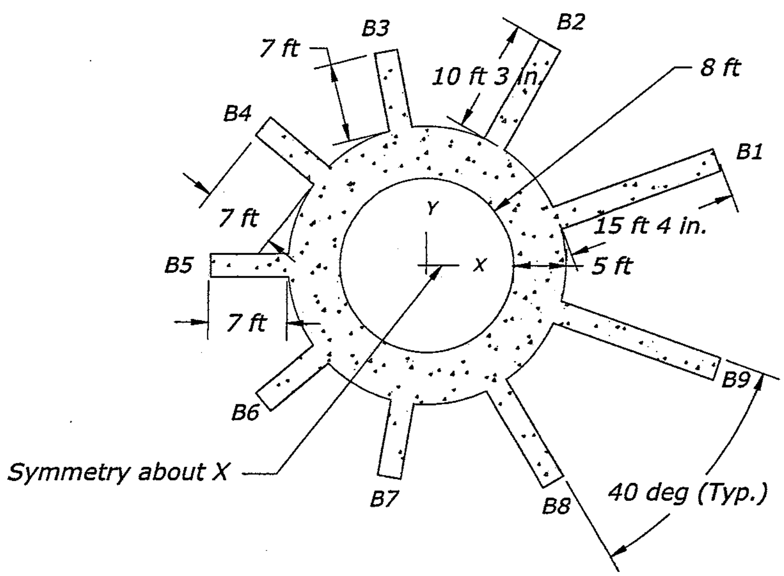



Figure 2 Section from Elevations 4422 to 4468

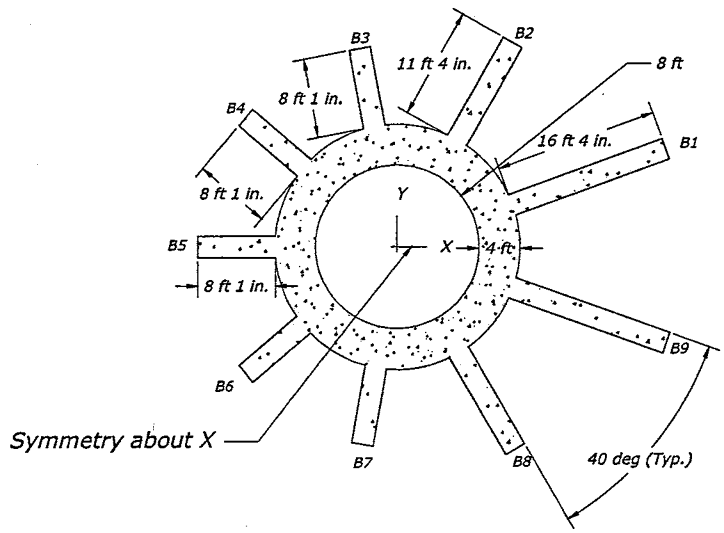



Figure 3 Section from Elevations 4468-4470

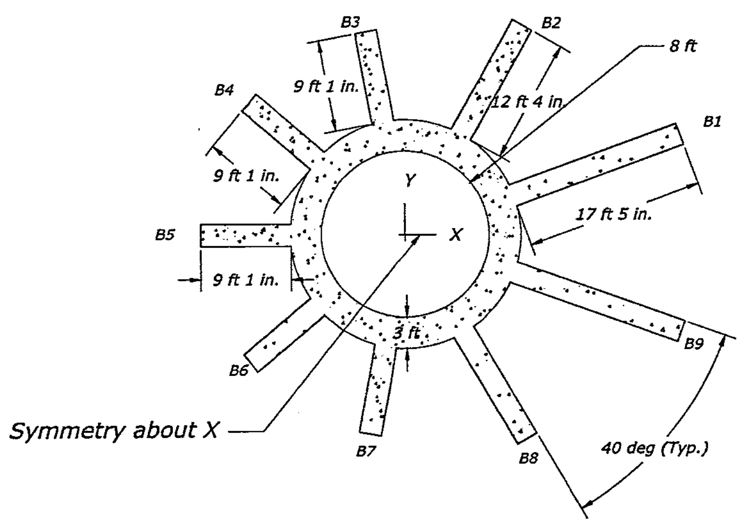



Figure 4 Section from Elevations 4470 to 4483

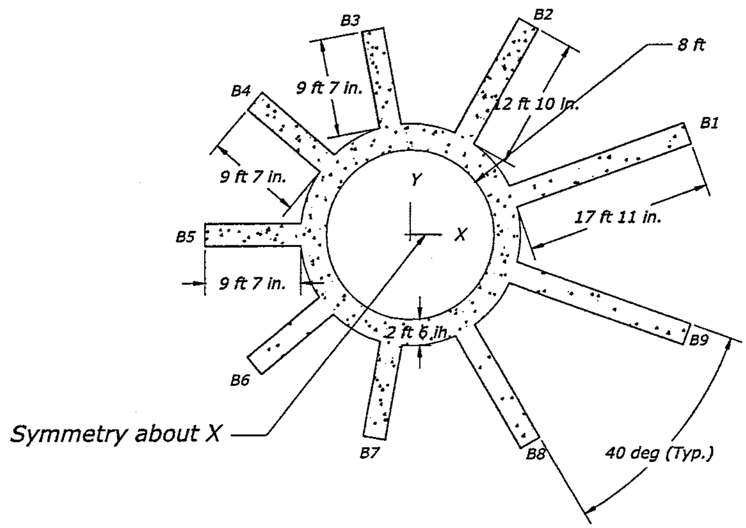



Figure 5 Section from Elevations 4483 to 4499

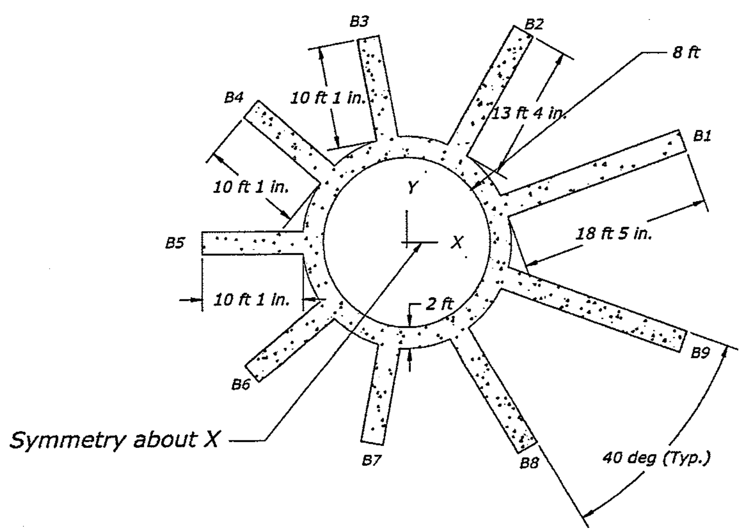



Figure 6 Section from Elevations 4499 to 4515.

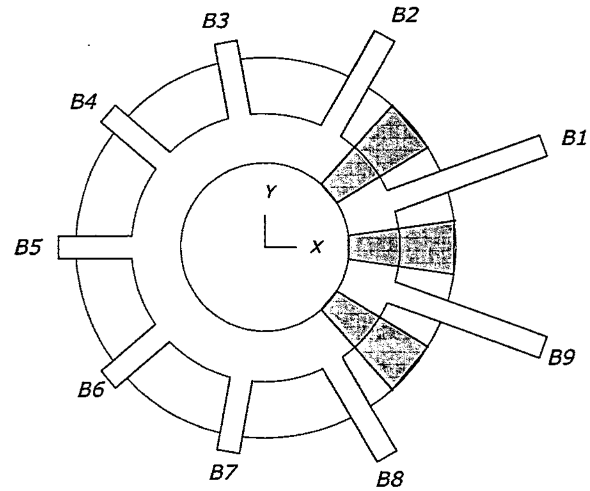



Figure 7 Section Showing Gate Locations

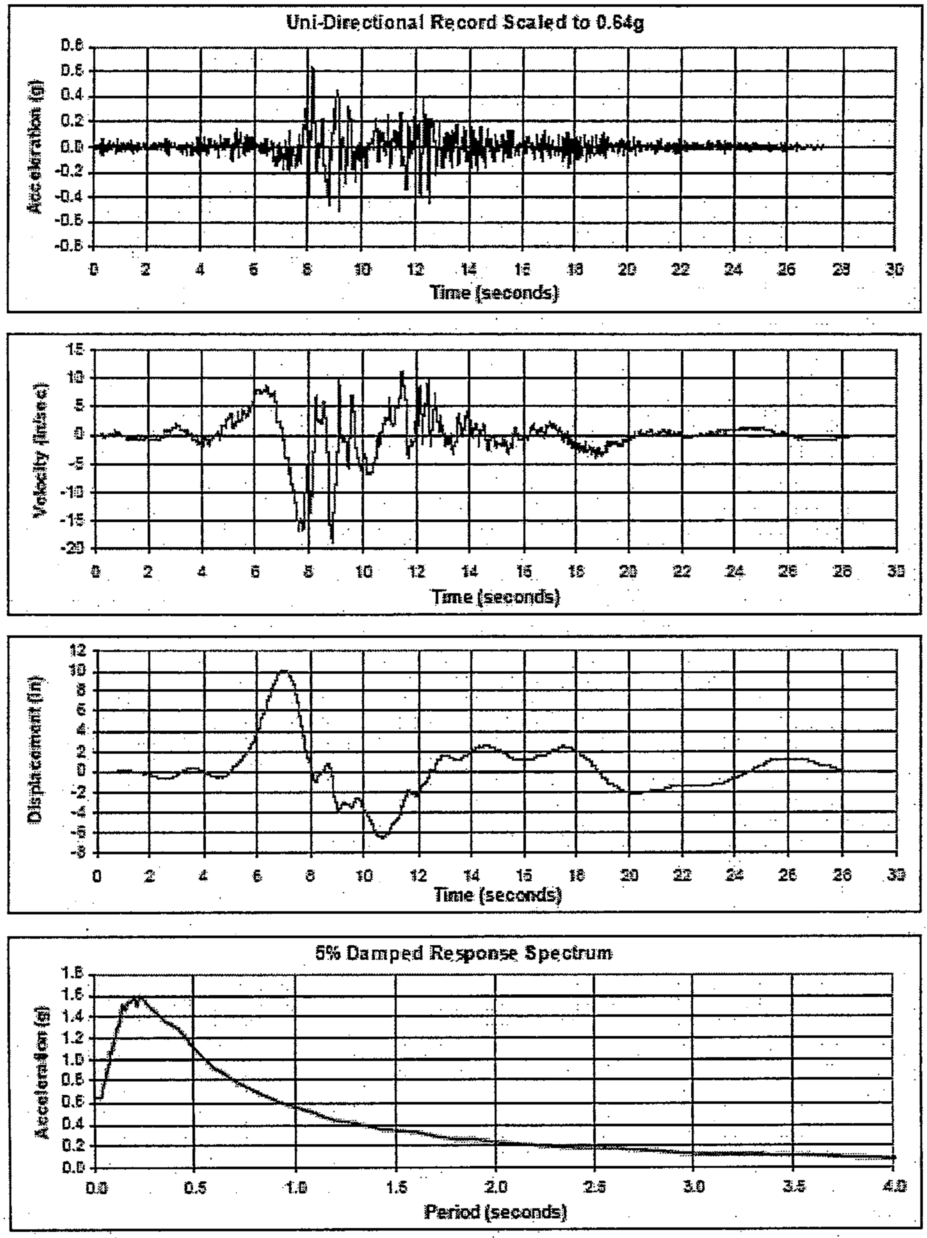



Figure 8. Unidirectional Ground Motion

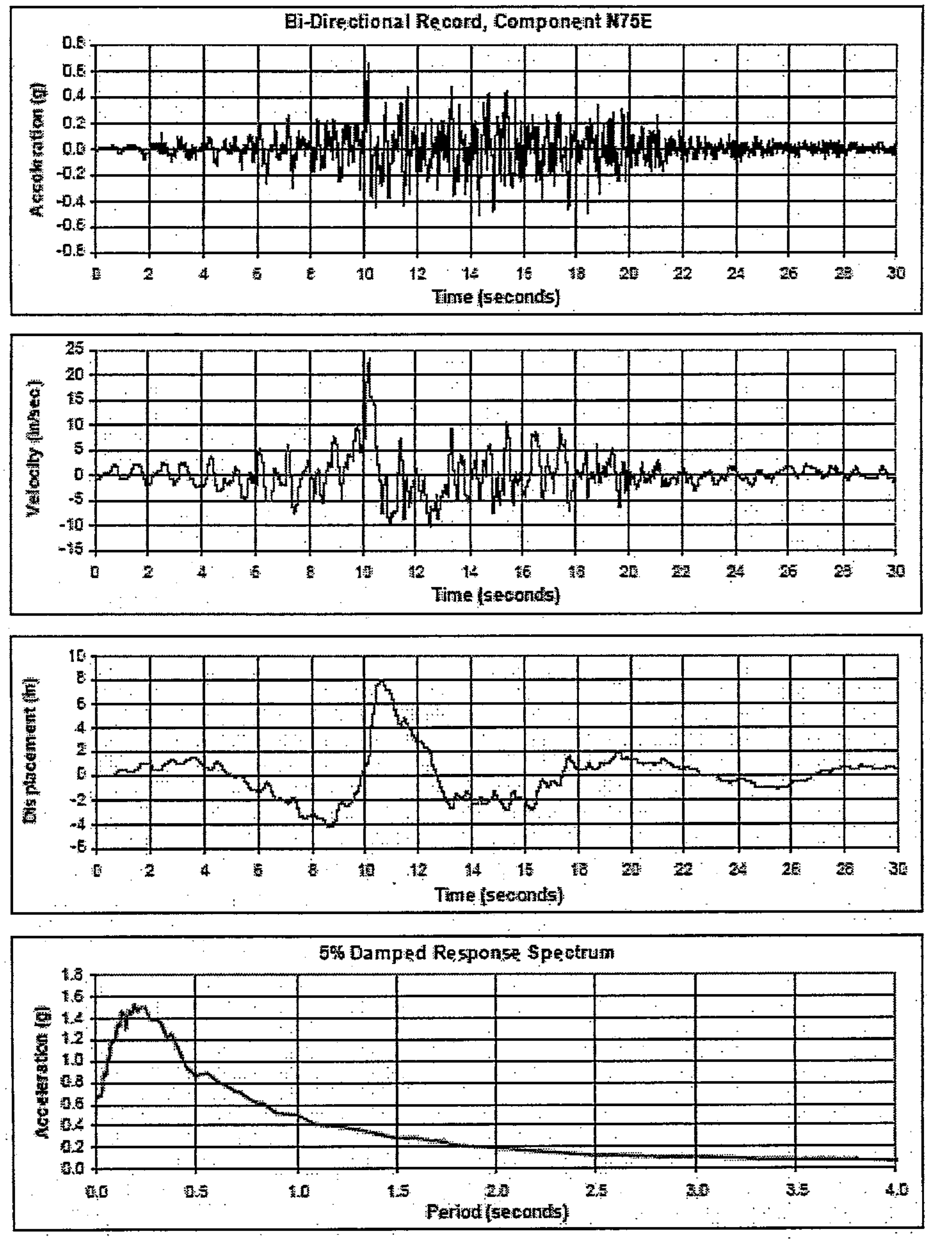



Figure 9 Ground Motion N75E

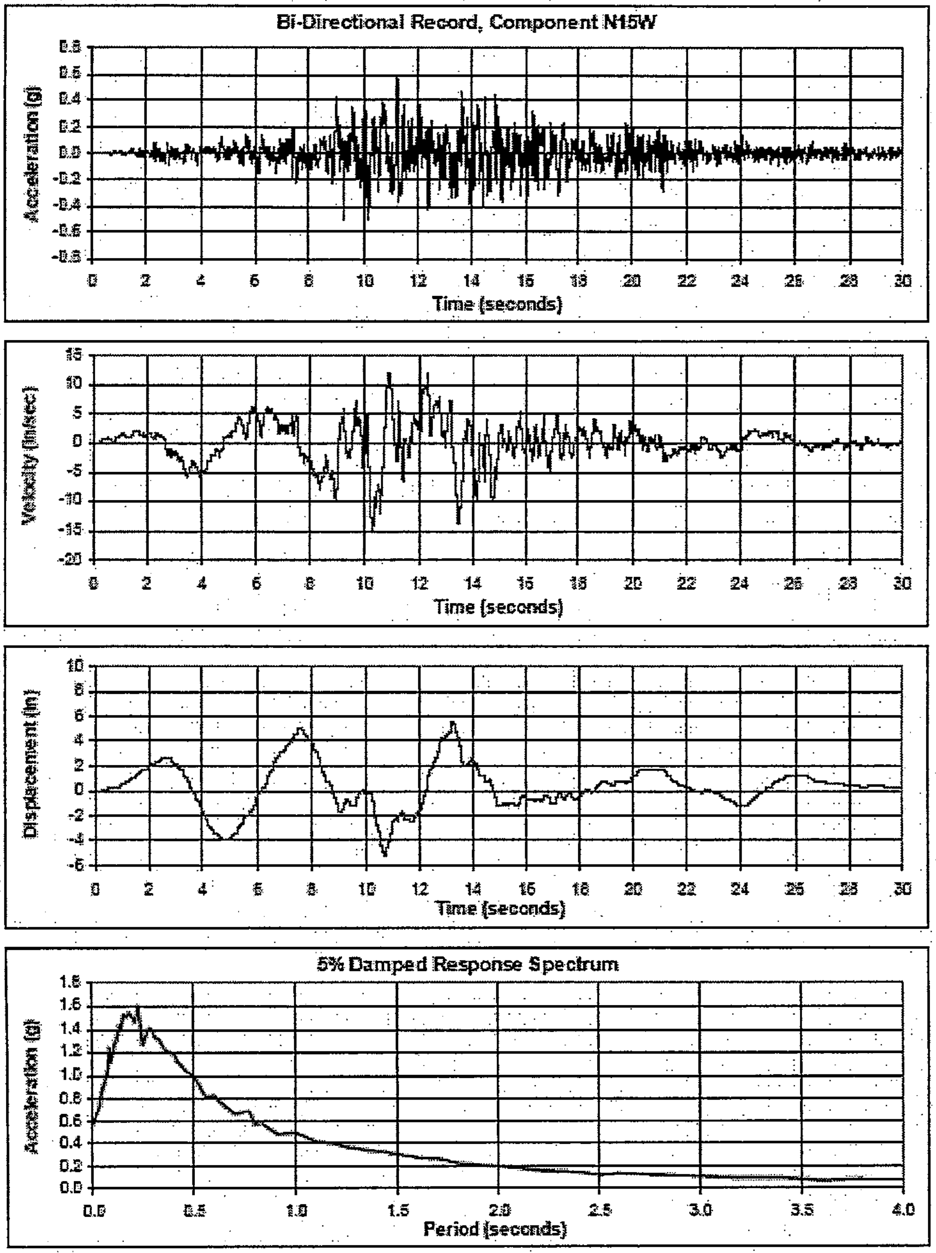
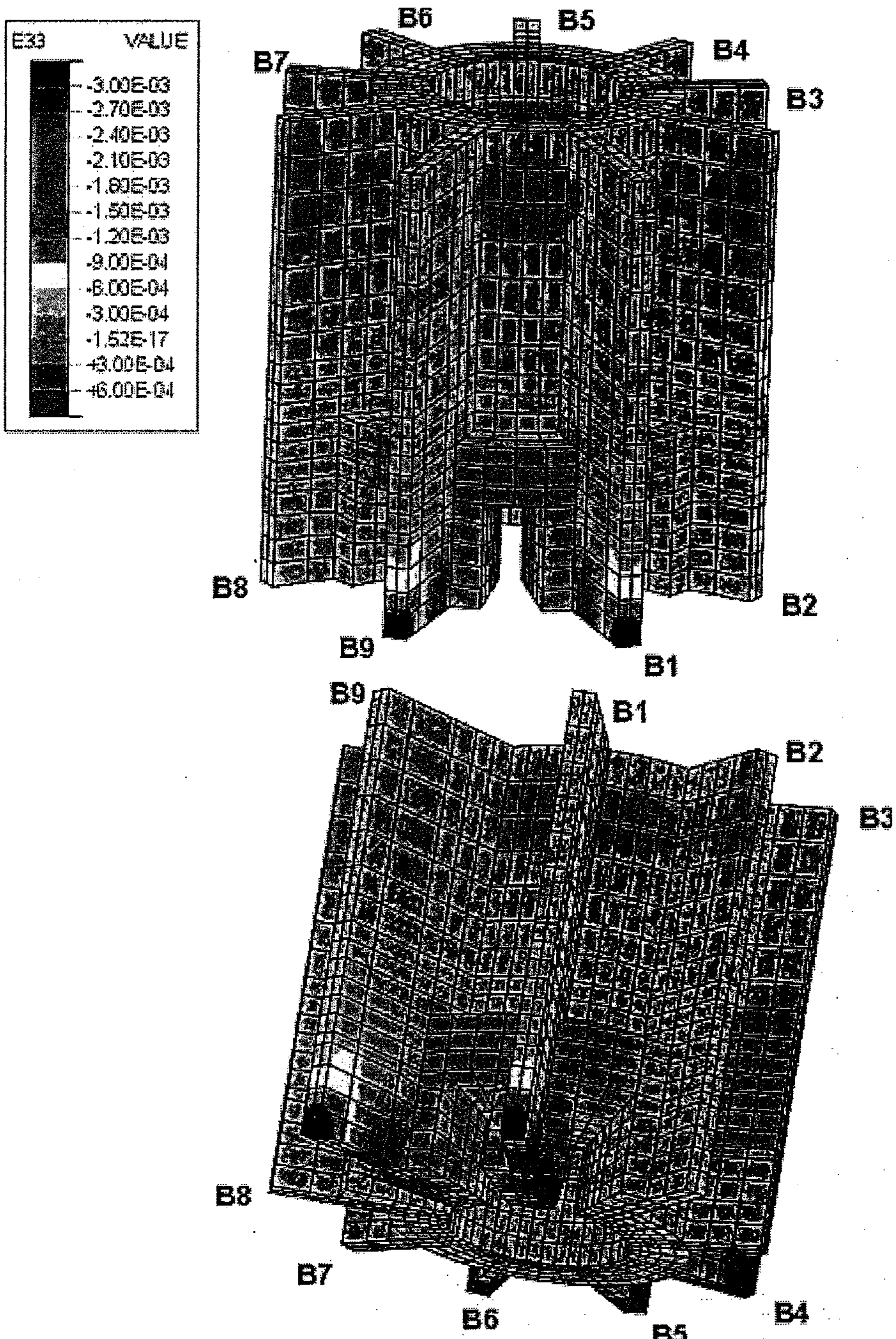
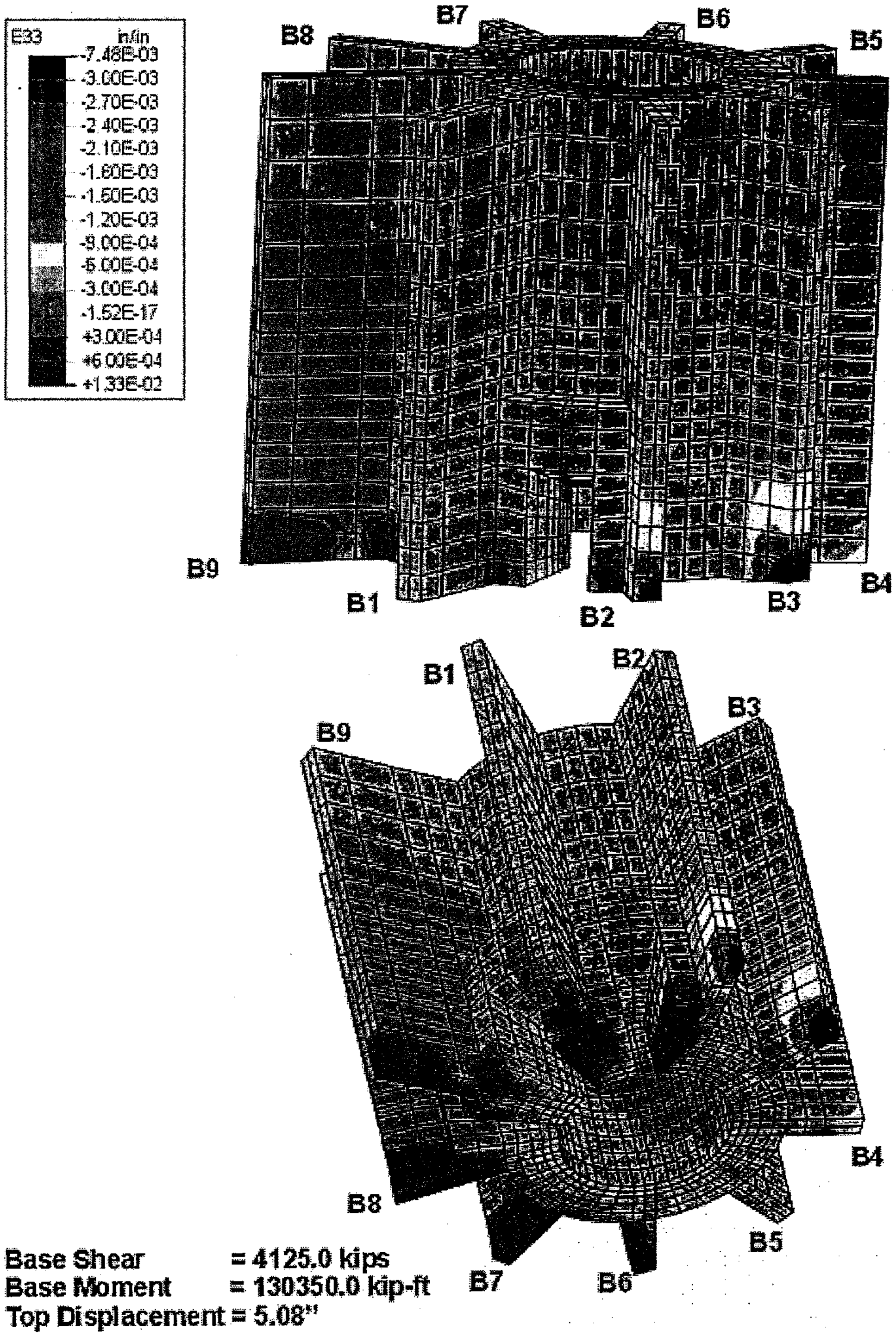



Figure 10. Ground Motion N15W




**Base Shear = 5080.0 kips**
**Base Moment = 165000.0 kip-ft**
**Top Displacement = 5.10"**

Figute 11 CalculatedVertical Strains for Loading in Direction X




Figute 12 CalculatedVertical Strains for Loading in Direction Y

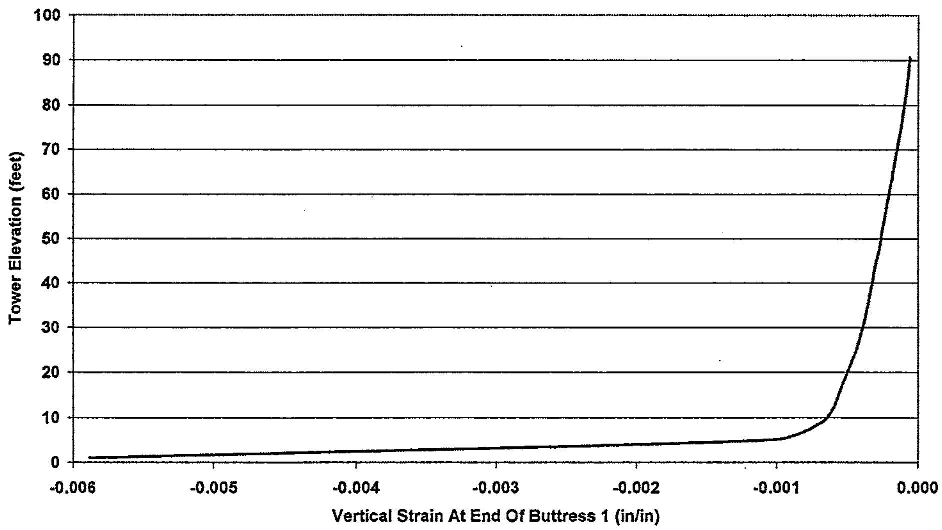



Fig. 13 Variation of Calculated Compressive Strain at Extreme Fiber of Buttress with Height Loading Direction X

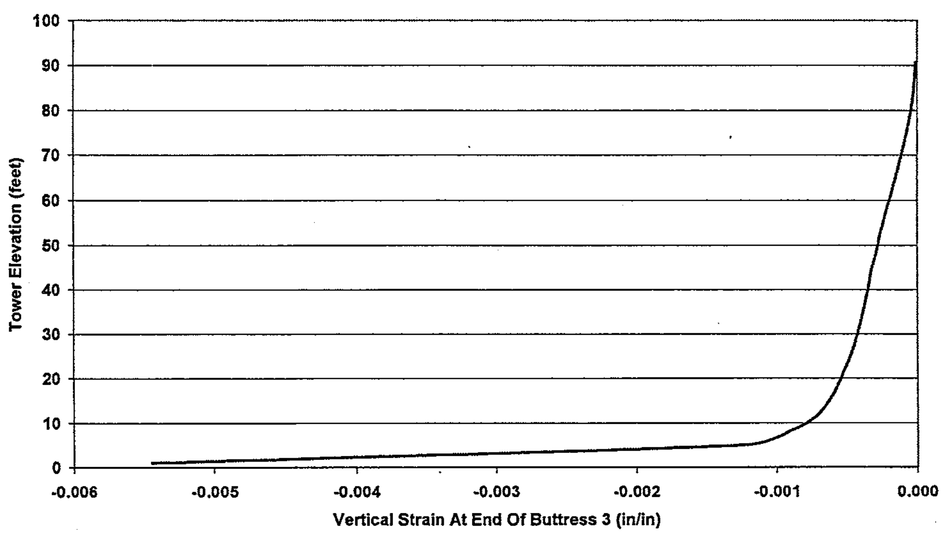



Fig. 14 Variation of Calculated Compressive Strain at Extreme Fiber of Buttress with Height Loading Direction Y

# Lake Almanor Intake Tower
# Plumas County – California:
# Seismic Analysis
# with
# Base Rocking

*prepared for*

**Pacific Gas & Electric Company**
**Hydro Generation Department**
**P.O. Box 770000**
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*prepared by*

**Martin Button, Ph.D., P.E.**
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**Austin, TX 78756**

**BE Project 0609**

**FINAL REPORT**
**rev. 1**
**July 2007**

## TABLE OF CONTENTS

# 1 SCOPE OF WORK

This report describes the development and analysis of a finite element model of the intake tower and a portion of the surrounding reservoir at Lake Almanor, Plumas County, California. The model is analyzed for excitation arising from seismic ground motion at the site. Pacific Gas and Electric Company (PG&E) provided acceleration records characterizing the seismic hazard at the site. The global response of the tower under seismic motion is described.

The analytical model explicitly includes the water surrounding the tower, allowing hydrostatic and hydrodynamic forces on the tower to develop naturally as part of the finite element solution. The base of the tower is permitted to uplift and rock in the model, since the vertical rebar in the tower appears to terminate at that elevation.

The methodology and assumptions behind these analyses, including a detailed description of the model and the seismic excitation, are described in Section 2. The results from the analyses are presented and discussed in Section 3. A summary of findings is presented in Section 4.

# 2 METHODOLOGY AND ASSUMPTIONS

## 2.1 DESCRIPTION OF TOWER AND FORM OF MODEL

According to an earlier report by URS Corporation [1]:

> "Lake Almanor intake tower was built in stages to accommodate the expansions of the hydro facilities and the lake. It was originally constructed in 1914 and was later raised in 1927. In the 1950's the lower two intake gates at invert elevation 4400 feet were sealed and filled with concrete. A large magnitude earthquake was not considered in either the original or modified design stages. The original 1914 Lake Almanor intake tower was a 65-foot high (top of tower at Elev. 4465 feet), lightly reinforced, concrete structure ... The inside diameter of the tower was 16 feet. After the 1927 expansion, the concrete walls varied in thickness from 2.0 feet at the top of the tower to 5 feet close to the base (Elev. 4436 feet). The base of the tower (below Elev. 4422 feet) is cast in a 22-foot deep trench excavated in rock. The intake tower also had nine 2-foot thick buttresses, which extend radially 9 to 17.5 feet from the outside perimeter of the cylindrical section.
>
> "The modifications in 1927 almost doubled the height of the tower to 115 feet (top of tower at Elev. 4515 feet), and added additional steel reinforcing ... The inside diameter of the tower remained 16 feet. The cylindrical section of the original tower was thickened from 2 and 2.5 feet to 5 feet between elevations 4441 feet to 4465 feet. The tower extension above elevation 4465 (feet) varies in thickness from 5 feet to 2 feet at the top (Elev. 4515 feet)."

The current shell model of the tower extends from elevation 4422 feet to 4515 feet (for a total height of 93 feet) and includes the reinforced concrete octagonal control house structure above 4515 feet, extending to the ridge of its roof at elevation 4544 feet. The cross sections of the tower at various elevations are taken from PG&E drawings, and from Figures 4-2 to 4-6 of [1]. The overall footprint of the tower (including buttresses) at its base measures approximately 46 feet in the strong direction by 40 feet in the weak direction.

The reservoir surrounding the intake tower is explicitly modeled using solid elements for both static and dynamic conditions. The bottom of the reservoir is assumed constant at elevation 4422 feet. The top surface of the reservoir is at elevation 4494 feet, resulting in a water depth of 72 feet in the model. The explicit water modeling allows hydrostatic and hydrodynamic forces on the tower to develop naturally as gravity and seismic loadings are applied to the model, without the need to represent hydrostatic loads through a set of concentrated forces, or hydrodynamic forces through approximate "added mass" techniques. In all analyses, the inside of the tower shaft is assumed to be dry.

An August 2006 photo of the intake tower is shown in Figure 1. The photo clearly shows five of the nine buttresses and the control house.

**Figure 1: Lake Almanor Intake Tower (August 2006)**

## 2.2 METHOD OF ANALYSIS

An integrated model of the intake tower and surrounding reservoir is developed using the commercial finite element code SAP2000 [2]. The model is excited by uni-directional and bi-directonal horizontal displacement traces applied at the base of the tower.

The dynamic response of the tower–reservoir system is computed using load-dependent Ritz vectors, with the two orthogonal tower base translation vectors and a set of vectors corresponding to deformations in the foundation gap elements used as starting vectors. Sufficient Ritz vectors are used so that the response of the tower is not sensitive to the number of vectors used in the dynamic analysis.

## 2.3 FINITE ELEMENT MODEL

The development of the combined tower–reservoir finite element model starts with a "dry" tower model. Once the tower model is complete, the surrounding reservoir is then explicitly added to produce an integrated tower–reservoir model.

2.3.1 Tower Model

The tower consists of a cylindrical section with nine radial buttresses equally spaced at 40° around the cylinder. The buttresses are of different lengths, but are symmetrical about the strong axis of the tower.

The tower is modeled using thick shell elements, with a total of 18 elements forming the circumference of the cylinder section. The shell elements are located on the cylinder centerline, which changes in plan as the thickness of the cylinder wall changes with elevation. This provides a plan dimension of the shell elements ranging from 38 to 44 inches. Massless shell elements are used to connect the cylinder centerline to its edge, where the buttress elements begin. The narrowest buttresses (extending 10'–1" from the outside of the cylinder at elevation 4515 feet) are modeled with three elements over their width, while the widest buttresses (extending 18'–5" from the outside of the cylinder at elevation 4515 feet) are modeled with five elements over their length. The dimension of the shell elements in the vertical direction varies from 24 to 48 inches, with a total of 26 elements over the height of the tower. At the base of the tower, the mesh is refined near the end of each buttress. Compression-only (gap) elements are added to the model at elevation 4422 feet to allow uplift and rocking at that elevation. The compression stiffness of these elements is based on an approximate evaluation of the concrete foundation material below 4422 feet. The total compression stiffness provides a vertical frequency of 56 Hz. A detail at the base of the tower model, showing the mesh refinement and gap elements is shown in Figure 2.

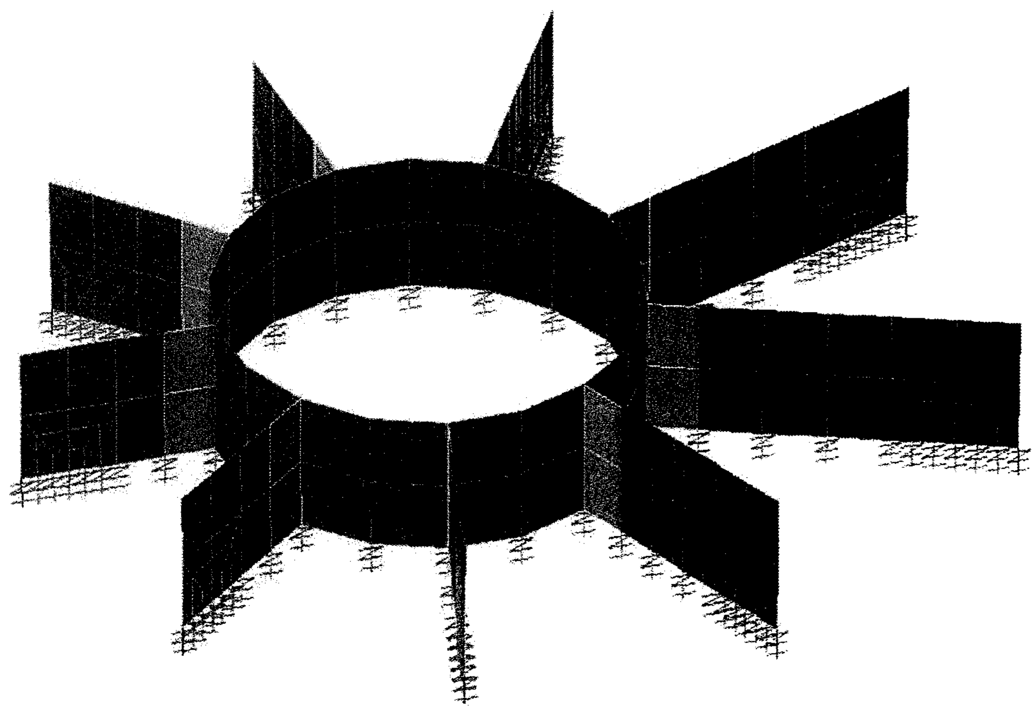

**Figure 2: Detail of Tower Base with Refined Mesh, Elevation 4422 Feet**

At four elevations throughout the tower height, a ring of reinforced concrete beams connects the nine buttresses near their edges. These beams are modeled with frame elements, and have their moments of inertia set at 50% of gross. The four ring beams are modeled with a total of 36 frame elements. One of the ring beams can be seen in Figure 3.

The structure above the top of tower concrete (from elevation 4515 to 4544 feet) is modeled explicitly using shell elements for the reinforced concrete walls, frame elements for the roof trusses and shell elements for the roof membrane. A diaphragm constraint connects the tower section to the control house at 4515 feet.

The weight of the tower and control house is approximately 7,000 kips.

### 2.3.2 Tower–Reservoir Model

Elements representing the water in the reservoir are added to the tower model described in Section 2.3.1 to produce an integrated model representing the tower and the surrounding reservoir. The water is modeled using solid elements with the correct density and bulk modulus, and with a very low shear modulus, to produce water-like behavior. The material properties used to achieve this effect are described in Section 2.4. This approach to water modeling has previously been demonstrated as accurate. The water surface is at elevation 4494 feet and the base of the reservoir is assumed at elevation 4422 feet, giving a water depth of 72 feet. The water model explicitly engages the shaft and buttresses of the tower over its height via a series of very stiff axial link elements. These links are oriented so that the external faces of the shaft and buttresses are subjected to normal pressure from the water. Unrestrained relative movement between water and tower is permitted along the interface local tangent plane.

The reservoir represented in the model extends radially outwards from the center of the tower for a distance of approximately 320 feet, or 4.4 times the assumed depth of the reservoir. The water elements are supported on the flat reservoir bottom so that only normal pressures develop between the water and the bottom, and so that the water can move horizontally relative to the bottom.

At the outermost boundary of the reservoir, vertical and horizontal displacements tangential to the model boundary are permitted. Horizontal dampers in the radial direction were included to produce a radiating boundary, so that a pressure wave propagating outwards passes through the outer boundary without reflection back into the model. However, the tower response was found to be insensitive to this behavior, and these dampers were excluded from the final model.

The combined tower–reservoir model consists of 2,768 frame, 2,680 shell, 4,920 solid and 98 nonlinear link (gap) elements. A portion of the model, showing the top surface of the water, and the tower and control house above the waterline, is presented in Figure 3.

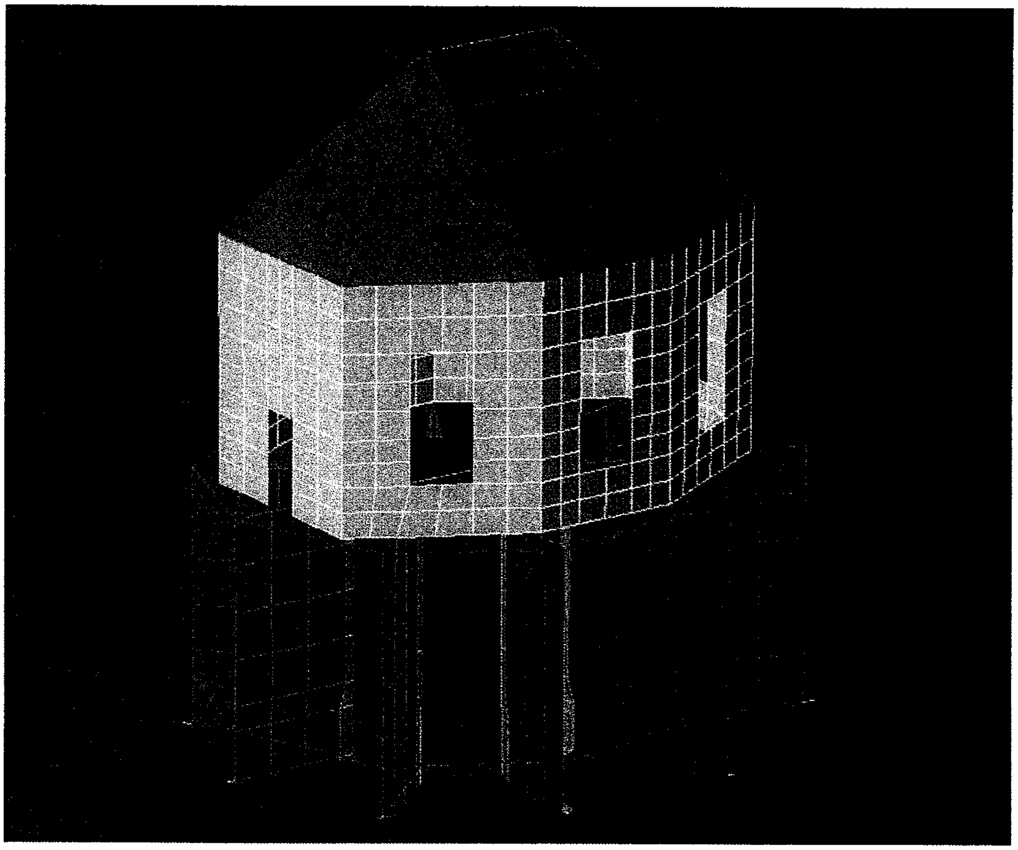

**Figure 3: Control House with Tower Shaft and Buttresses Above Waterline**

## 2.4 MATERIAL PROPERTIES

As noted above, the current tower was partially constructed in 1914, and extended in 1927. Cores taken from the 1927 concrete indicate an average compression strength of 5,600 psi. No cores were taken from the 1914 concrete, but its static strength was assumed at 3,000 psi, with 4,000 psi under dynamic loading. The Young's modulus for the <u>1914 concrete</u> in the tower structure was taken as 3,120 ksi based on a compressive strength of 3,000 psi, while that for the <u>1927 concrete</u> was taken as 4,300 ksi. All tower and control house concrete was assumed to be cracked. This was achieved approximately by using 50% of the above values for the respective Young's moduli. The concrete weight density was taken as 150 $lb/ft^3$.

The solid elements representing <u>water</u> have a weight density of 62.4 $lb/ft^3$, a bulk modulus of 300 ksi, and a shear modulus of 30 psi. The latter two properties are achieved by specifying a Young's modulus of 0.090 ksi and a Poisson's ratio of 0.499950. While the actual shear modulus of water is very close to zero, previous work

[3] has clearly demonstrated that the analytical results for hydrodynamic force is relatively insensitive to a range of water shear moduli from 3 psi to 300 psi.

The steel members of the roof trusses have a weight density of 490 lb/ft$^3$ and a Young's modulus of 29,000 ksi.

## 2.5 LOADS AND LOAD CASES

Three types of loads were considered in this analysis. The load types are as follows:

1. Dead Load – Dead load originates from gravity acting on the self weight of the members of the tower structure and control house. The resulting tower weight is approximately 7,000 kips.
2. Hydrostatic Load – Hydrostatic load originates from gravity acting on the self weight of the water in the reservoir, and results in normal pressures acting on the cylindrical shaft and buttresses of the tower structure. Since the tower is completely surrounded by water, the net horizontal hydrostatic load on the tower is zero.
3. Seismic Load – Digitized horizontal acceleration traces for one single component earthquake record and one bi-directional earthquake record were provided by PG&E. The single component record was scaled to a peak ground acceleration of 0.64g. The bidirectional record was not scaled. The acceleration records were double integrated to obtain ground displacement records at a time step of 0.002 seconds for use in analysis. The ground motions and their 5% damped response spectra are shown in Figures 4 and 5 for the uni- and bi-directional records respectively.

### 2.5.1 Application of Dead and Hydrostatic Loads

Dead and hydrostatic loads arise directly from the application of gravity to the combined tower–reservoir model.

### 2.5.2 Application of Seismic Loads

The absolute displacement formulation of the structural dynamics problem is used as the basis for this analysis. The absolute displacement formulation allows both inertial and hydrodynamic forces to develop as part of the analytic solution as the base of the tower is pushed back and forth in the reservoir by the ground displacement motion.

For the unidirectional record, the combined tower–reservoir model is subjected to the horizontal displacement ground motion separately along each of the tower's two principal axes. Two cases are constructed for bi-directional excitation. Firstly, the bidirectional record is used directly, with the N75E component applied along the strong axis of the tower and the N15W component applied in the weak direction. Secondly, the unidirectional record is applied along the strong axis of the tower and the N15W component of the bi-directional record is applied simultaneously in the weak direction.

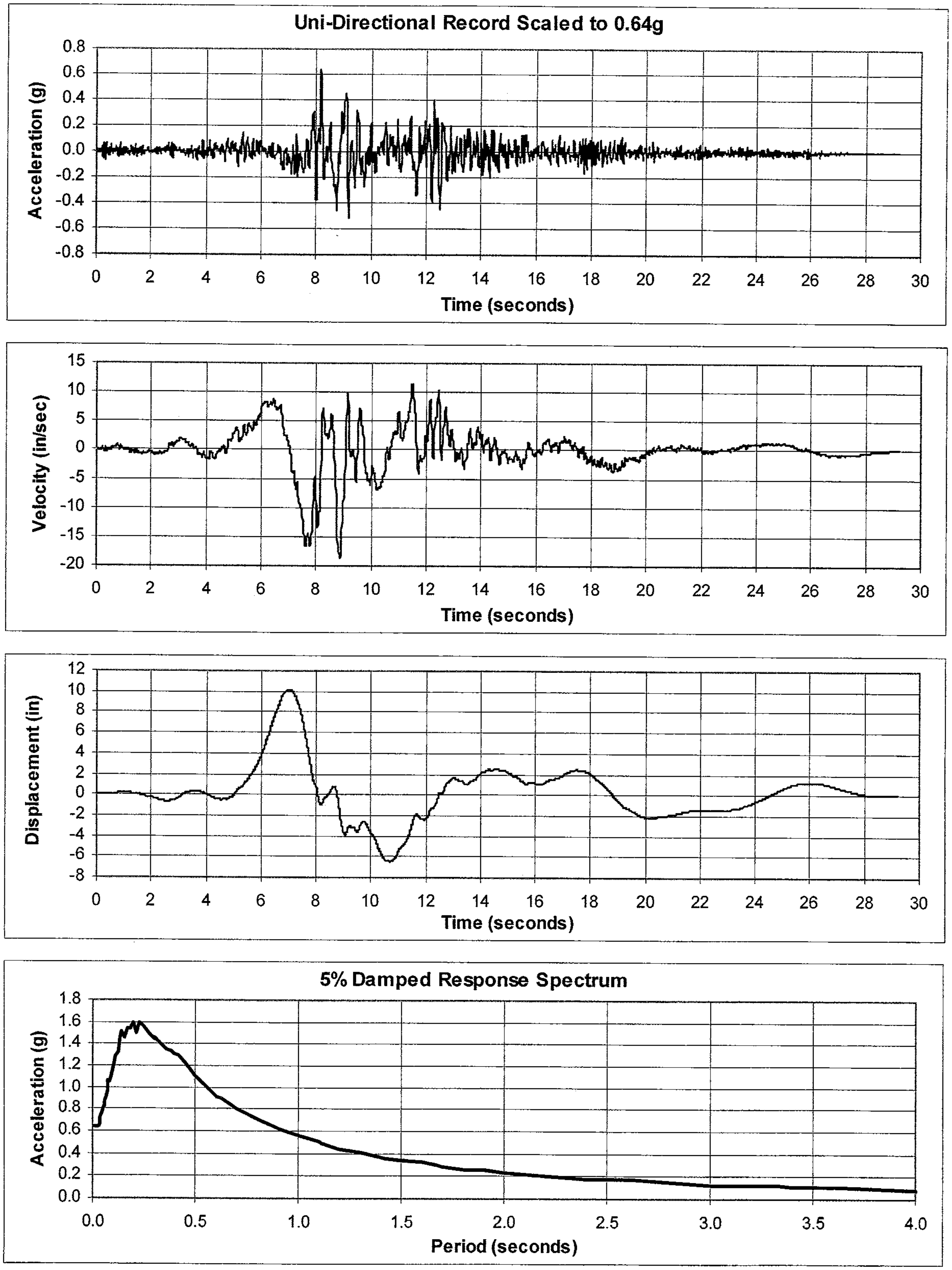


**Figure 4: Characteristics of Uni-Directional Record, Scaled to 0.64g**

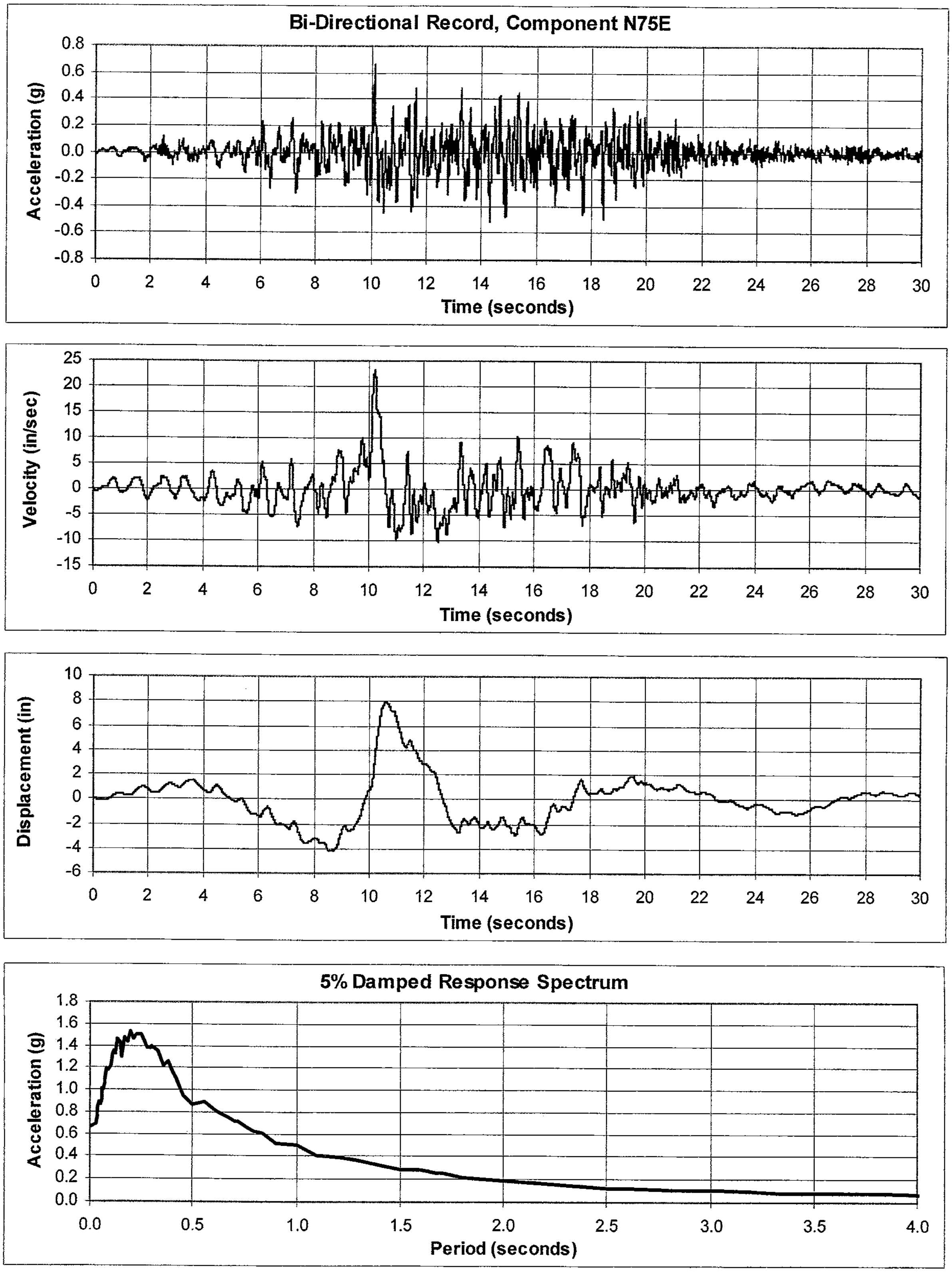


**Figure 5a: Characteristics of N75E Record**

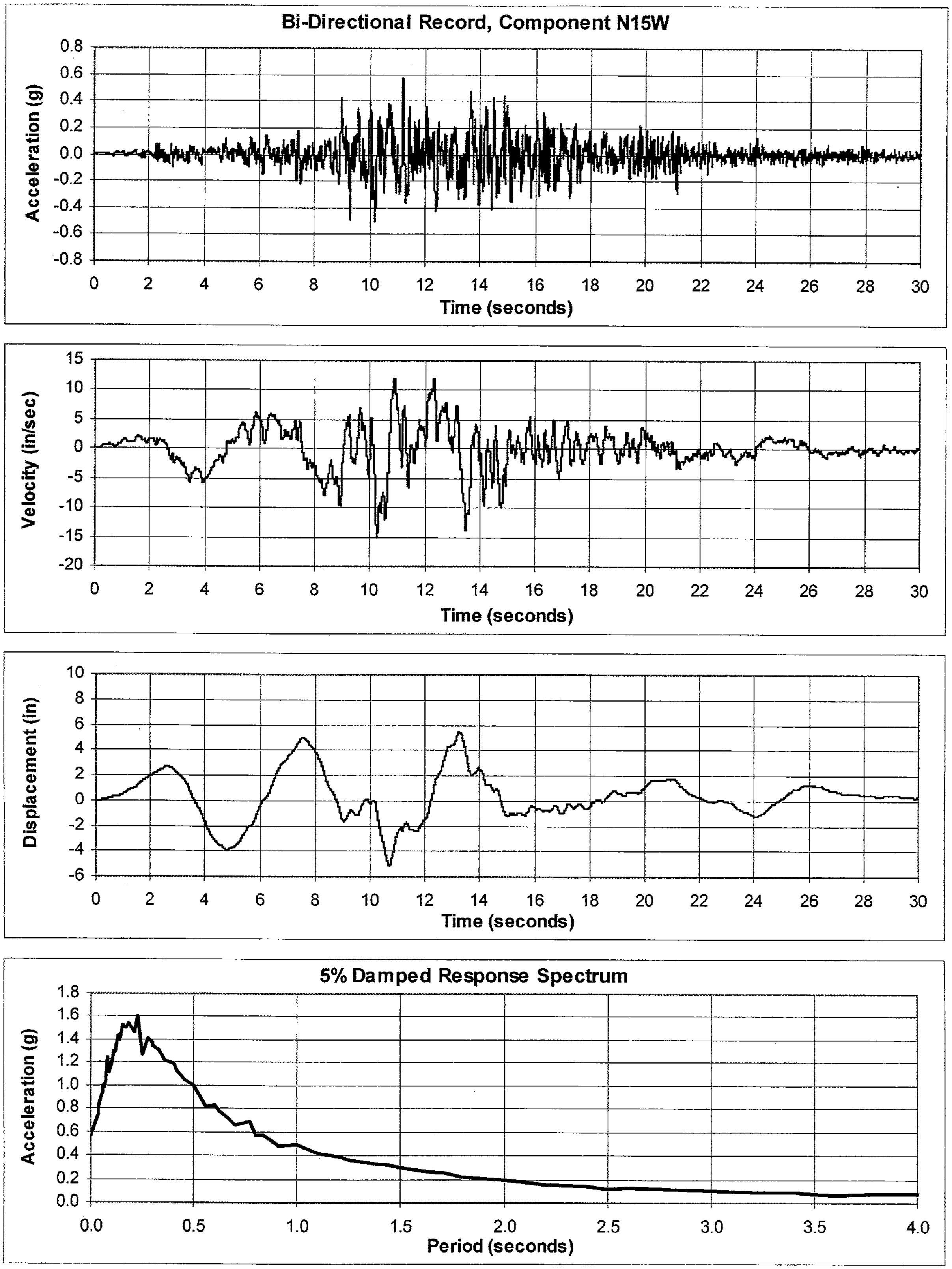


**Figure 5b: Characteristics of N15W Record**

# 3 RESULTS OF ANALYSES

This section presents the results from three distinct runs on the rocking tower–reservoir model. Section 3.1 presents tower response under uni-directional excitation. The earthquake record shown in Figure 4 is used to excite the tower–reservoir system separately along its strong and weak axes. The results presented for each direction are the displacement of the tower at elevations 4415 feet (top of tower) and 4435 feet (top of control house walls), the horizontal shear at the base of the tower (elevation 4422 feet), the vertical uplift at the base of the tower, and the average shear stress in the control house walls. Section 3.2 presents similar results for the two cases of bi-directional excitation.

The strong direction of the tower corresponds to its one axis of symmetry, as shown in Figure 6. The weak direction of the tower is orthogonal to the strong axis. For uni-directional seismic loading in the strong direction, the tower responds without twisting. However, for uni-directional seismic loading in the weak direction, the tower is not symmetric, and in addition to shear and moment associated with the direction of loading, the tower twists.

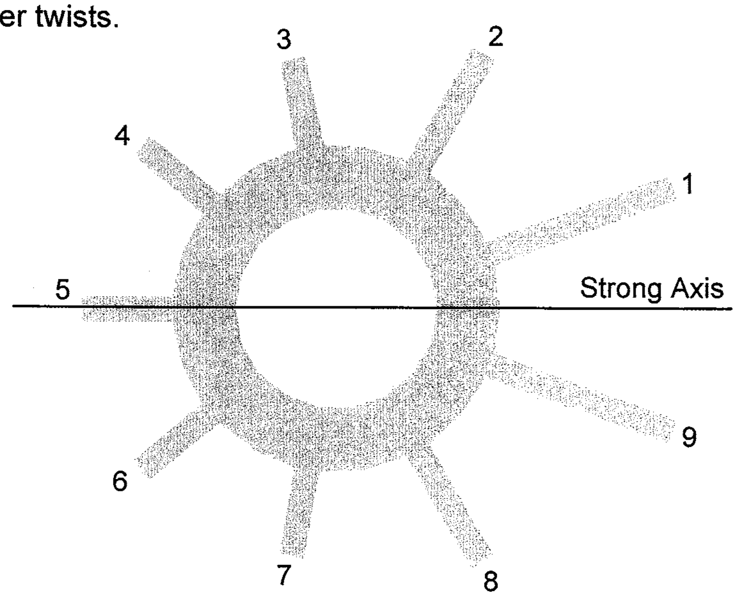


**Figure 6: Strong Axis of Tower and Buttress Numbering**

All reported analyses use 600 Ritz modes to define the dynamic characteristics of the tower–reservoir system. To generate these Ritz modes, staring vectors corresponding to two orthogonal tower base translations and deformations in all 98 nonlinear gap elements are used. The computed response with 600 modes was shown to be converged when nearly identical response was computed with 1,200 Ritz vectors. 5% damping is assigned to all modes.

The current analytical model uses elastic elements for the concrete shaft and buttresses. While cracking of concrete is approximated by the use of a reduced elastic modulus as described in Section 2.4, potential crushing of concrete is not considered, and concrete compression stresses at the base of the tower can exceed the crushing strength of the concrete.

## 3.1 RESPONSE UNDER UNI-DIRECTIONAL EXCITATION

The uni-directional excitation shown in Figure 4 is used to excite the tower–reservoir system separately along the tower strong and weak axis directions. The resulting response is reported in Table 1, where $\Delta_H$ and $\delta_V$ signify horizontal deformation (relative to tower base) and uplift displacement respectively. Control house $\tau$ is the average shear stress at the base of the walls in the control house at elevation 4515 feet.

**Table 1: Uni-Directional Response of Existing Tower with Rocking Base**

| Response | Units | Excitation Direction | |
|---|---|---|---|
| | | Strong Axis | Weak Axis |
| $\Delta_H$ at 4535' | *inches* | 3.5 | 4.9 |
| $\Delta_H$ at 4515' | *inches* | 2.7 | 4.1 |
| $V_{base}$ at 4422' | *kips* | 7,300 | 6,200 |
| $\delta_V$ at 4422' | *inches* | 1.1 | 1.6 |
| Control house $\tau$ | *psi* | 56 | 79 |

Selected response history traces are shown in Figures 7 and 8. Figure 7 shows a trace of tower horizontal deformation at elevation 4515 feet, under weak axis excitation. Since the earthquake is applied to the model as base displacements, the solution is computed in terms of absolute displacements. The relative displacement between the base and top of the tower is processed outside of the analysis. Figure 7 shows rocking behavior very clearly. The period of tower response lengthens dramatically between approximately 7 and 12 seconds into the record. The maximum deformation at the top of the tower (elevation 4515 feet) is 4.1 inches. Figure 8 shows the vertical displacements at the end of four buttresses (identified in Figure 6) for weak axis excitation. Again, rocking behavior is clearly evident: when buttresses 7 and 8 are uplifting, buttresses 2 and 3 are in compression, and vice versa.

The shear stresses at the base of the control house walls remain well below $2\sqrt{f'_c}$ throughout the duration of shaking in both the strong and weak directions.

It should be noted that the tower rocks in a manner very close to that of a rigid body. In the weak axis direction, the aspect ratio (height to elevation 4515 feet divided by plan dimension) is approximately 2.3. Taking the maximum uplift at the base (1.6 inches) for weak axis excitation, and multiplying by the tower aspect ratio, produces a horizontal displacement at elevation 4515 feet of 3.7 inches. The total deformation at that level in the weak direction is 4.1 inches (see Table 1). Therefore, rigid body rocking accounts for approximately 90% of the total horizontal deformation at elevation 4515 feet.

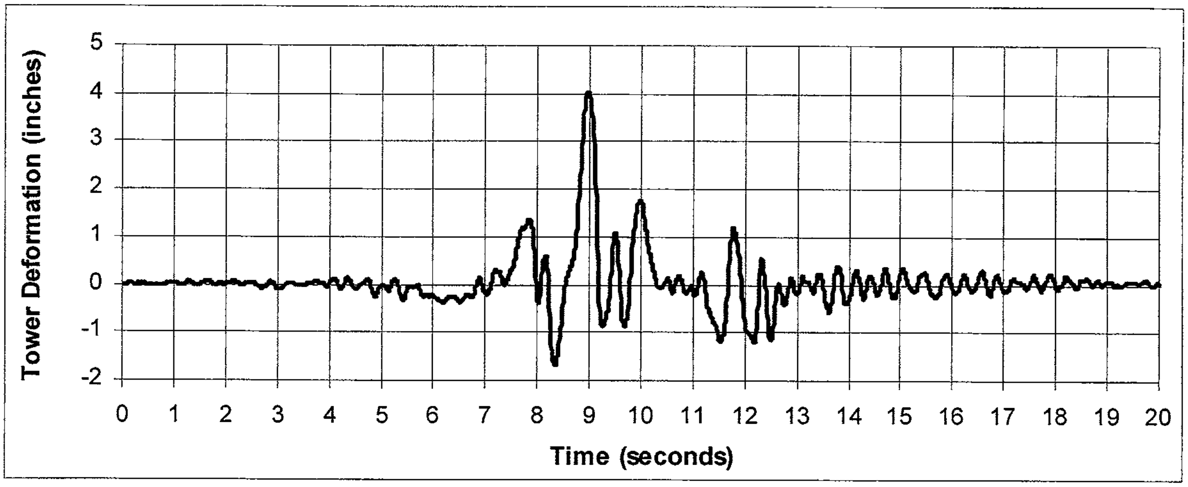


**Figure 7: Tower Weak Direction Horizontal Deformation, Elevation 4515 Feet**

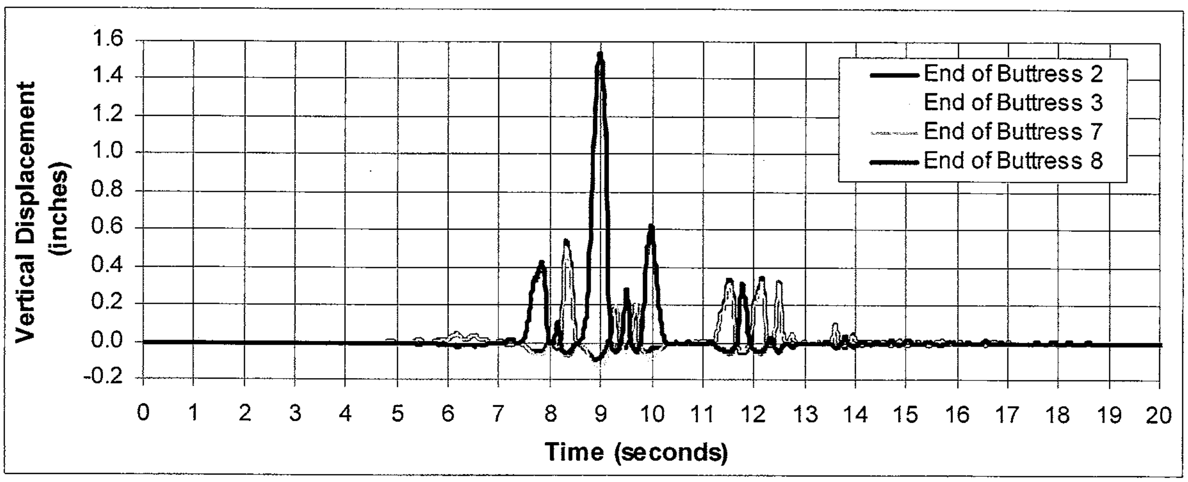


**Figure 8: Vertical Displacements at Elevation 4422 Feet (Weak Direction EQ)**

## 3.2 RESPONSE UNDER BI-DIRECTIONAL EXCITATION

Two bi-directional excitation cases are considered. The first case applies the two components shown in Figure 5a and 5b, with the N75E component aligned with the strong axis of the tower and the N15W component with the weak direction. The second case consists of the uni-directional record of Figure 4 applied along the strong axis of the tower, with the N15W record of Figure 5b applied along the weak direction.

Table 2 presents a summary of the tower response for bi-directional case 1 (N75E / N15W), while Table 3 presents similar response information for bi-directional case 2 (uni-directional / N15W).

**Table 2: Response of Existing Tower with Rocking Base, Bi-Directional Case 1**

| Response | Units | Response Direction | |
|---|---|---|---|
| | | Strong Axis | Weak Axis |
| $\Delta_H$ at 4535' | *inches* | 6.2 | 6.0 |
| $\Delta_H$ at 4515' | *inches* | 5.2 | 4.9 |
| $V_{base}$ at 4422' | *kips* | 7,100 | 5,700 |
| $\delta_V$ at 4422' | *inches* | see Fig. 10 | see Fig. 10 |
| Control house $\tau$ | *psi* | 55 | 88 |

**Table 3: Response of Existing Tower with Rocking Base, Bi-Directional Case 2**

| Response | Units | Response Direction | |
|---|---|---|---|
| | | Strong Axis | Weak Axis |
| $\Delta_H$ at 4535' | *inches* | 3.7 | 3.7 |
| $\Delta_H$ at 4515' | *inches* | 2.9 | 2.9 |
| $V_{base}$ at 4422' | *kips* | 7,200 | 5,700 |
| $\delta_V$ at 4422' | *inches* | 1.3 | 1.2 |
| Control house $\tau$ | *psi* | 72 | 95 |

Case 1 (Table 2) proves to be the more critical bi-directional record for tower response. The response under bi-directional case 2 is similar to that for the uni-directional case. The case 1 maximum tower horizontal deformation (elevation 4515 feet relative to the base) is 5.2 inches in the strong axis direction. Figure 9 shows a trace of this response.

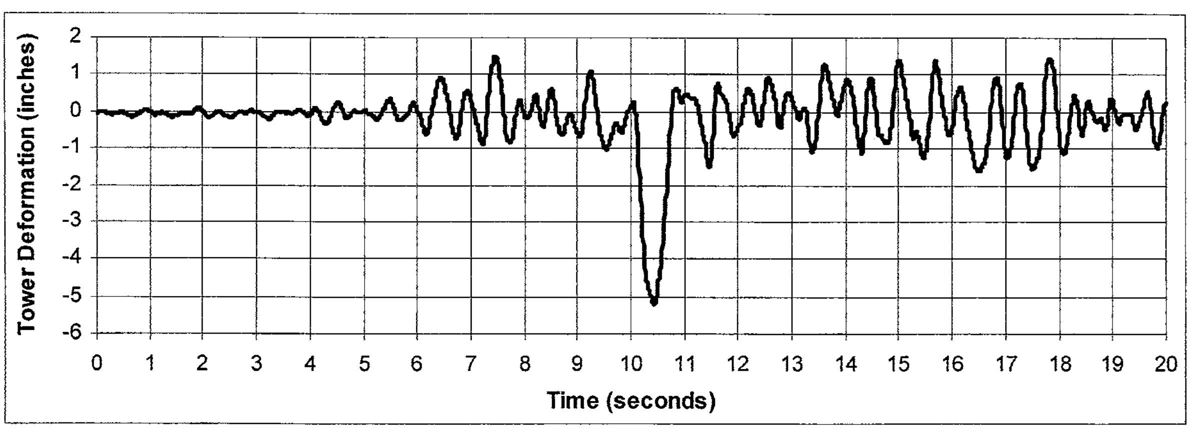


**Figure 9: Tower Strong Axis Horizontal Deformation at 4515 Feet (Case 1)**

Figure 10 shows a portion of the vertical displacement response history at the ends of all nine buttresses at elevation 4422 feet. The shift in time of maximum uplift from one buttress to the next indicates a rolling response of the tower.

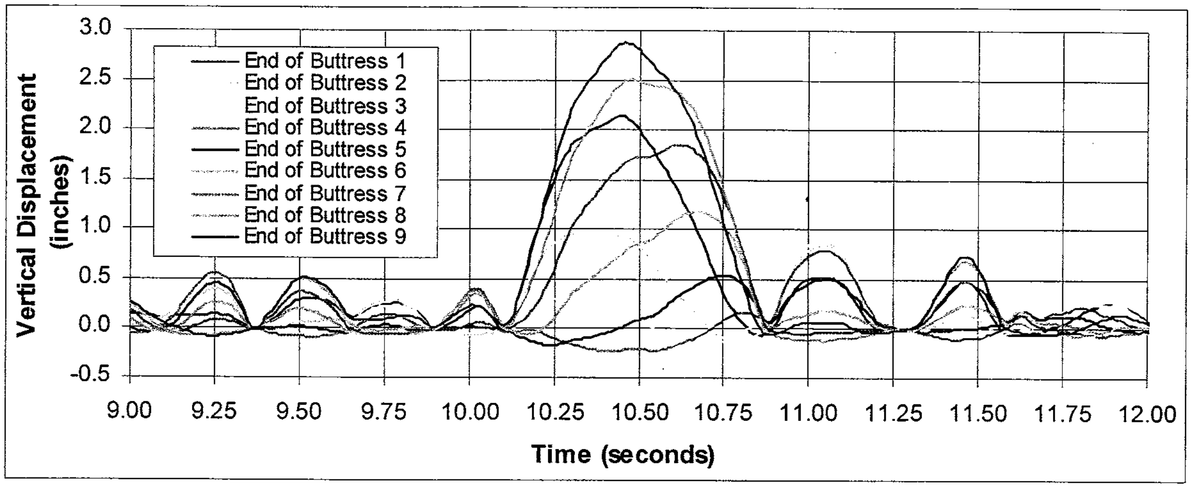


**Figure 10: Vertical Displacements at Elevation 4422 Feet (Case 1)**

The shear stresses at the base of the control house walls remain well below $2\sqrt{f'_c}$ throughout the duration of the case 1 shaking in both the strong and weak directions.

Response history plots are not presented for case 2, since the response under this excitation is lower than that for case 1 (compare Tables 3 and 2).

Comparing Tables 1, 2 and 3, it is clear that of the three single-component records considered, the most critical for the tower is the N75E component of the bi-directional record, shown in Figure 5a. Of the three records considered, this one has the largest peak ground velocity, 23.3 inches/second, compared to 15.1 and 18.7 inches/second for the N15W component (Figure 5b) and the uni-directional record (Figure 4) respectively. The maximum horizontal strong axis displacement at elevation 4515 feet (see Figure 9) occurs at approximately 10.5 seconds, just after the pulse containing the peak velocity in the input motion (see Figure 5a).

# 4 SUMMARY OF FINDINGS

This report has presented the results from a series of analyses of the seismic response of the intake tower at Lake Almanor. The results are derived from a finite element model of the tower and a portion of the surrounding reservoir. In this model, the base of the tower is permitted to locally uplift and rock. The concrete in this model has linear elastic properties, with a reduced Young's modulus value to approximately account for softening due to concrete cracking.

The results of the analysis under three seismic scenarios provided by PG&E indicate that the tower will rock. The horizontal displacements at elevation 4515 feet range from 2.7 to 5.2 inches, depending on earthquake and direction. The uplift displacements at the ends of the buttresses range from 1.1 to 2.9 inches depending on the excitation.

The contribution of elastic flexural and shear deformations within the height of the tower to the total horizontal relative displacement at elevation 4515 feet is small. The results presented in this report clearly demonstrate that rigid body rocking contributes over 90% of the maximum horizontal relative displacement at that elevation.

The evaluation of the integrity of the intake tower at horizontal displacements of these magnitudes during seismic shaking is beyond the scope of this work. Such an evaluation would need to consider an inelastic model for the concrete, including concrete crushing. However, the lateral displacements determined from the current analyses may be used as reasonable estimates of target displacements for a separate lateral nonlinear push analysis.

The stresses developed in the reinforced concrete walls of the control building remain within the concrete capacity for all three earthquake scenarios considered. During the site visit conducted during the course of this work, it was observed that there was no positive physical connection between the ends of the roof trusses and the top of the concrete walls of the control house. Regardless of the conclusions reached regarding the integrity of the tower itself, these connections should be re-worked to provide positive force transfer between the roof system and the concrete walls of the control building.

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# Pushover

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# 1 INTRODUCTION

## Background

Lake Almanor on the North Fork of the Feather River in Northern California was established in 1914 via construction of the earth-filled Canyon Dam for the purpose of water storage for use in hydroelectric power generation. The intake tower for releasing water from the reservoir is a lightly reinforced, 115' tall, cylindrical concrete structure with stepped walls and 9 radially oriented buttresses of varying lengths around the circumference. The original structure, built in 1912-1914, was approximately 65' high. In the mid 1920's, the dam and intake tower were raised by 50' to increase the water storage capacity. Uncontrolled releases of water from the reservoir could occur if the tower fails or is sufficiently damaged during a seismic event. General details of the intake tower taken from the original plan sheets are shown in Figures 1-1 through 1-5.

Previously Pacific Gas & Electric Company (PG&E) reviewed the seismicity of the dam site. This showed the Maximum Credible Earthquake (MCE) could be expected to produce peak ground accelerations up to one G for the 84$^{th}$ percentile earthquake. Subsequently, the hazard potential for the tower has been reclassified, and the seismic records have been scaled down to represent the 50$^{th}$ percentile ground motions for the MCE.

Button Engineering has completed dynamic time history analyses of the baseline structure with the SAP2000 finite element program, [1]. The purpose of the time history analysis is to determine the global response and target displacement demands of the tower under the seismic motion.

## Objective and Scope

The objective of this study is to evaluate the capacity and structural integrity of the tower, primarily at the inlet gate openings, at the target displacement demand levels as computed with the SAP2000 time history analyses. This evaluation was performed with the general-purpose finite element program ABAQUS [2] coupled with the ANACAP-U [3] nonlinear material constitutive models. Uniaxial pushover analyses using two different loading methodologies have been performed for this capacity evaluation.

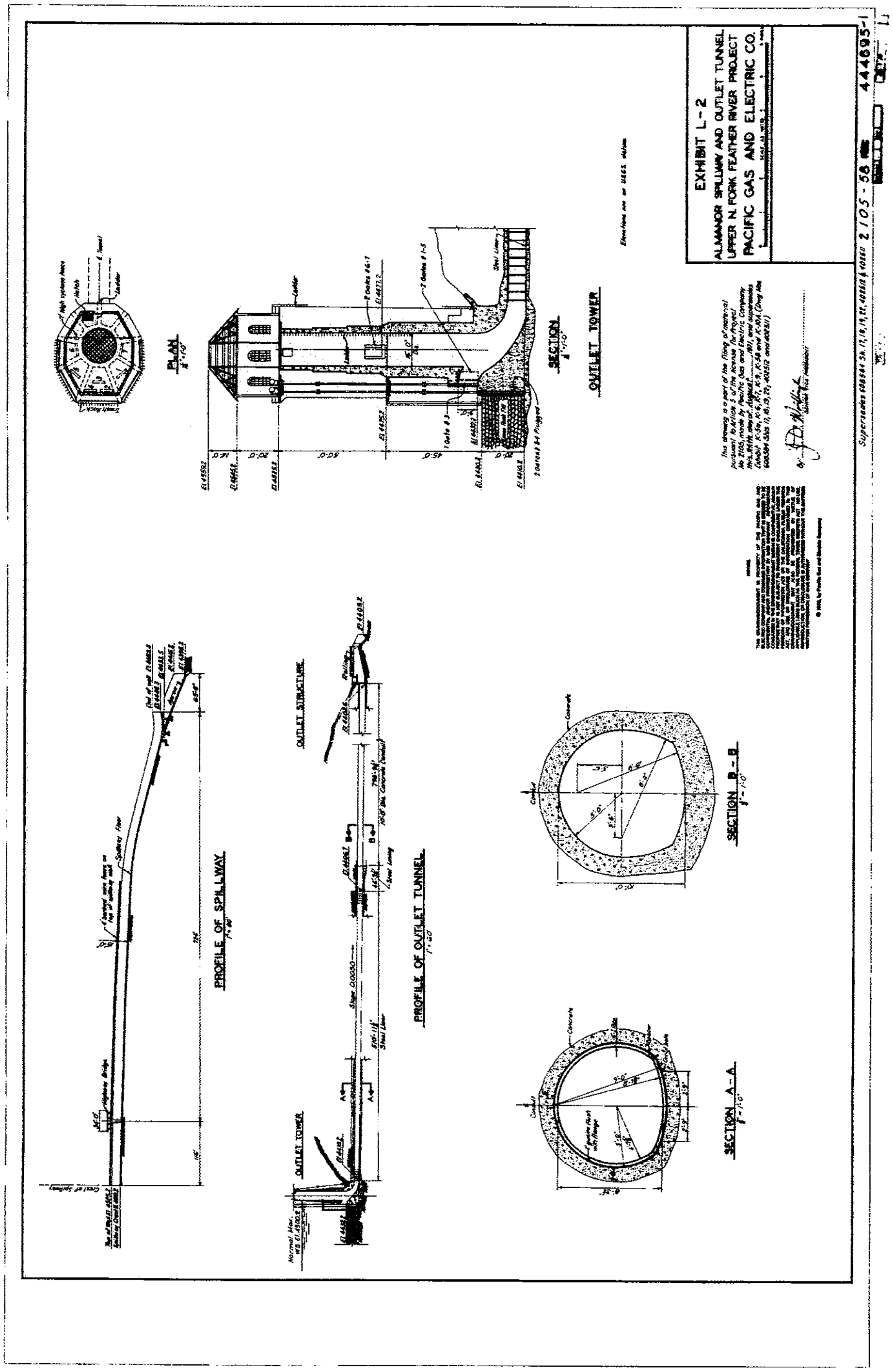


**Figure 1-1**
**Lake Almanor Spillway and Outlet Tunnel**

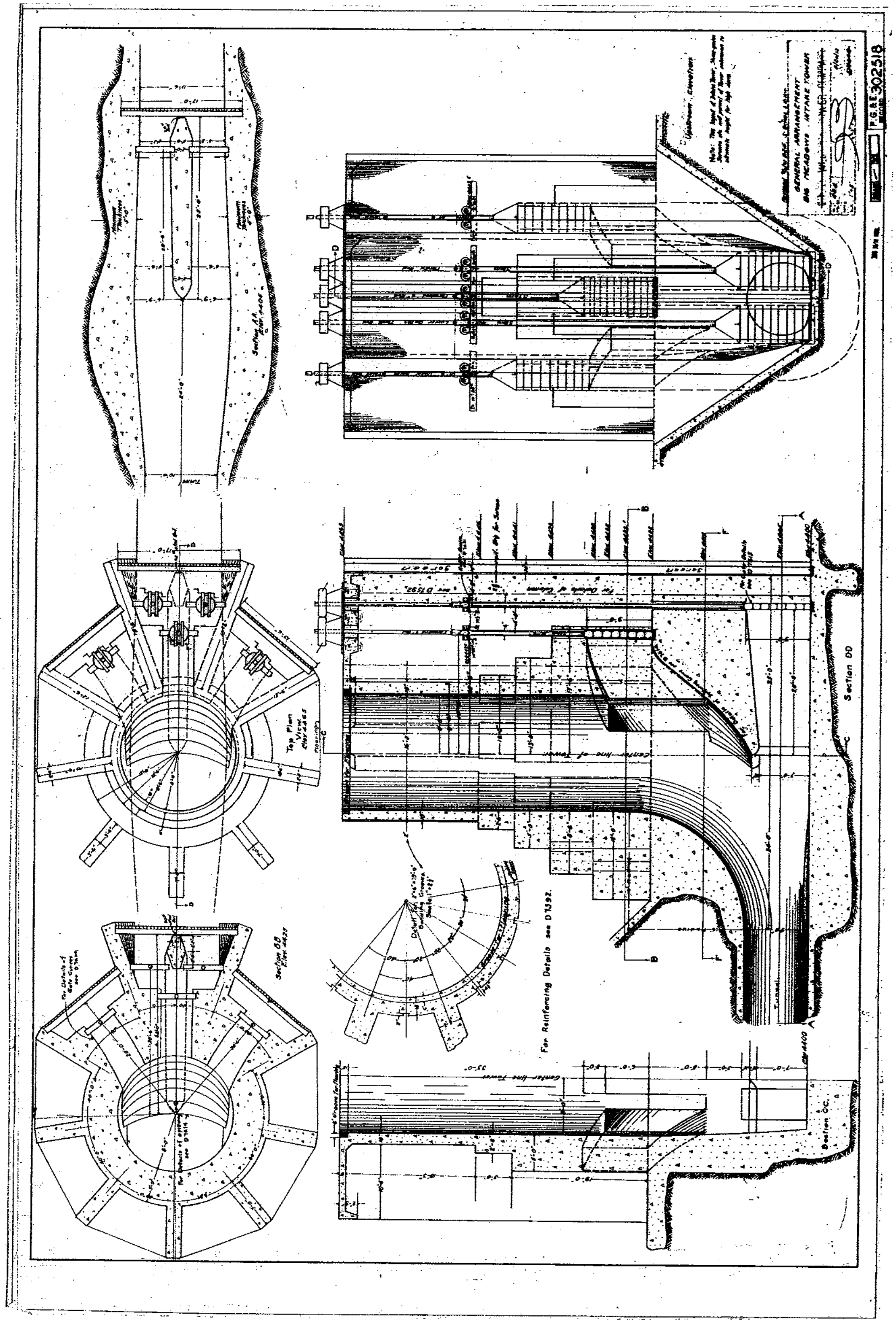

**Figure 1-2**
**Lake Almanor Intake Tower, As-Built 1914 Construction**

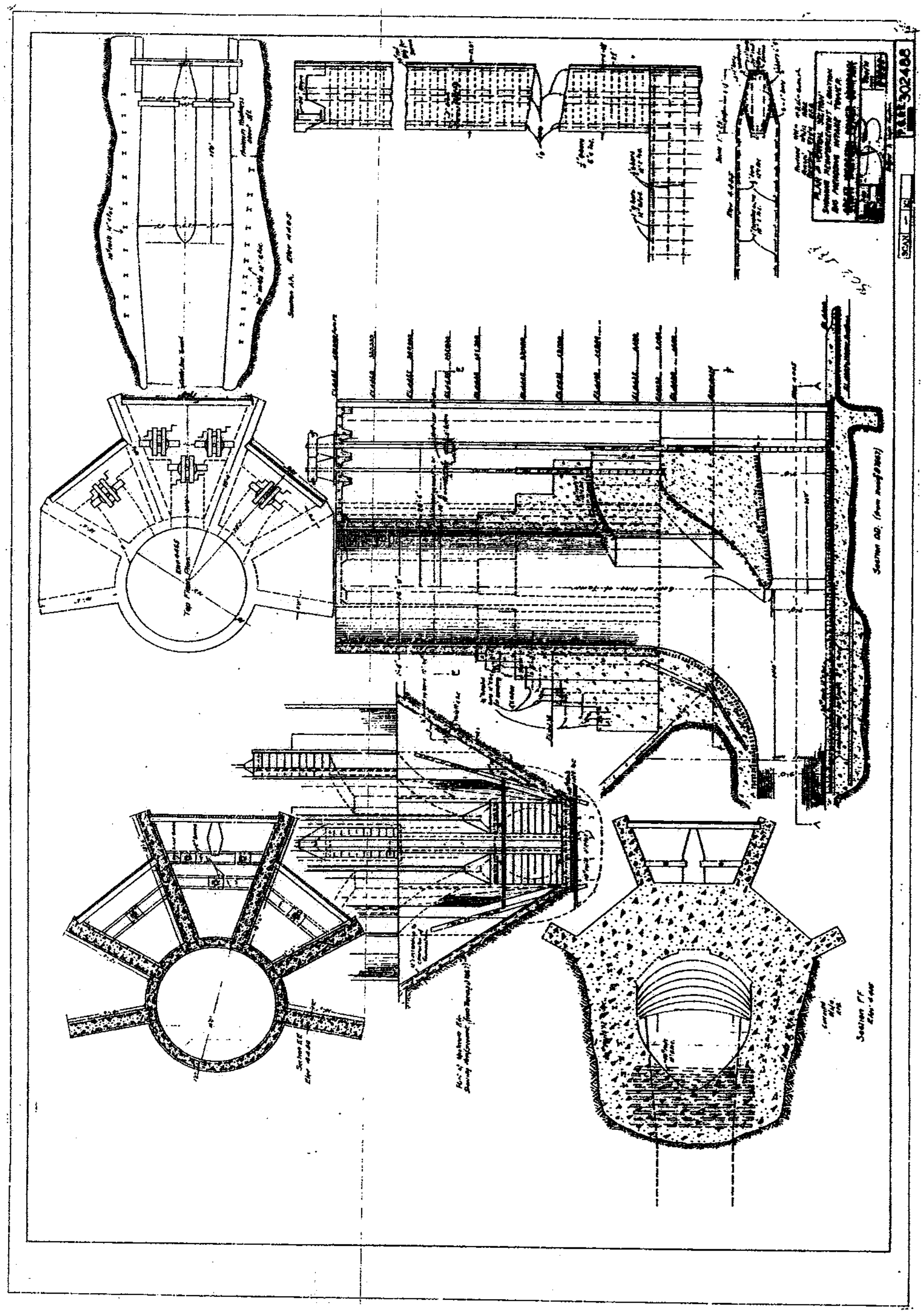

**Figure 1-3**
**Lake Almanor Intake Tower, As-Built 1927 Addition**

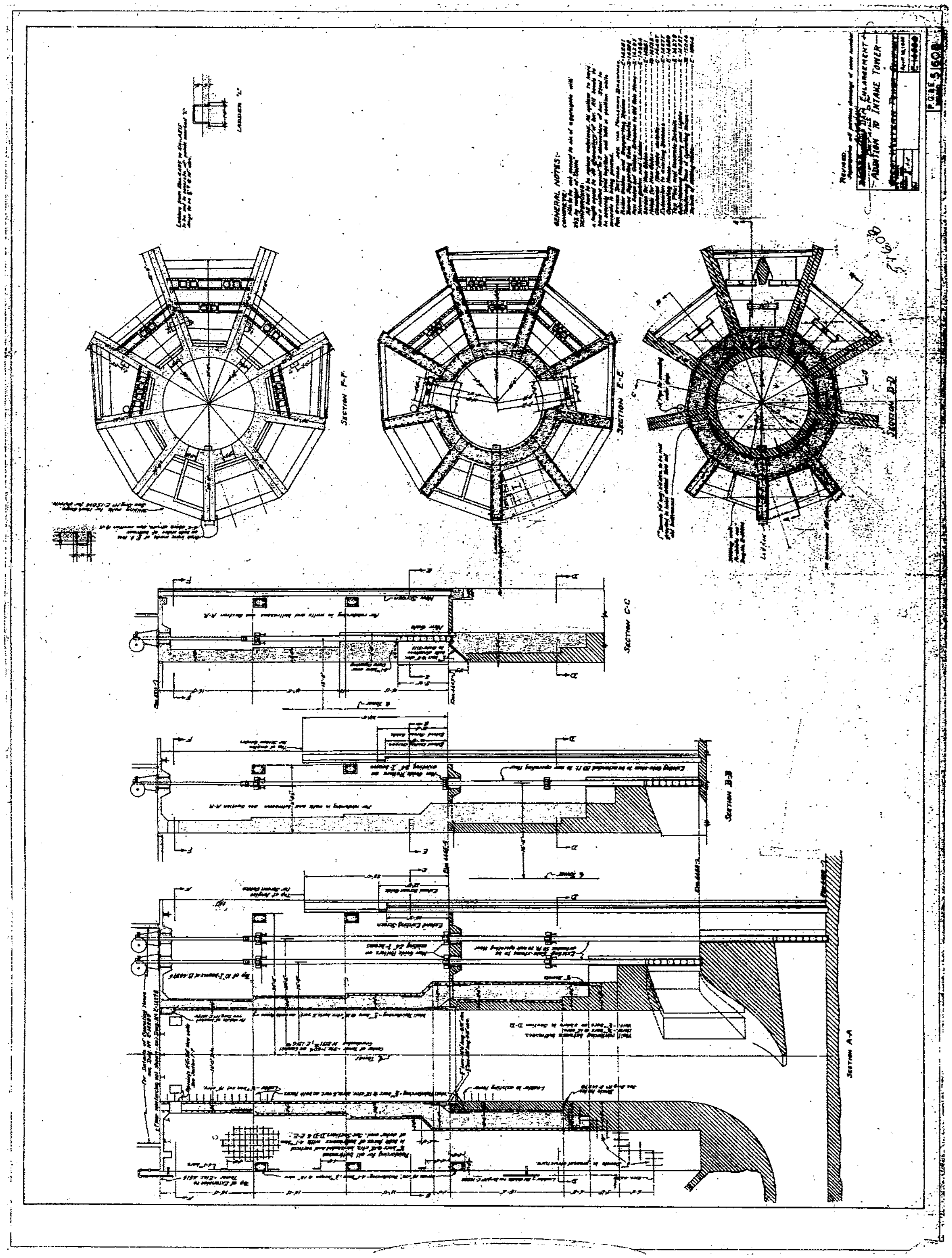

**Figure 1-4**
**Lake Almanor Intake Tower, As-Built 1927 Addition**

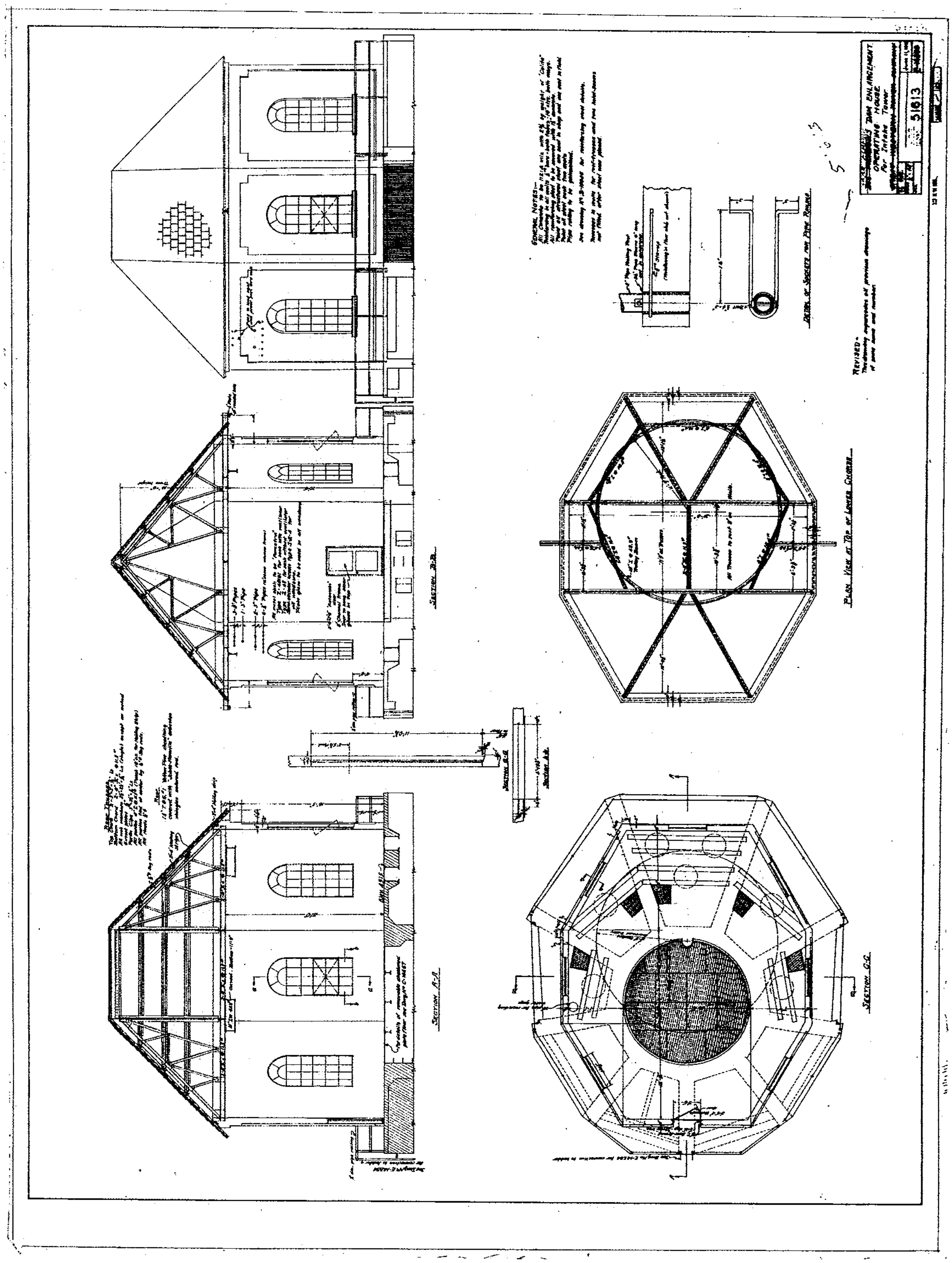

**Figure 1-5**
**Lake Almanor Intake Tower, As-Built 1927 Addition, Valve House**

# 2 MODEL DESCRIPTION

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## Model Description

A full 3D finite element model of the intake tower was developed using 8-Node continuum (brick) elements using the ABAQUS [2] general-purpose finite element code. The base of the model is at elevation 4422' and the top of the model is at elevation 4515'. The model was developed such that distinct element groups were assigned to the original 1914 structure and to the subsequent addition in 1927. At the base of the model, below elevation 4422' there is a thin six-inch tall row of elements that have a preset crack in the horizontal plane. This layer simulates a no tension interface allowing uplift of the tower during the pushover evaluation. The 6" floor slab for the valve house at the top of the tower is explicitly modeled, but the valve house is represented with a cantilever beam and lumped mass. The weight of the valve house was assumed to be 300 kips. The stiffness of the beam was back calculated to achieve a frequency of 25 Hz . The 18"x30" horizontal struts between buttresses were modeled with beam elements.

The "ceiling" of the inlet gates is reduced through the thickness of the main cylinder wall from 9' above elevation 4422' at the outside surface to 6' above elevation 4422' at the inside surface. The vertical members of the gate frame are simulated with truss elements from elevation 4422' to 4436'. The inlet gate track is approximately 25 square inches and assumed to be adequately bonded to the sides of the inlet gate openings. The truss elements only act in the axial direction of the elements and do not provide any shear or bending restraint. Nonlinear steel material properties were assigned to the frame elements. The full continuum element model along with the distinct 1914 and 1927 sections is shown in Figure 2-1.

## Concrete Material Properties

The ANACAP-U Nonlinear Concrete Material Model [3] is coupled with the ABAQUS finite element code for the analyses. The material model includes the effects of concrete cracking, shear retention across cracks due to aggregate interlock, shear stiffness and strength of the cracked concrete as a function of crack opening width, and compressive yielding with subsequent strain softening of concrete. As recommended by the technical review board, the concrete strength, fc', defined for these analyses was 3000 psi for the 1914 structure and 4000 psi for the 1927 structure, [4]. The mass of the structure is calculated at the integration point level based on the weight density of the concrete. Table 2-1 lists the material properties assigned for the as-built concrete in the models. The monotonic compressive stress versus compressive strain behavior for the concrete model using fc' of 3000 and 4000 psi are presented in Figures 2-2 and 2-3.

| Material Property | 1914 Concrete | 1927 Concrete |
|---|---|---|
| f'c | 3000 psi | 4000 psi |
| E | 3.122e+06 psi | 3.605e+06 psi |
| $\nu$ | 0.15 | 0.15 |
| $\rho$ | 150 lb/ft$^3$ | 150 lb/ft$^3$ |

**Table 2-1**
**Material Properties Used In Concrete Constitutive Models**

## Steel Material Properties

All the steel reinforcement was included in the model as sub-elements embedded within the concrete elements at the appropriate locations. This includes all the vertical and hoop bars within the main tower wall, the vertical and transverse bars in the buttresses, the horizontal bars in the top slab, and the horizontal 30$^\#$ rails above the water inlet openings at the base of the model.

The ANACAP-U nonlinear steel model was used to model all of the steel components within the model. This captures yielding, strain hardening, and cyclic Bauschinger effects. The steel is assumed to be a typical A30 type steel. The uniaxial stress-strain curve for the material model is illustrated in Figure 2-4.

## Loading And Boundary Conditions

The pushover analyses are performed to determine the capacity of the structure as opposed to demands due to a seismic event. The hydrodynamic effect of the surrounding reservoir was not required for the capacity analysis.

The tower model included a six-inch tall layer of base elements that was pre-cracked across a level horizontal plane. This base was fixed in the vertical DOF and the two horizontal DOF below the horizontal pre-crack. The horizontal pre-crack provides a no tension interface that allows vertical uplift, compressive stiffness, and shear transfer across the closed crack (compression zone). No boundary conditions were imposed at the top of the tower. The force pushover analyses were performed by applying a linearly increasing concentrated force on the tower at approximately 1/3 the height from the base of the model, at elevation 4453'.

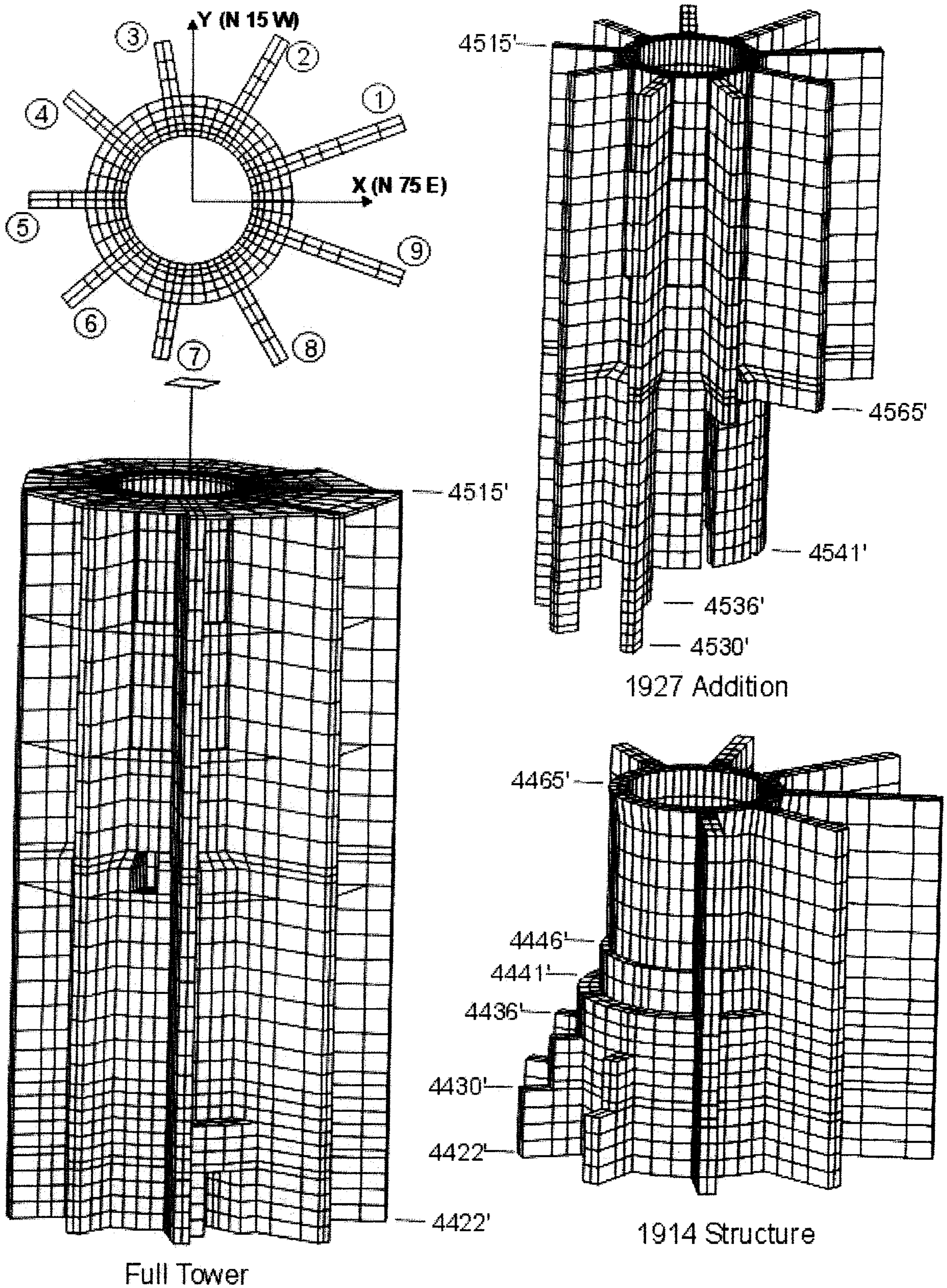



**Figure 2-1**
**Lake Almanor Intake Tower, 3D Finite Element Model**

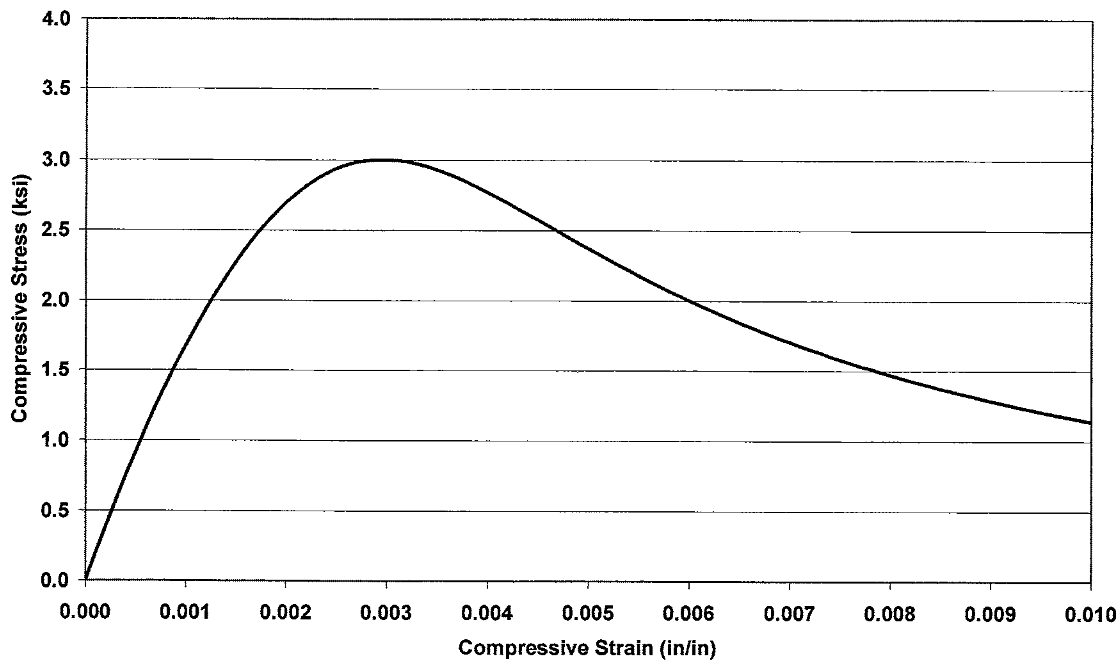



**Figure 2-2**
**Compressive Stress Versus Strain Behavior, fc' = 3000 psi**

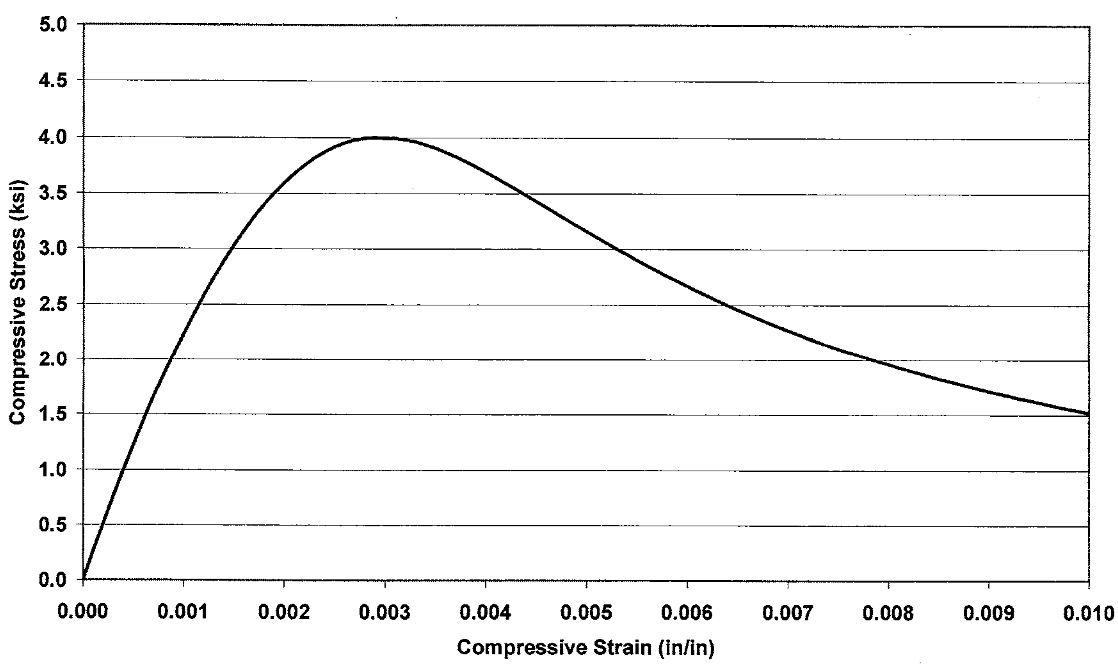



**Figure 2-3**
**Compressive Stress Versus Strain Behavior, fc' = 4000 psi**

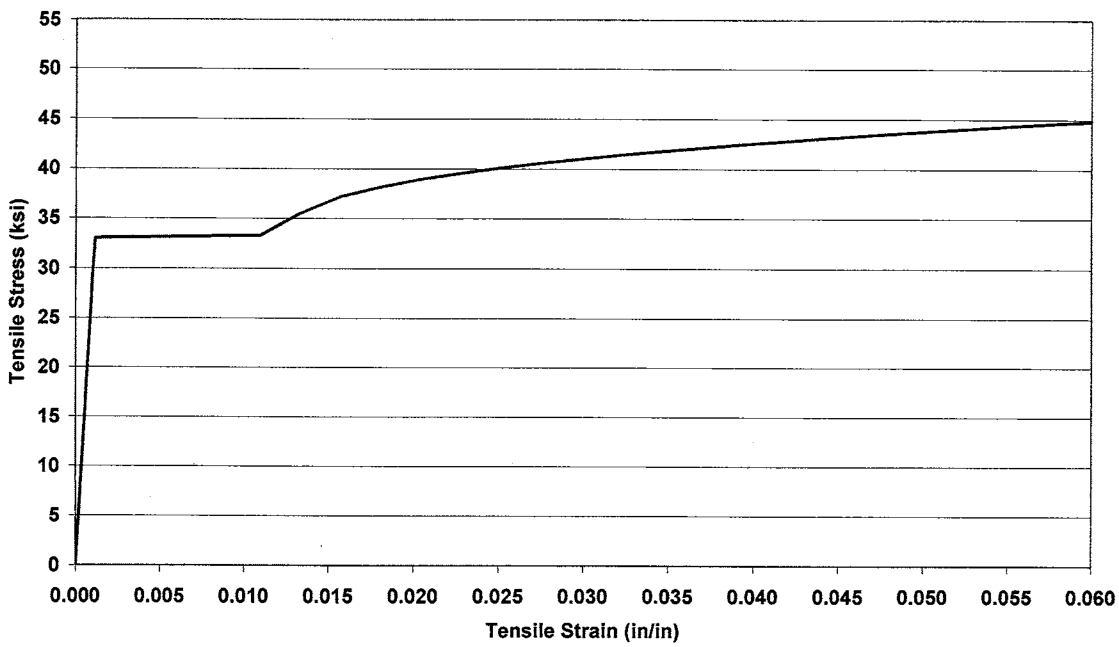



**Figure 2-4**
**Rebar Steel Model Monotonic Stress Strain Behavior**

# 3
# PUSHOVER ANALYSIS

In accordance to FEMA-356, a nonlinear time history analysis was performed on the intake tower [1]. This analysis showed that the primary response of the tower is rigid body rocking at the tower base. A nonlinear static analysis was performed as a force pushover analysis to further characterize the performance of the structure in accordance to FEMA-356 (section 2.4.2.1). A linearly increasing force was applied to the tower at elevation 4453', 1/3 the height of the model. Two pushover analyses were performed, applying the force in the two primary axis directions. Additionally a mass proportional inertia pushover was performed to confirm the force pushover analysis.

## Strong Axis Push Results

For the strong axis pushover the force was applied in the global X direction (N 75 E), as shown in Figure 2-1. The results are presented in Figures 3-1 through 3-10.

Figure 3-1 plots the base shear (applied force) versus the tower displacement at elevation 4515'. The base shear capacity is approximately 5080 kips and the analysis was stopped at a displacement of 5.5 inches. The vertical dashed line indicates the target displacement demand as calculated from the SAP2000 time history analysis. At this level of displacement the tower shear capacity is 5080 kips. Figure 3-2 presents the vertical displacement at the ends of the buttress plotted versus the tower displacement. The base uplift is 2.02 inches at a target displacement of 5.1 inches. Figures 3-3 and 3-4 present the inlet gate opening deformation across the diagonal and vertically versus the tower displacement respectively. The inlet gate deformations do not exceed 0.025 inches.

The vertical strain and stress profile at the end of buttress 1 through the entire height of the tower at a top displacement of 5.1" are shown in Figures 3-5 and 3-6. These plots show that the peak demand at the end of the buttress occurs within 10' of the bottom of the tower. Figures 3-7 through 3-10 show stress and strain contour plots near the base of the tower. The vertical stress component at a lateral tower top displacement of 5.1 inches is shown in Figure 3-7. Figure 3-8 shows the compressive principal stress at the same displacement. These plots show peak compressive stresses approximately 2700 psi at the ends of buttresses 1 and 9. Figures 3-9 and 3-10 show the vertical strain component and principal strain at 5.1 inches tower displacement. The vertical strain reaches –0.0057 in/in and a principal strain of –0.0059 in/in develops at the very end of buttresses 1 and 9. At a concrete compressive strain of -0.003 in/in the material reaches its maximum strength, at larger strains the material model will begin to soften. At this displacement level these figures indicate that softening will occur at the ends of the buttresses and the stresses will redistribute inward along the buttress axis. The high compression zone occurs in buttresses 1 and 9 and does not spread into the main cylinder wall or around the inlet gate openings.

## Weak Axis Push Results

For the weak axis pushover the force was applied in the global Y direction (N 15 W), as shown in Figure 2-1. The results are presented in Figures 3-11 through 3-20.

Figure 3-11 plots the base shear (applied force) versus the tower displacement at elevation 4515'. The base shear capacity is approximately 4140 kips and the analysis was stopped at a displacement of 6.0 inches. The vertical dashed line indicates the target displacement demand as calculated from the SAP2000 time history analysis. At this level of displacement the tower shear capacity is 4140 kips. Figure 3-12 presents the vertical displacement at the ends of the buttress plotted versus the tower displacement. At the displacement demand of 4.9 inches the uplift is 1.66 inches at the end of buttress 8. The base uplift sustained by the tower is 2.98 inches at a tower displacement of 6.0 inches. Figures 3-13 and 3-14 present the inlet gate opening deformation across the diagonal and vertically versus the tower displacement respectively. The inlet gate deformations do not exceed 0.05 inches.

The vertical strain and stress profile at the end of buttress 3 through the entire height of the tower at a top displacement of 5.08" are shown in Figures 3-15 and 3-16. Similar to the strong axis pushover, the peak demand is occurring in the bottom 10' of the tower. Figures 3-17 through 3-20 show stress and strain contour plots near the base of the tower. The vertical stress component at a tower displacement of 5.08 inches is shown in Figure 3-17. Figure 3-18 shows the compressive principal stresses at 5.08 inches. These plots show peak compressive stresses less than 3000 psi at the ends of buttresses 2 and 3. Figures 3-19 and 3-20 show the vertical strain component and principal strains at 5.08 inches tower displacement. The largest vertical strain of –0.0075 in/in and principal strain of –0.0081 in/in develops at the very end of buttresses 2 and 3. The high compression zone occurs in buttresses 2 and 3 and does not spread into the main cylinder wall or around the inlet gate openings.

For both the strong axis and weak axis pushover analyses the stress in the tower above the lower gates are small and within the allowable range.

## Rigid Body Deformation

The deformation of the intake tower is primarily a rigid body rocking motion. Figures 3-21 and 3-22 show uplift displacement profiles along the base of the tower at increasing levels of tower top displacement for the strong and weak axis pushover analyses. The vertical dashed lines indicate the location of the outside surface of the main cylinder wall. These plots indicate a linear uplift displacement from the end of one side across the main tower section with softening constrained in the opposite side buttress region only. Figure 3-23 shows deformed grid plots from both pushover analyses at a displacement of approximately 5 inches, magnified by a factor of 25. It is apparent from the deformed shape of the tower that the displacement is primarily rigid body rocking.

It should be noted that large displacement kinematics (P-Δ) were not activated for these analyses. Typically P-Δ effects should be considered in structural analyses of slender structures, i.e. bridge columns, high-rise buildings. The Lake Almanor intake tower is a stout structure, approximately 43'x 93' as modeled. As assumed for these analyses the mass is acting at approximately 1/3 the height of the tower. The lateral displacement is predominately due to rigid body rocking of the structure, thus a linear distribution can be assumed through the height. At the 1/3 point the displacement is approximately 1.7 inches and the induced P-Δ moment is approximately 2% of the total base moment. This effect is insignificant to the overall response of the structure.

## Mass Proportional Inertia Pushover

A nonlinear dynamic mass proportional inertia (MPI) pushover was performed on the tower to confirm the results of the static force pushover analyses. For this analysis the model was subjected to a linearly increasing acceleration at the base. Performing the pushover analysis in this manner allows the structure to behave in its preferred mode as damage occurs, i.e. nonlinear tensile cracking and/or compressive softening of the material, the load within the structure will redistribute appropriately.

The MPI loading methodology confirmed that the primary behavior of the tower is rigid body rocking. Figure 3-24 presents a plot comparing the base shear versus the relative tower top displacement from the MPI and static force pushover analyses in the strong axis direction. This figure shows that the two different loading methodologies induce very similar base shear displacement responses of the structure. The MPI methodology has a slightly softer response as the tower transitions from the elastic to the nonlinear range. This can be attributed to the more uniform distribution of lateral load on the tower as provided by the MPI methodology.

Figures 3-25 and 3-26 show plots of the inlet gate deformation in the vertical direction and across the diagonal as a function of the tower top displacement. These show that the gate deformations remain well below the allowable gap spacing at the target displacement of 5.2"

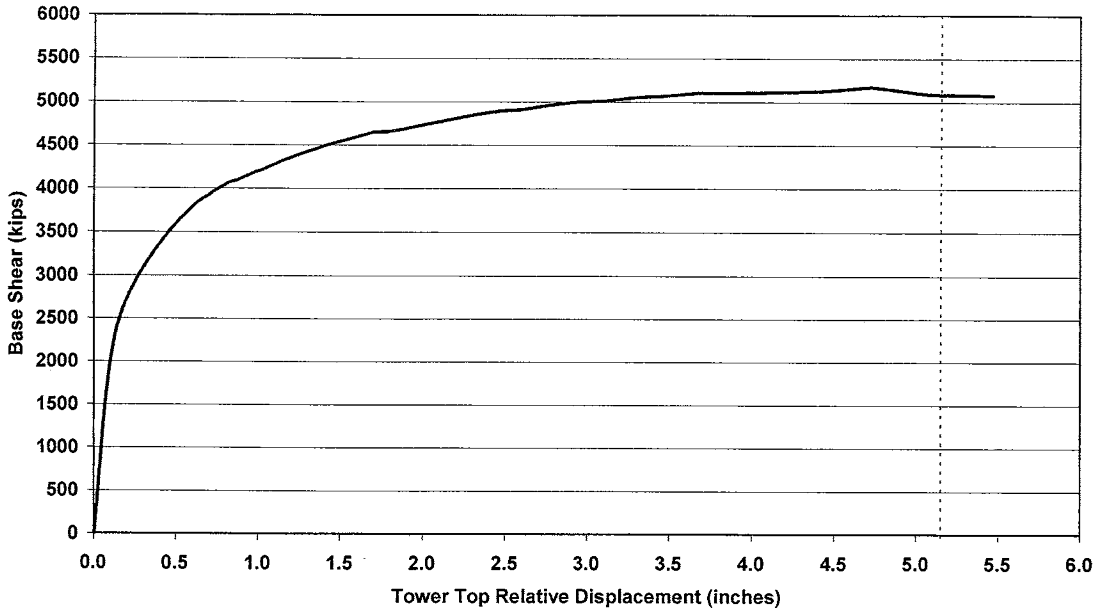



**Figure 3-1**
**Base Shear Versus Tower Displacement For Strong Axis Pushover**

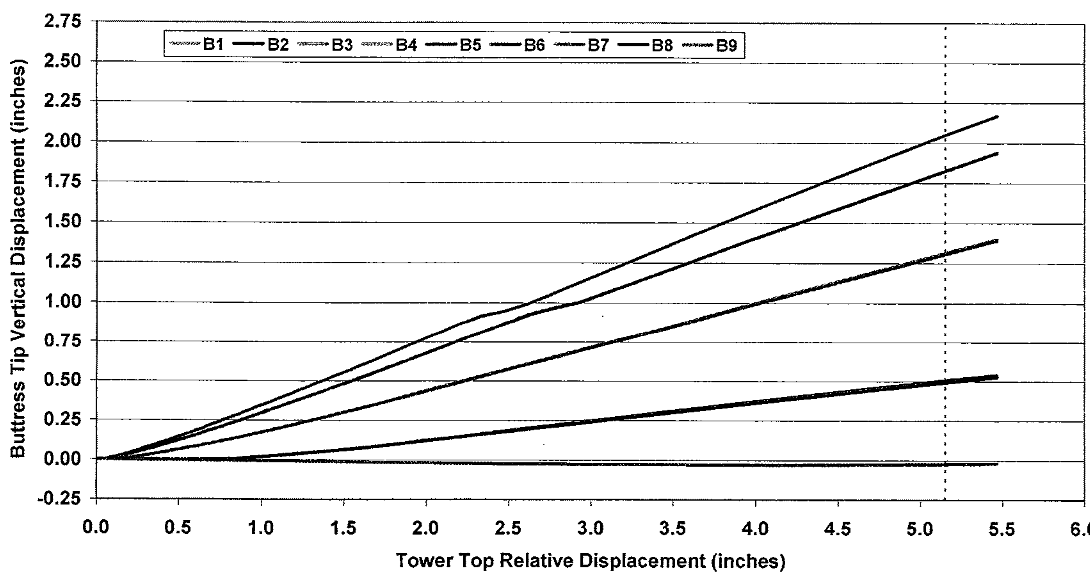



**Figure 3-2**
**Buttress Tip Uplift Versus Tower Displacement For Strong Axis Pushover**

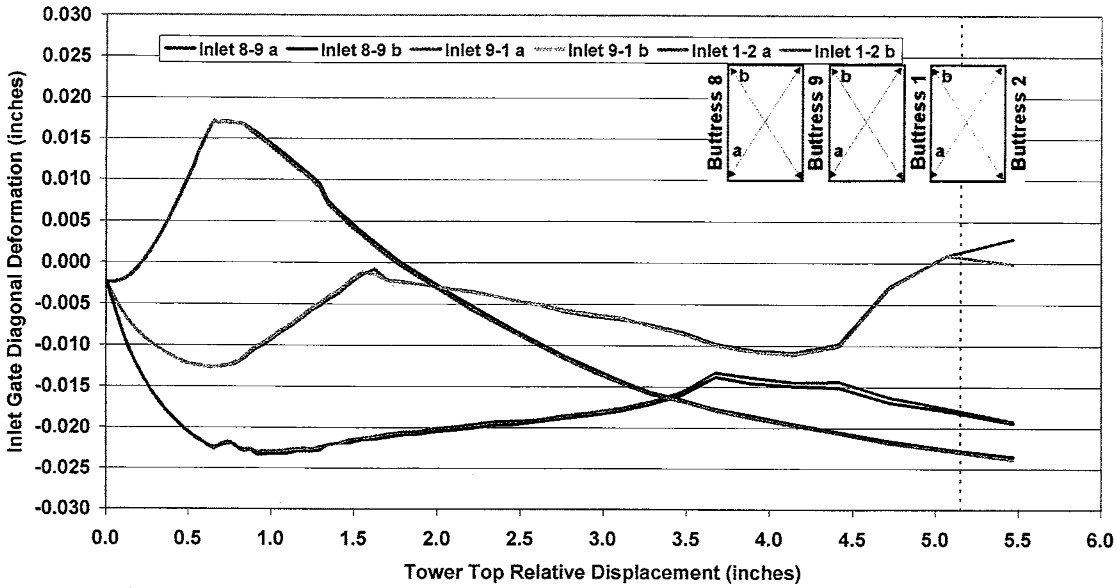



**Figure 3-3**
**Inlet Gate Diagonal Deformation Versus Tower Displacement For Strong Axis Pushover**

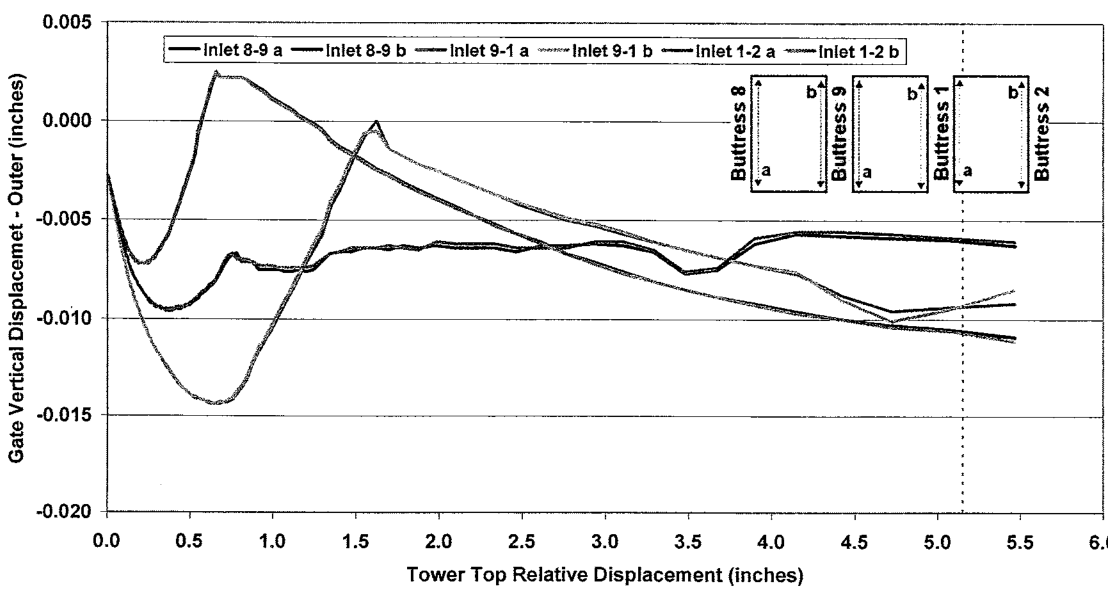



**Figure 3-4**
**Inlet Gate Vertical Deformation Versus Tower Displacement For Strong Axis Pushover**

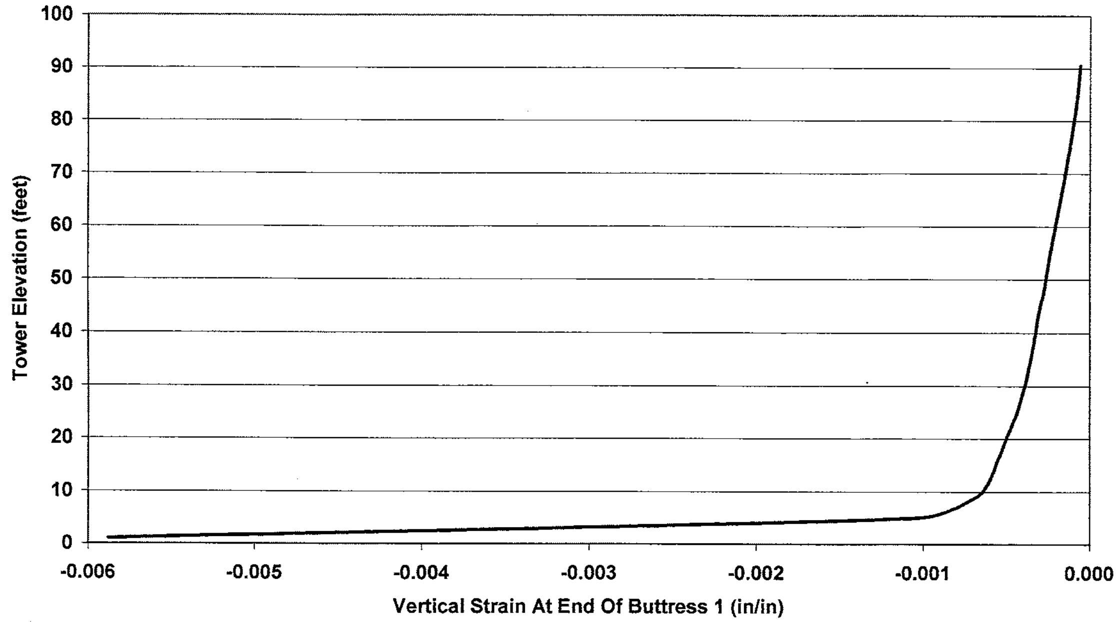



**Figure 3-5**
**Vertical Strain Through Height Of Tower At End Of Buttress 1, Tower Displacement, 5.1" Strong Axis Pushover**

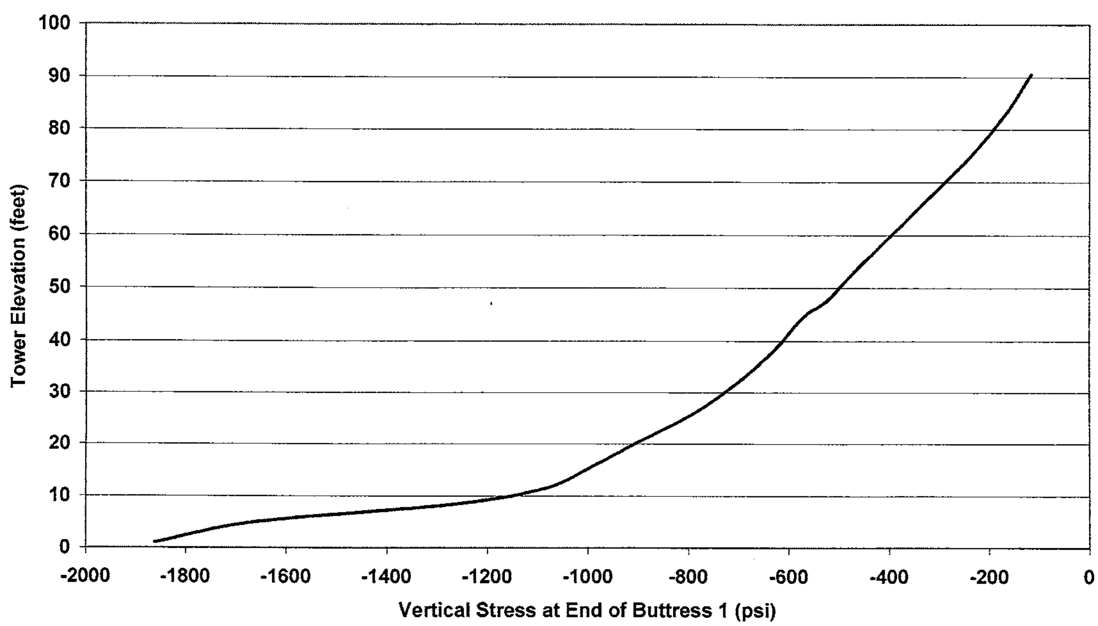
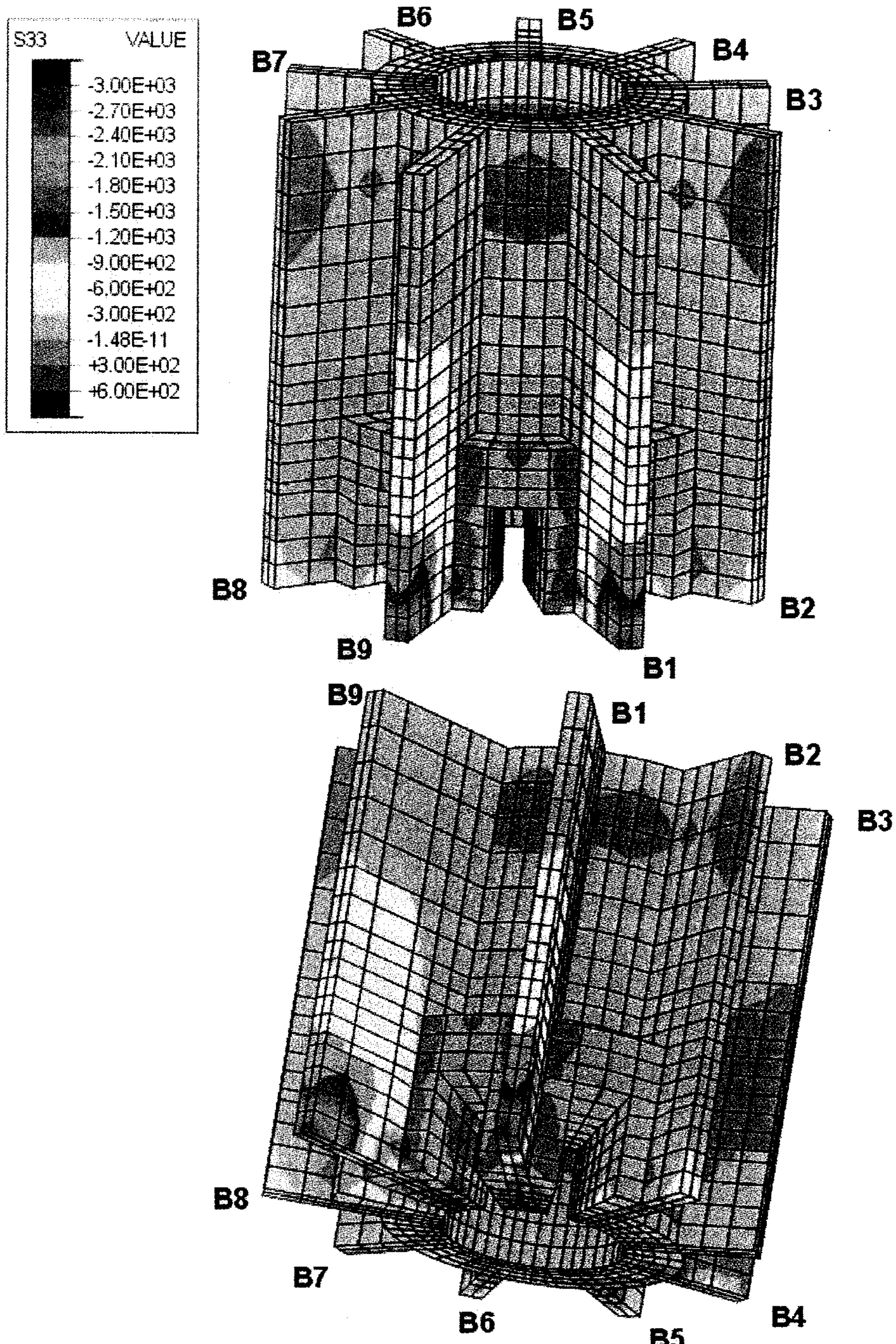



**Figure 3-6**
**Vertical Stress Through Height Of Tower At End Of Buttress 1, Tower Displacement, 5.1" Strong Axis Pushover**




**Base Shear = 5080.0 kips**
**Base Moment = 165000.0 kip-ft**
**Top Displacement = 5.10"**

**Figure 3-7**
**Vertical Stress Contour Plot Of Tower Base At A Top Displacement Of 5.10" For Strong Axis Pushover**

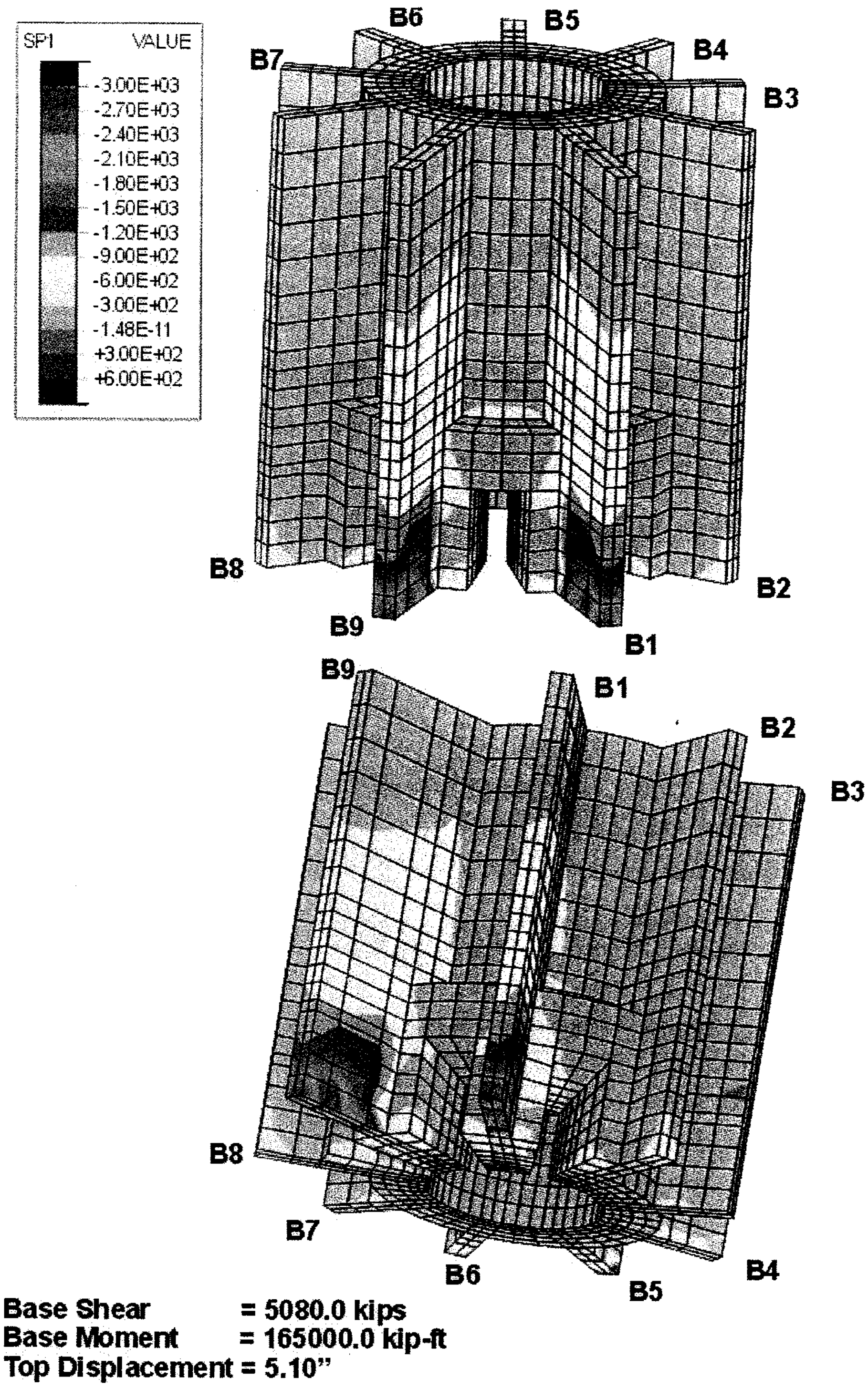



**Figure 3-8**
**Compressive Principal Stress Contour Plot Of Tower Base At A Top Displacement Of 5.10" For Strong Axis Pushover**

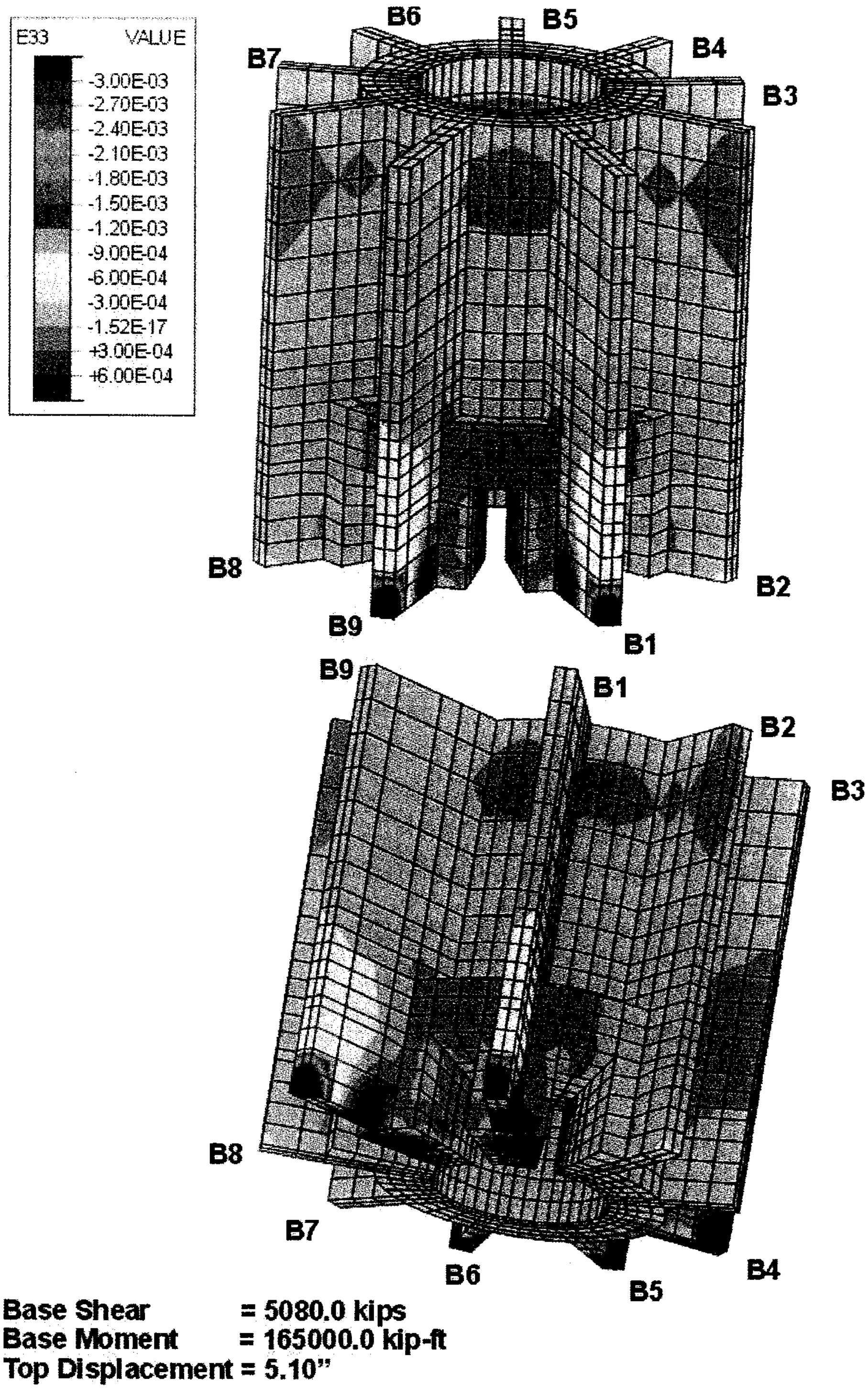
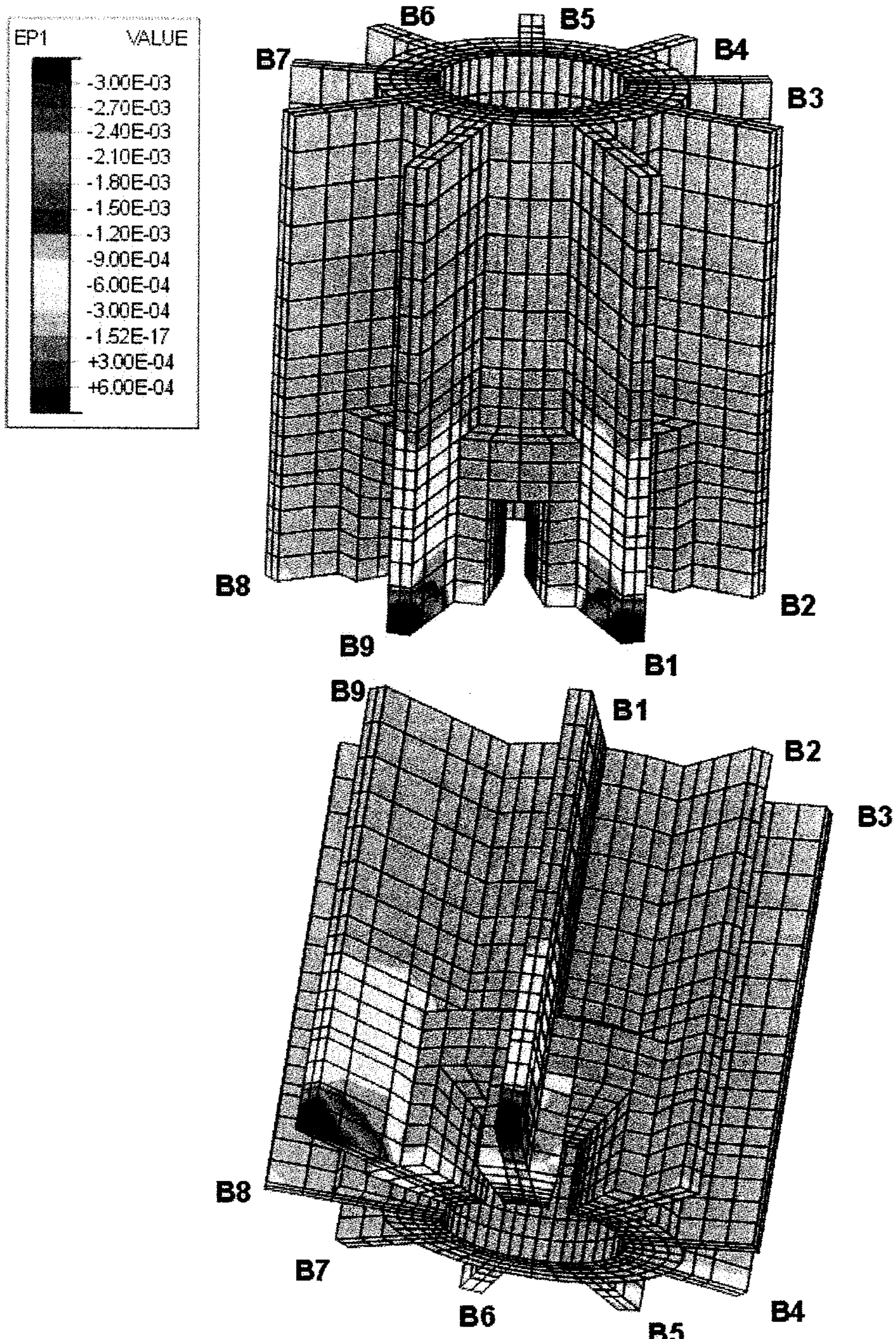



**Figure 3-9**
**Vertical Strain Contour Plot Of Tower Base At A Top Displacement Of 5.10" For Strong Axis Pushover**




**Base Shear = 5080.0 kips**
**Base Moment = 165000.0 kip-ft**
**Top Displacement = 5.10"**

**Figure 3-10**
**Compressive Principal Strain Contour Plot Of Tower Base At A Top Displacement Of 5.10" For Strong Axis Pushover**

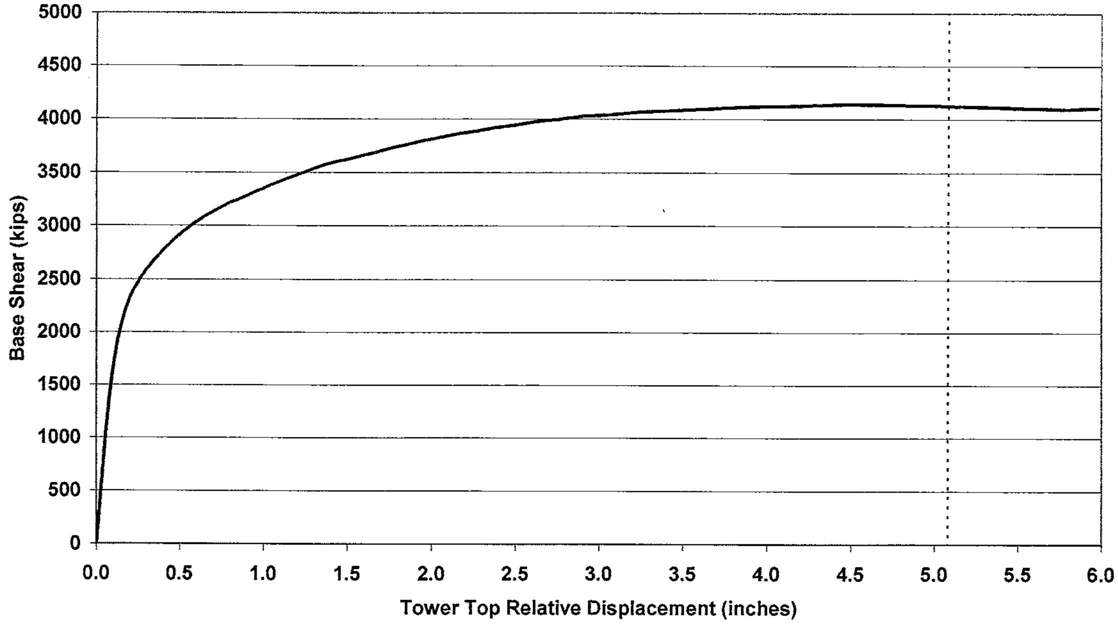



**Figure 3-11**
**Base Shear Versus Tower Displacement For Weak Axis Pushover**

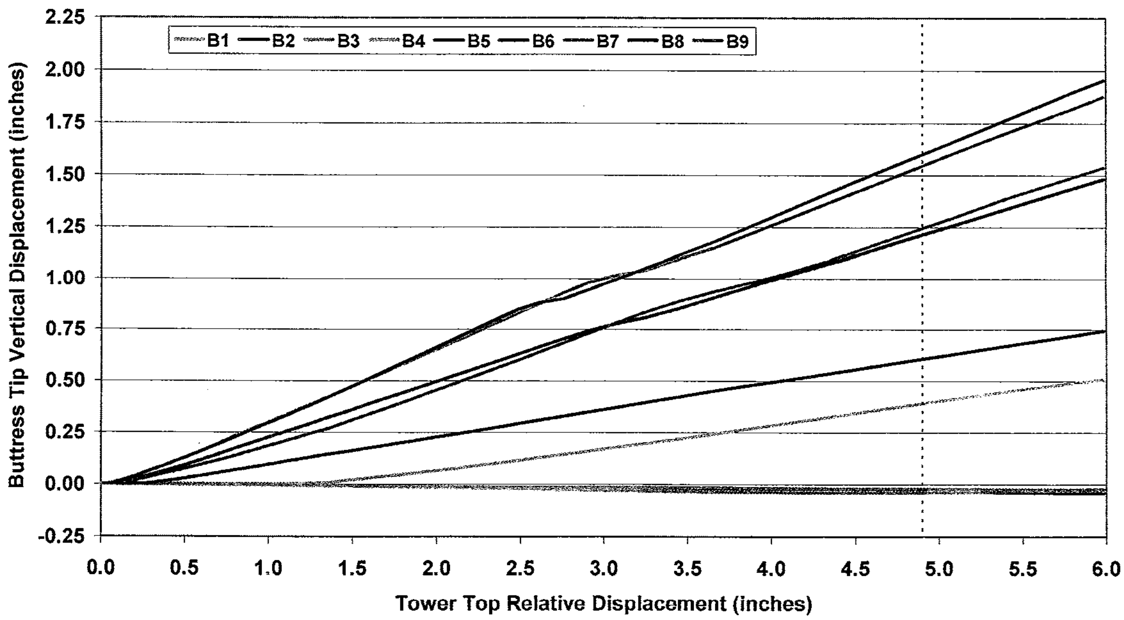



**Figure 3-12**
**Buttress Tip Uplift Versus Tower Displacement For Weak Axis Pushover**

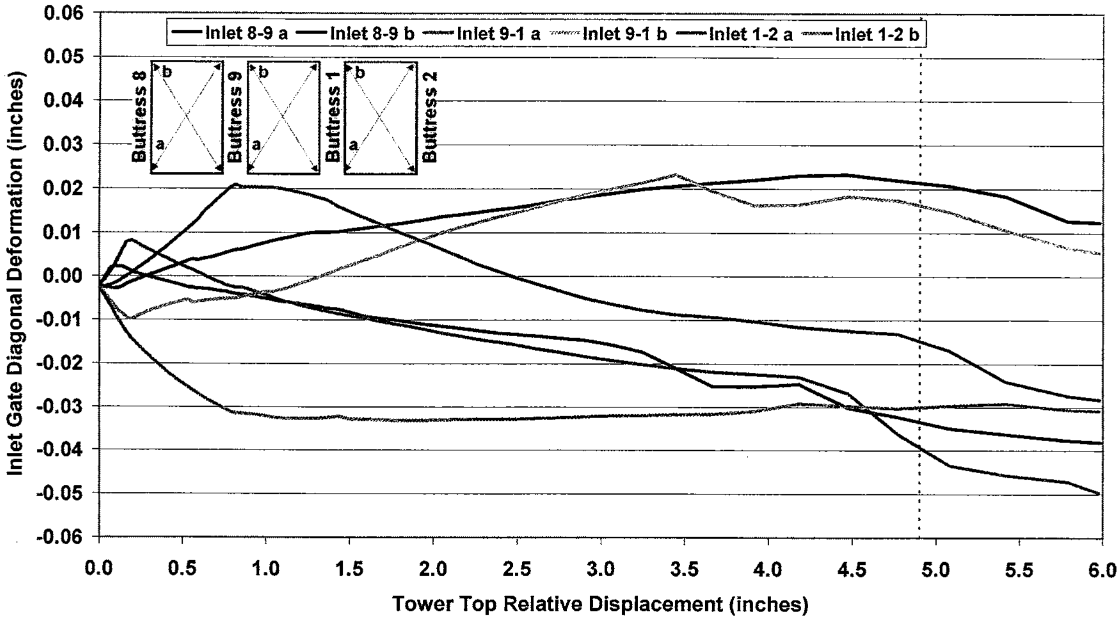



**Figure 3-13**
**Inlet Gate Diagonal Deformation Versus Tower Displacement For Weak Axis Pushover**

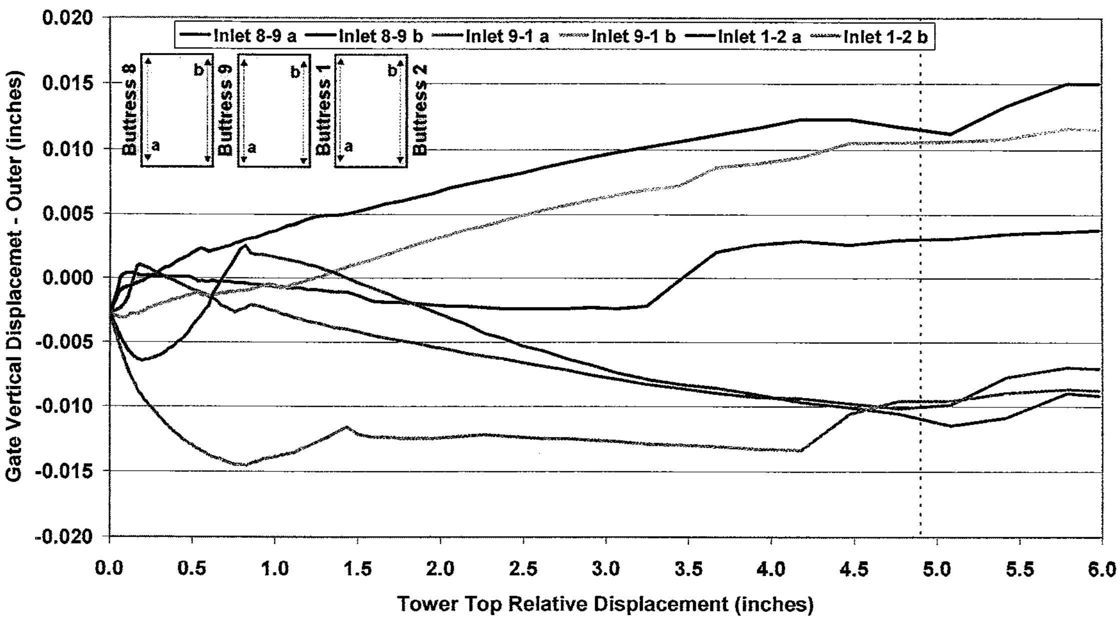



**Figure 3-14**
**Inlet Gate Vertical Deformation Versus Tower Displacement For Weak Axis Pushover**

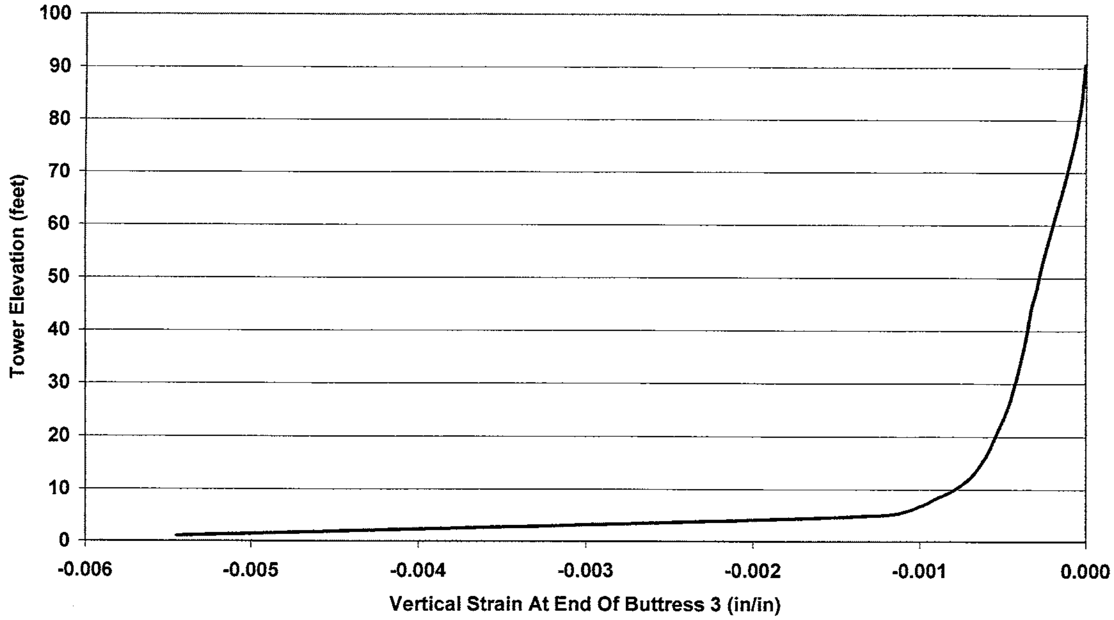



**Figure 3-15**
**Vertical Strain Through Height Of Tower At End Of Buttress 1, Tower Displacement, 5.08" Weak Axis Pushover**

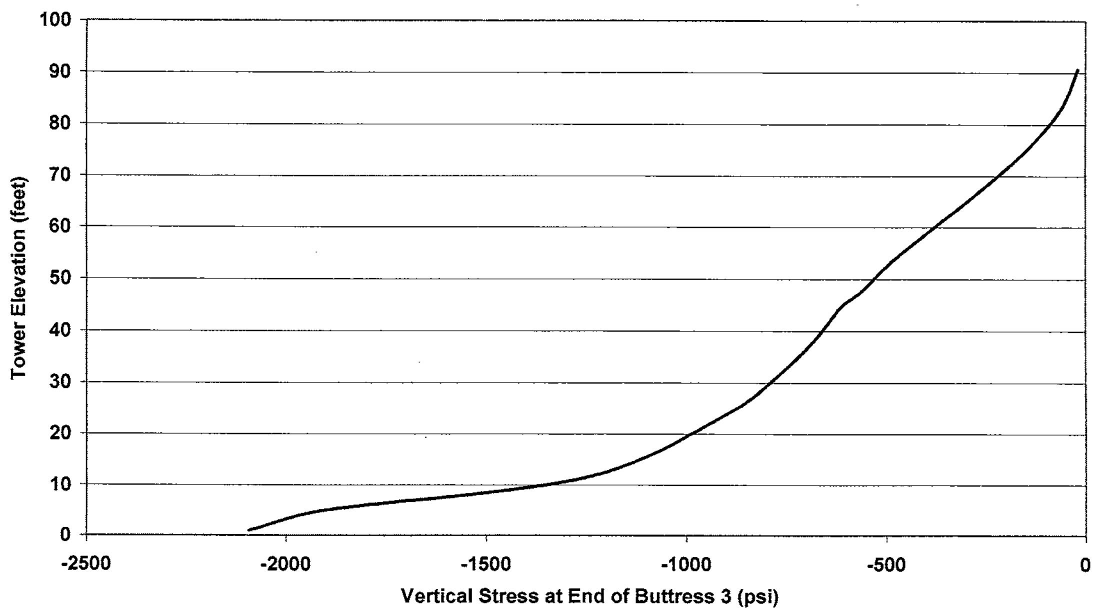
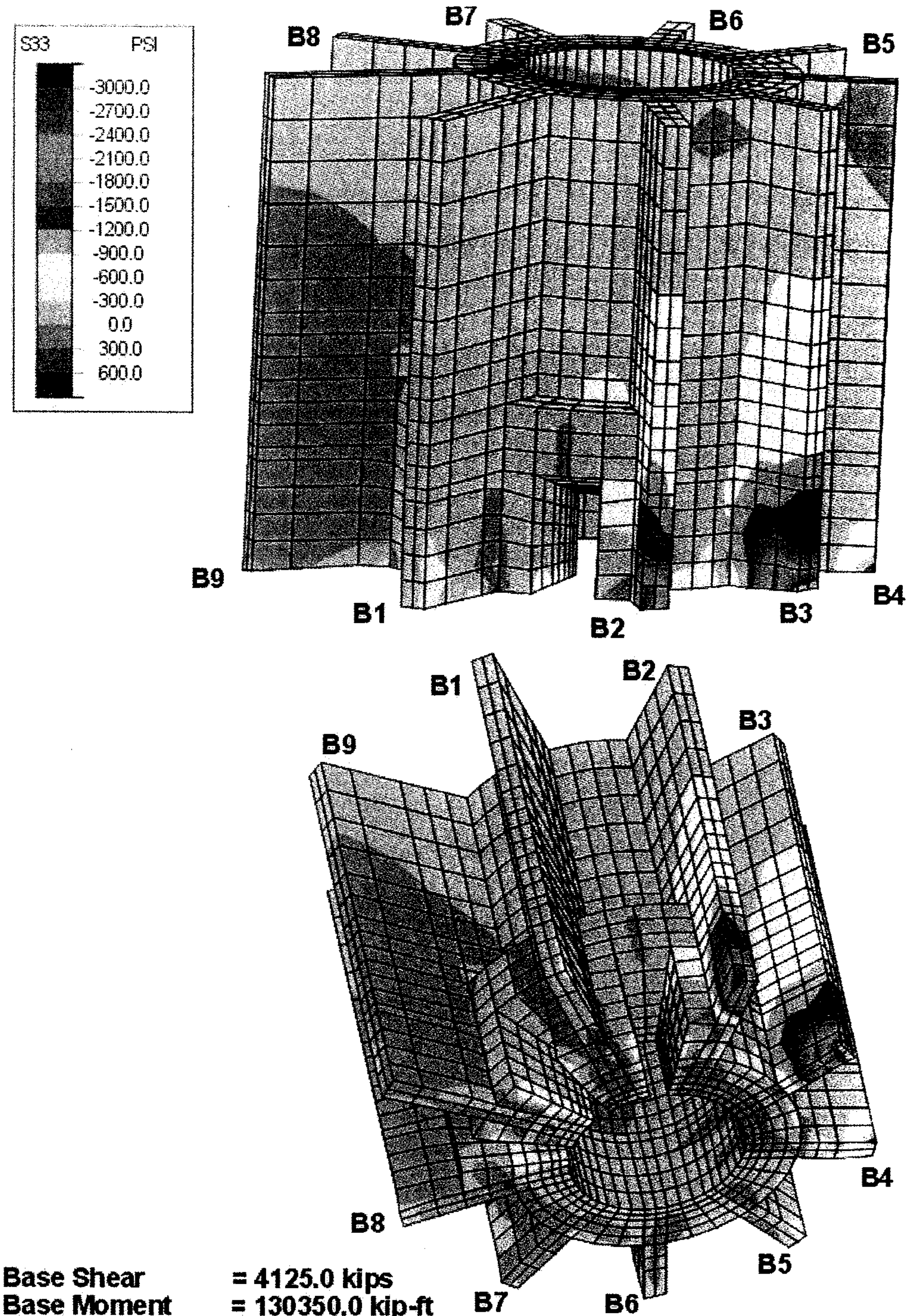



**Figure 3-16**
**Vertical Stress Through Height Of Tower At End Of Buttress 1, Tower Displacement, 5.08" Weak Axis Pushover**




**Base Shear = 4125.0 kips**
**Base Moment = 130350.0 kip-ft**
**Top Displacement = 5.08"**

**Figure 3-17**
**Vertical Stress Contour Plot Of Tower Base At A Top Displacement Of 5.08" For Weak Axis Pushover**

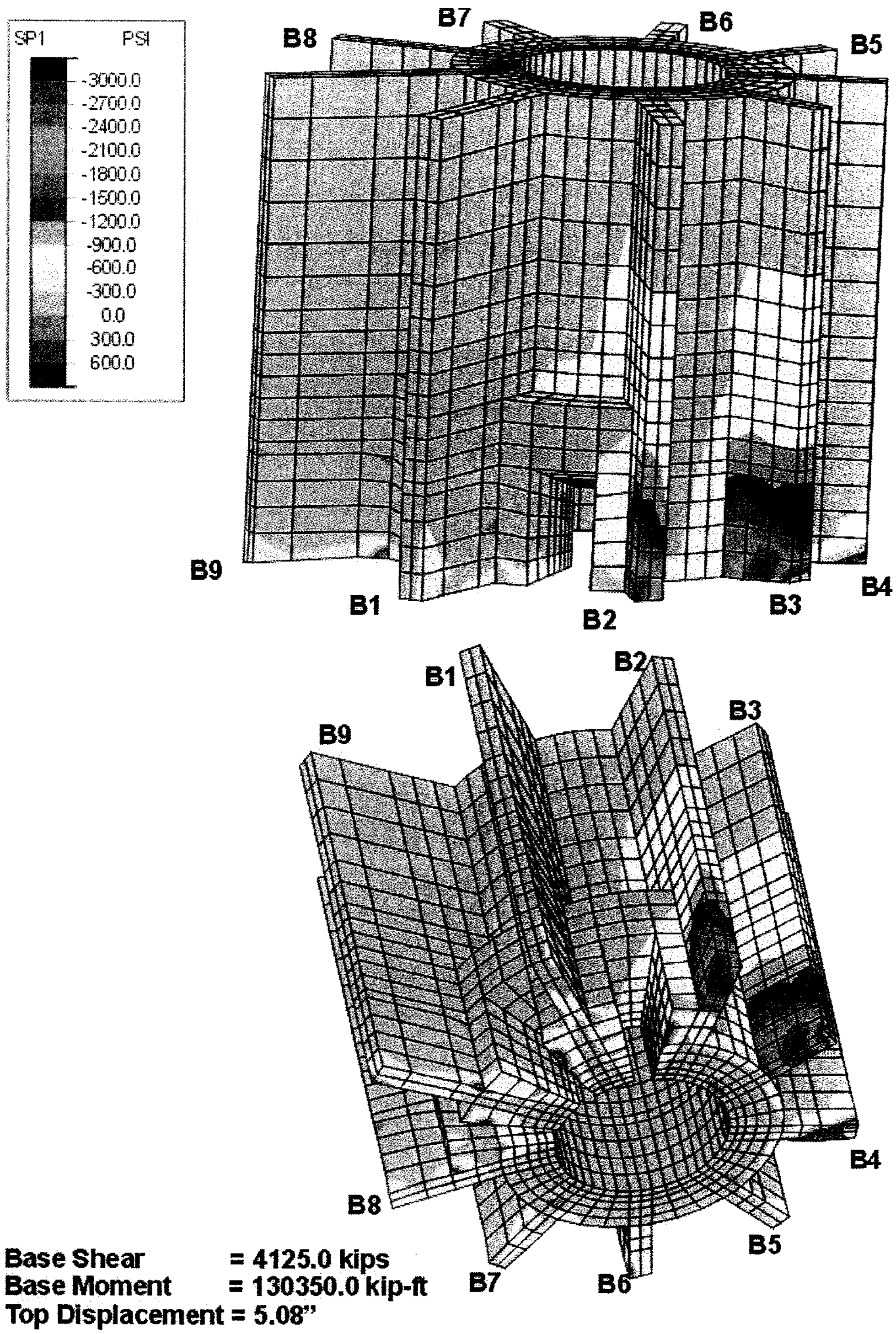



**Figure 3-18**
**Compressive Principal Stress Contour Plot Of Tower Base At A Top Displacement Of 5.08" For Weak Axis Pushover**

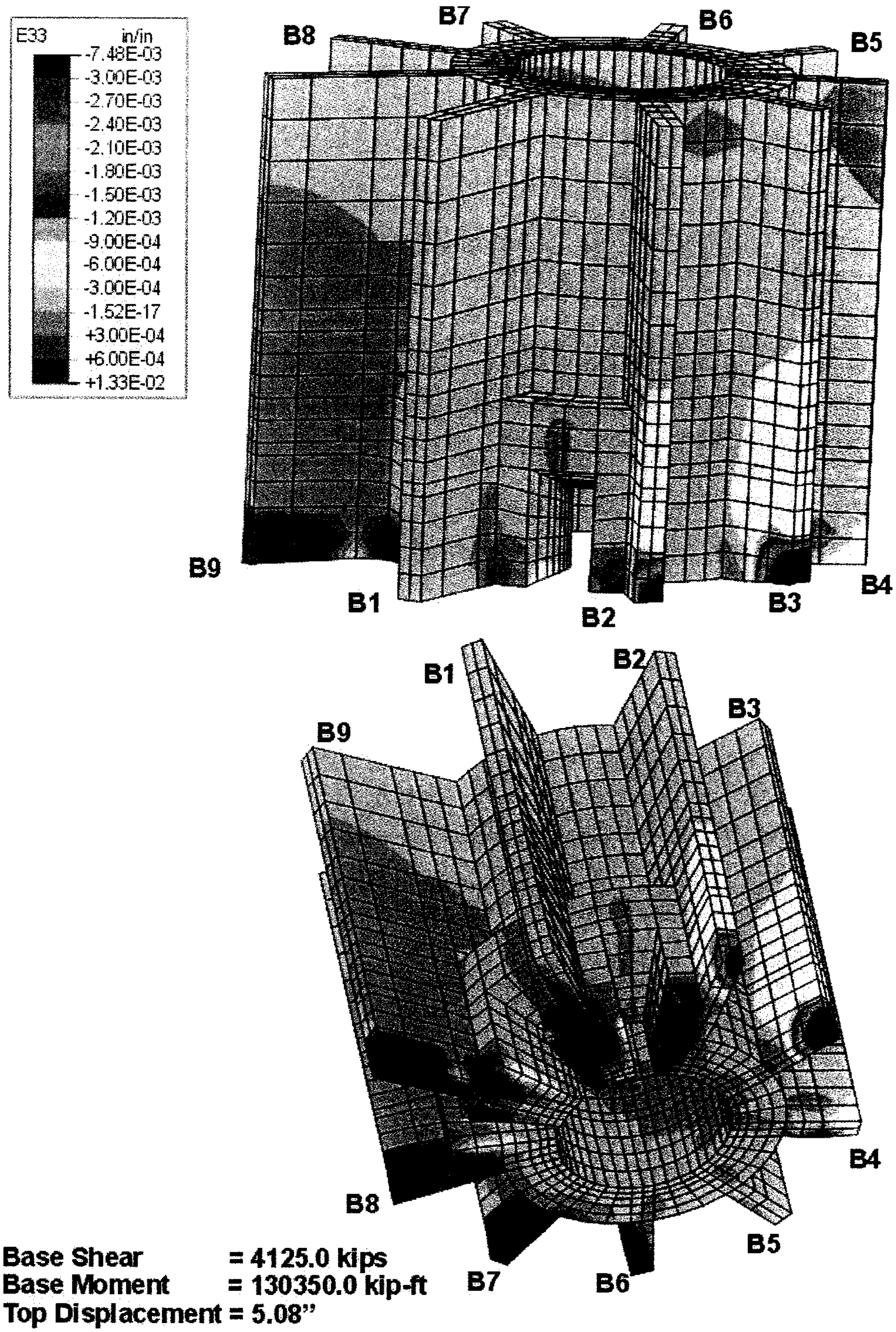
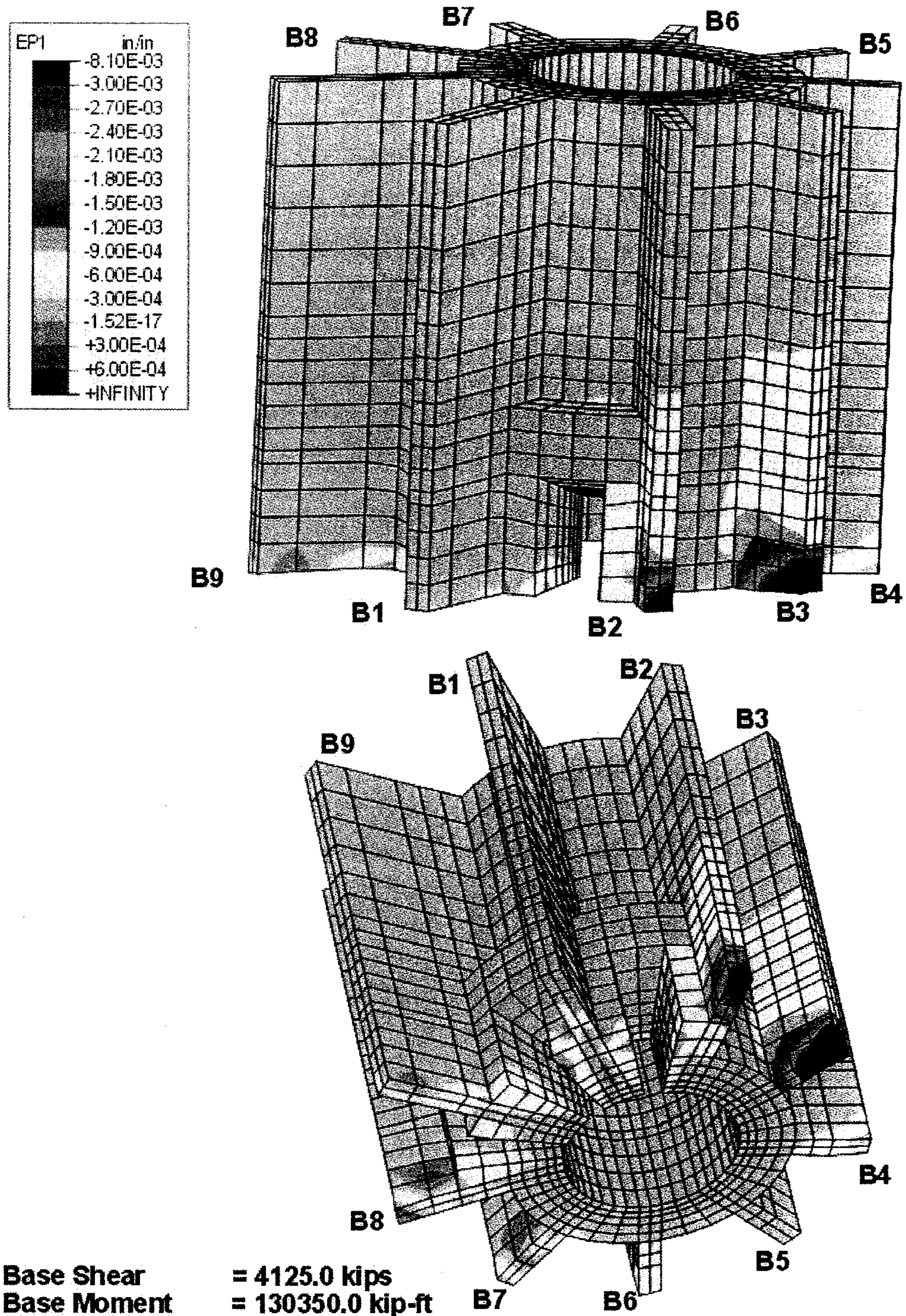



**Figure 3-19**
**Vertical Strain Contour Plot Of Tower Base At A Top Displacement Of 5.08” For Weak Axis Pushover**




**Base Shear = 4125.0 kips**
**Base Moment = 130350.0 kip-ft**
**Top Displacement = 5.08"**

**Figure 3-20**
**Compressive Principal Strain Contour Plot Of Tower Base At A Top Displacement Of 5.08" For Weak Axis Pushover**

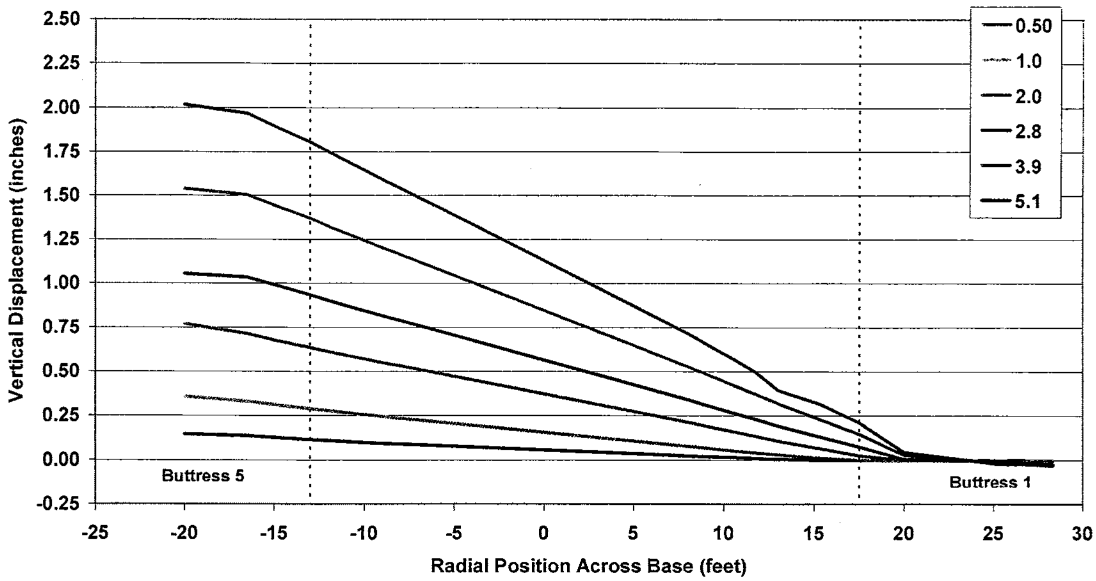



**Figure 3-21**
**Uplift Profiles Across Tower Base At Increasing Tower Top Displacement For Strong Axis Pushover**

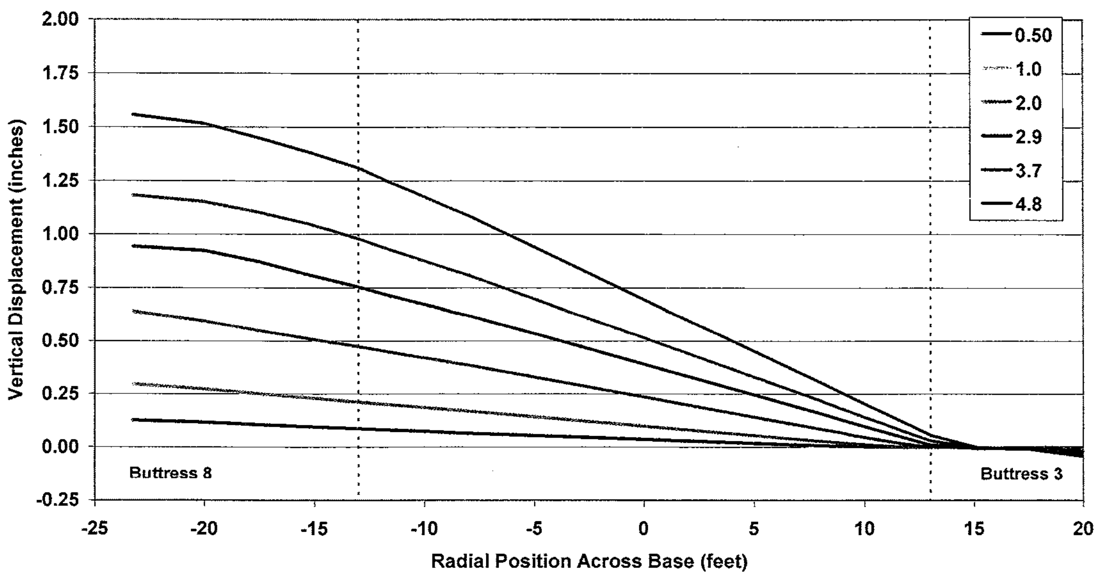
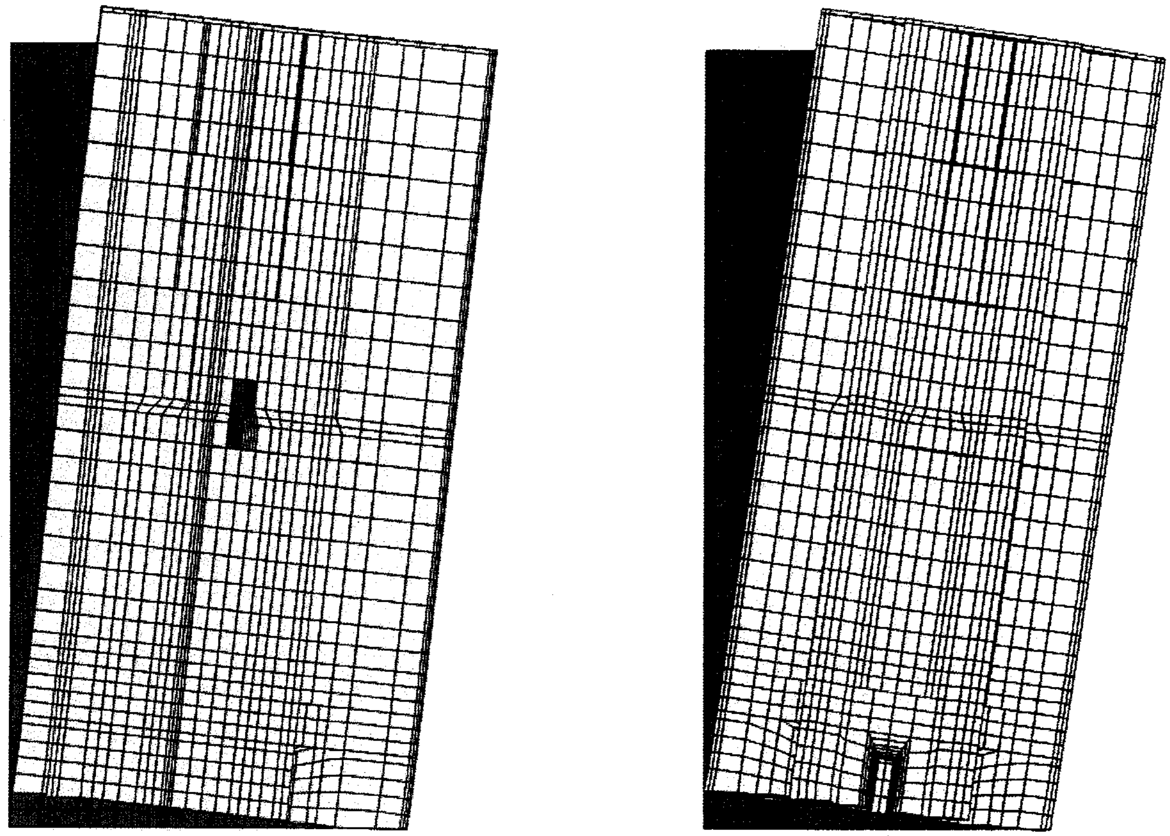



**Figure 3-22**
**Uplift Profiles Across Tower Base At Increasing Tower Top Displacement For Weak Axis Pushover**



**Strong Axis Pushover** **Weak Axis Pushover**

**Figure 3-23**
**Deformed Grid At Approximately 5" Tower Top Displacement, Magnification x 25**

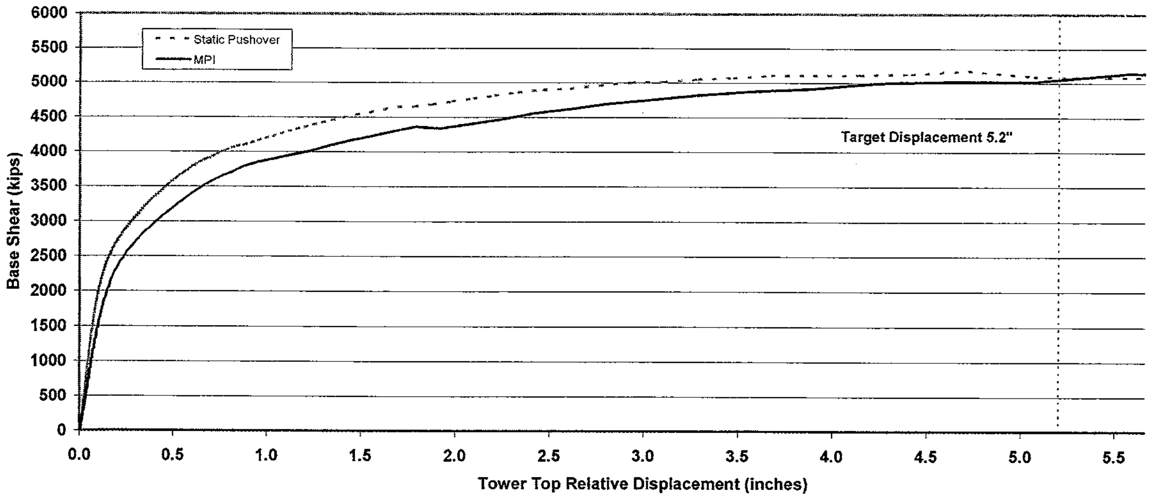



**Figure 3-24**
**Base Shear Versus Tower Displacement In Strong Axis Direction, Comparison Between Static Pushover and MPI Loading Methodologies**

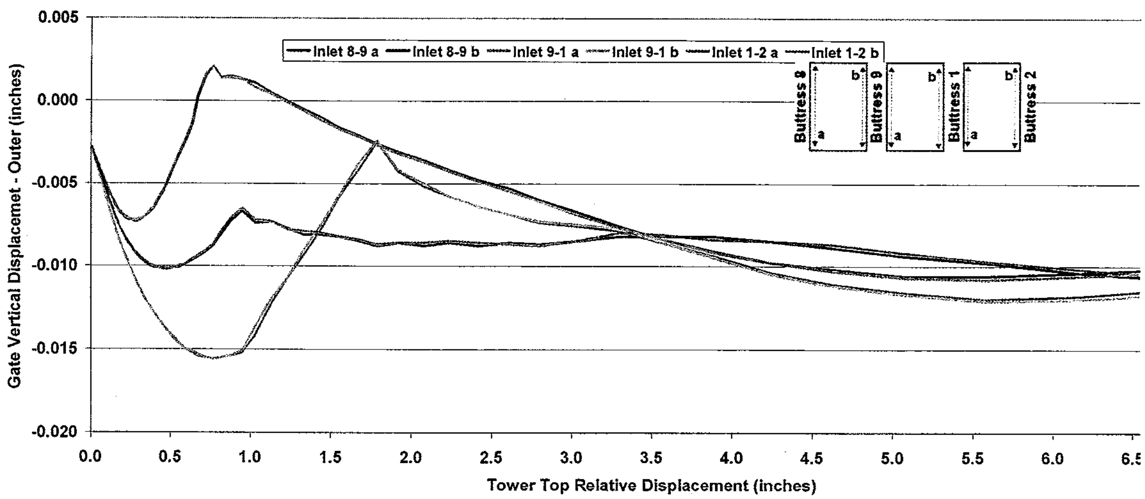



**Figure 3-25**
**Inlet Gate Vertical Deformation Versus Tower Displacement, Strong Axis MPI Pushover**

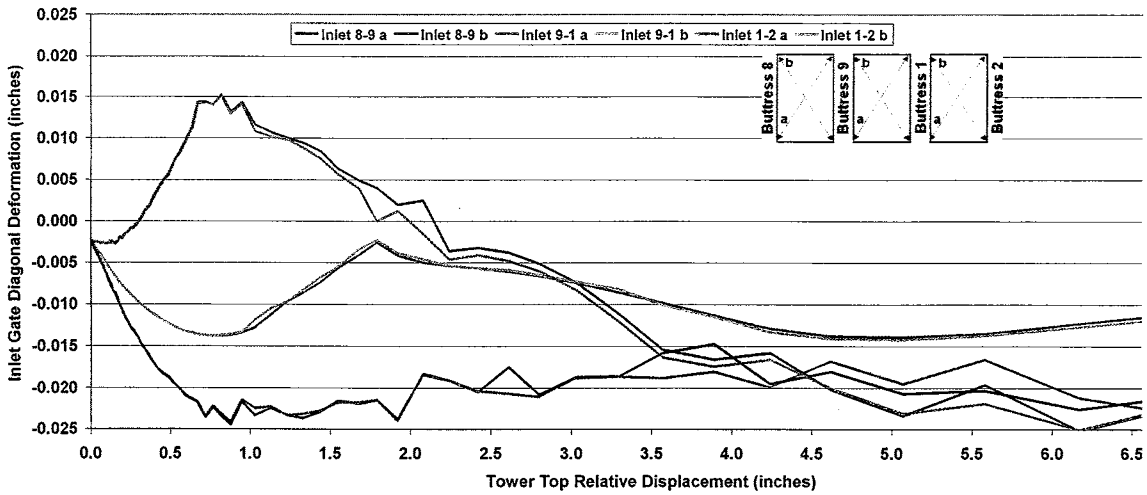



**Figure 3-26**
**Inlet Gate Diagonal Deformation Versus Tower Displacement, Strong Axis MPI Pushover**

# 4
# CONCLUSIONS

## Conclusions

Nonlinear pushover analyses in the two primary directions were performed for the Lake Almanor intake tower. The main objective was to evaluate the capacity and structural integrity of the base of the tower, primarily at the inlet gate openings, at the displacement demand levels as computed with the SAP2000 time history analyses [1]. Table 4-1 summarizes the capacities computed from the nonlinear pushover analyses at the target displacement demands.

| | | **Strong-Axis** | **Weak-Axis** |
|---|---|---|---|
| **$\Delta$H @ 4515' Demand*** | inches | 5.2 | 4.9 |
| **$\Delta$H @ 4515' **** | inches | 5.10 | 5.08 |
| **$V_{base}$ @ 4422'** | kips | 5080 | 4140 |
| **M @ 4426' ***** | kip-ft | 150,800 | 121,530 |
| **$M_{base}$ @ 4422' ****** | kip-ft | 165,100 | 128,340 |
| **$\delta_v$ @ 4422'** | inches | 2.02 | 1.66 |
| **Max $\delta_{diag}$ inlet gate** | inches | -0.024 | -0.05 |
| **Max $\delta_{vert}$ inlet gate** | inches | -0.014 | -0.015 |
| **$\varepsilon_{conc}$ @ 4422'** | in/in | -0.0057 | -0.0075 |

* Target displacement from SAP2000 time history analysis
** Displacement from ABAQUS nonlinear force pushover analysis
*** Element Section Force Moment
**** Moment based on $V_b$ x 31', (1/3 tower height)

**Table 4-1**
**Summary Of Tower Section Force And Displacement Capacities**

The inlet gate openings can sustain deformations of approximately 0.5 inches without any significant interference in the operation of the inlet gates. This is based on PG&E drawings that indicate a gap between the inlet gate and the guide track around the opening. The nonlinear pushover analyses show that the largest distortion demand is –0.04" in the diagonal across inlet gate 9-1 during the weak axis pushover at a target displacement demand of 4.9" as computed with the SAP2000 time history analysis. The uniaxial nonlinear pushover analyses indicate concrete softening will occur at the ends of the buttresses at the target displacement demand levels of 5.1 inches in the strong axis direction and 4.9 inches in the weak axis directions. This high compression zone is localized at the ends of the buttress and does not significantly affect the structural integrity of the tower or the functionality of the inlet gates. The deformation response is primarily rigid body rocking of the tower with localized compressive softening at the ends of the buttresses.

# 5 REFERENCES


1. "Lake Almanor Intake Tower Plumas County – California: Seismic Analysis with Base Rocking", Button Engineering, July 2007.
2. ABAQUS, Version 5.8, Hibbit, Karlson & Sorenson, Pawtucket, RI, 1998.
3. "ANACAP-U, Nonlinear Concrete and Steel Material Models", ANATECH Corp., 2005.
4. Correspondence with Mohammad Aslam, PG&E.