SWEETWATER AUTHORITY
505 GARRETT AVENUE
CHULA VISTA, CALIFORNIA 91910
(619) 420-1413
FAX (619) 425-7469
www.sweetwater.org

December 12, 2024

Subject: REQUEST FOR PROPOSALS TO UPDATE THE SEISMIC EVALUATION OF SWEETWATER DAM OUTLET TOWER AND CONDUIT STUDY

To Whom It May Concern:

Sweetwater Authority (Authority) is seeking a professional engineering services Consultant to update the Seismic Evaluation of Sweetwater Dam Outlet Tower and Conduit study, attached as Exhibit A. The Authority invites respondents to provide a proposal, including proposed project approach and costs, project team qualifications, and experience with relevant past projects in response to this Request for Proposals (RFP).

The Authority encourages participation by local, small, and/or disadvantaged businesses. Persons or entities submitting a proposal in response to this RFP are referred to herein as “Respondent”, whereas the successful Respondent to which the Authority would award a contract is referred to herein as “Consultant”.

A. BACKGROUND INFORMATION

The Authority

The Authority was formed in 1977 as a Joint Powers Agency between the City of National City and South Bay Water. The Authority is a publicly-owned water agency that serves potable water to a population of approximately 200,000 in the City of National City, the western portion of the City of Chula Vista, and the unincorporated areas of Bonita and Lincoln Acres, in San Diego County, CA.

The Authority’s service area covers approximately 36 square miles. The Authority owns, operates, and maintains a water distribution system with approximately 395 miles of transmission and distribution mains and 25 reservoirs, including 19 metallic water storage tanks. The Authority has several sources of water supply including surface water, fresh and brackish groundwater, and raw and treated imported supplies purchased from the San Diego County Water Authority (SDCWA).

The Authority’s mission is *“to provide its current and future customers with a safe and reliable water supply through the use of the best available technology, sound management practices, public participation and a balanced approach to human and environmental needs”*.

<u>Sweetwater Dam and Outlet Tower</u>

The Sweetwater Dam is located on the Sweetwater River in the Southern part of San Diego County, about six miles northeast of Chula Vista. The dam was originally constructed between 1886 and 1888 as a masonry arch dam with a height of 90 feet. Significant modifications were made to the dam and appurtenances in 1911 and again after the 1916 flood, as follows:

1. The dam was structurally raised 20 feet in 1911, and converted to a curved gravity dam by placing mass cyclopean concrete against the downstream face of the dam.

2. The dam was overtopped in 1916 and experienced some damage at the abutments. No damage was reported to the composite masonry section of the dam or to the outlet tower. The dam was repaired and the parapet well raised, bringing the dam crest to the present maximum height of 127 feet.

The South dike was originally constructed in 1910 and was reconstructed to its current configuration in 1916 after the flood.

The original freestanding outlet tower was constructed in 1888, and was presumably constructed out of the same masonry as the dam. It is located inside the reservoir, about 40 feet from the base of the Sweetwater Dam, and is adjacent to the lower portion of the right abutment slope. The outlet tower was raised in 1911 by 20 feet when the main dam was raised. A 51-foot one-span steel footbridge provides access to the tower from the dam crest. The bridge is attached to the tower by four 5/8-inch x 12-inch carriage bolts and on the dam side, its lower and upper members are supported on two bearing pads indented into the spillway crest.

The present tower is about 100 feet high, from its foundation base to the top of its circular operating platform. The shaft cross-section is hexagonal, with a maximum outside width of 13.4 feet, a maximum inside width of 5.2 feet, and a wall thickness of about 3.55 feet, as scaled from the drawings. The upper platform has a radius of about 21 feet, and a thickness of about 8 inches.

B. PROJECT OBJECTIVE

The objective of the project is for the consultant to review the 2003 report form GEI Consultants, Inc., titled “Seismic Evaluation of Sweetwater Dam Outlet Tower and Conduit”. After reviewing the report, the Consultant will be tasked with detailing a comprehensive update to the original 2003 report, and completing a conceptual level design and budgetary cost for strengthening the tower to withstand an earthquake with a return period of approximately of 144 years.

The Consultant shall use the US Army Corps of Engineers Engineer Manual titled "Earthquake Design and Evaluation of Concrete Hydraulic Structures" for the basis of the update.

The evaluation criteria shall be developed as deterministic or probabilistic response spectra. The deterministic response spectra shall represent the mean (50^{th} percentile) levels of ground motion that could be induced at the site by a Maximum Credible Earthquake (MCE) centered along the La Nacion Fault or other upper-bound magnitude events centered along more distant faults, such as the Rose Canyon, Agua Blanca-Coronado, San Miguel-Vallecitos, San Diego Trough, and Elsinore faults. The La Nacion and Rose Canyon faults have low rates of slip. The probabilistic criteria are representative of ground motion levels with 10 or 50 percent probabilities of occurrence during a 50-year period, corresponding to return periods of 144 and 72 years.

Once the update is conducted, the Consultant shall complete a conceptual level design and budgetary cost that would strengthen the tower to a level that it would be capable of safely withstanding ground motion with a horizontal peak ground acceleration equal to a seismic event with a return period of 144 years.

C. SCOPE OF WORK

The Consultant's scope of work shall be broadly based on the following tasks. Respondents are encouraged to add tasks as needed based on their understanding of the Project and proposed approach to performing the work.

TASK 1: KICK-OFF MEETING

Consultant shall schedule an in-person kick-off meeting at the Authority's office at 505 Garrett Avenue, Chula Vista, CA. The consultant will produce an agenda with all the items to be discussed and follow-up with minutes of the meeting.

The meeting should include, but not limited to the following items:

- Review the scope of work
- Review the budget
- Determine the team member's roles and responsibility in the application process
- Determine the schedule so the application will be submitted on time
- Discuss data and documents needed by the consultant from the Authority
- Determine any potential issues that may delay the application

TASK 2: DOCUMENTS TO BE PROVIDED BY THE AUTHORITY

The Authority will provide the following documents:

- Seismic Evaluation of Sweetwater Main Dam Outlet Tower and Conduit
- Other information and data as requested from the Consultant

Consultant shall create a list of additional documents needed from the Authority.

TASK 3: SCHEDULE DEVELOPMENT

Consultant shall create a schedule that will result in the update being submitted to the Authority on time and on budget. Consultant will be responsible to monitor the schedule to make sure that the progress of the project is on schedule.

The schedule should include, but not limited to the following:

- Milestones of importance
- Deadline submissions to the Authority
- Time dedicated to review by the Authority
- Time for a presentation to the Authority Board or committee.

TASK 4: COMPLETION OF THE UPDATE TO THE AUTHORITY

Consultant, in conjunction with the Authority as detailed above, shall submit the update to the 2003 Seismic Evaluation of Sweetwater Main Dam Outlet Tower and Conduit. Consultant will be responsible for developing the narrative, exhibits, budget, schedules, workplans and other necessary components for the report. Consultant shall have an internal quality assurance/quality control process, and conduct interactive internal reviews of the report before issuing a draft and final package to the Authority.

TASK 5: PROJECT MANAGEMENT

Consultant shall assume the following meetings:

- Project kick-off
- Progress check meetings at key milestones and more frequently as needed during the project process
- One presentation to the Governing Board and/or Engineering and Operations Committee

Respondents shall provide with their proposal a proposed schedule starting on as assumed notice to proceed date.

D. PROPOSAL REQUIREMENTS

Proposals submitted by Respondents shall be concise, well organized, and demonstrate the Respondent's experience applicable to the requirements of this RFP. A proposal submitted in response to this RFP shall be in the following order and shall include:

1. *Introductory Letter:* Describe Respondent's basic understanding of the Project objective and the proposed approach. The letter should also contain a statement regarding the qualifications of the firm and any summary information that may be useful or informative to the Authority.

2. *Identification of Respondent:*
   a. Provide legal name and address of company.
   b. Provide legal form of company (partnership, corporation, joint venture, etc.) and state of incorporation.
   c. Identify any parent companies.
   d. Provide addresses of office(s) and number of employees.
   e. Addresses of office(s) containing key proposed Project personnel.
   f. Provide name, title, address, phone number(s), and email of a person to contact concerning the proposal.

3. *Financial Relationships Disclosure(s):*
   a. Identify all existing and past financial relationships between the Respondent's firm and current members of the Authority's Governing Board, staff, and entities for which said members are employed or have an interest, both past and present. If there are none, clearly state this.
   b. Identify all existing and past financial relationships between the Respondent's proposed subconsultants and current members of the Authority's Governing Board, staff, and entities for which said members are employed or have an interest, both past and present. If there are none, clearly state this.
   c. For a list of the Authority's Governing Board members, see the following link: http://www.sweetwater.org/35/Governing-Board.

4. *Approach for Completing the Work:* Based on review of this RFP and any publicly available data or resources pertaining to the outlet tower, describe the approach for completing the report. Include detailed tasks for completing the work, which may expand upon the above Scope of Work, deliverables to the Authority for each task identified in the proposal, and a timeframe for completing each task.

5. *Required Qualifications:* The following are the minimum required qualifications for Respondents. Interested parties should not submit a proposal if they do not meet these required qualifications:

   a. The Respondent's primary business or the primary business of a department within the Respondent's firm shall be engineering consulting services for large-scale dam evaluations, and shall have been in the business of providing such services for at least five (5) years.

   b. The Respondent shall provide a single project manager as the primary point of contact with the Authority. This project manager must have at least five (5) years total experience with current firm or other employers in projects related to large-scale dam evaluations, and shall be registered as a professional engineer in the state of California.

   c. Provide a list of past and ongoing qualifying projects for which the Respondent's services were or are similar to those described in this RFP. Limit the list to no more than ten projects the Respondent believes are most relevant to the RFP. For each project, include the following:

      - A brief description of the project, date initiated, date completed (if applicable).
      - Name of owner and owner's project manager with contact information (email and/or phone number).
      - Identify role of the key personnel proposed for the grant funding application.

   d. Present the experience of any proposed subconsultants in the same manner.

   e. Provide evidence of the experience and competence of the Respondent's team proposed to work on the Project, with specific emphasis on experience in working on large-scale dam evaluation.

6. *Respondent's Firm and Key Personnel:* Provide an organizational chart showing the relationship and titles of key personnel. Describe Respondent's firm, including identification and responsibilities of key personnel and subconsultants. For each of the key personnel, identify their main work location. Identify the project manager who will be responsible for the direct supervision and coordination of all work activities.

7. *Costs:* Provide costs for every task identified in the proposal, in Portable Document Format (PDF). Costs shall be provided in a separate document from the proposal submittal.

8. *Exceptions to the RFP and/or Professional Services Agreement:* The Respondent shall certify that it takes no exceptions to this RFP, including but not limited to, the Authority's Agreement for Services (Agreement), as attached in Exhibit B. If the

Respondent does take exception(s) to any portion of the RFP or Agreement, the specific portion of the RFP or Agreement to which exception(s) is taken shall be identified and proposed alternative language shall be provided and explained in the proposal.

9. *Proposal Authorization:* The proposal shall be signed by an individual authorized to bind the consultant and shall contain a statement to the effect that the submittal is in effect for ninety (90) days.

10. *Proposal Submittal:* Provide one (1) electronic copy of the proposal document and one (1) electronic copy of the proposed costs in separate PDF files. The proposal document file and separate cost proposal file shall be uploaded to PlanetBids at the link below.

    https://vendors.planetbids.com/portal/69501/bo/bo-detail/124636

Proposals in response to this RFP are due to PlanetBids no later than 4:00 p.m. on Thursday, January 16, 2025.

Proposals submitted after this deadline will not be accepted.

E. CONSULTANT SELECTION PROCESS

1. The Authority will evaluate all proposals based on the evaluation criteria presented in this section, as well as other information obtained through background information and references.

2. The Authority will convene a selection committee to review the submitted proposals. Using the established evaluation criteria and associated scores in this section, the selection committee will evaluate and rank the proposals.

3. The evaluation criteria that will be used by the Selection Committee are as follows:

Category	Maximum Points
Approach to complete the report	60
Completeness of proposal in addressing requested information	10
Relevant qualifications and experience of the Respondent’s personnel assigned	30

4. The selection committee may choose to interview the top-ranked Respondents. The selection committee may re-evaluate the interviewed Respondents and rank them considering both the proposal and interview. The Authority reserves the right to eliminate the interview step of the selection process.

5. The Authority will notify the top-ranked Respondent and will proceed with negotiations regarding cost or any exceptions the Respondent took to this RFP or the Standard Agreement for Services. Should the Authority and top-ranked Respondent not reach agreement, the Authority will proceed with negotiations with the next-ranked Respondent until agreement is reached. The Authority reserves the right to cancel the RFP process at any time.

6. A Services Agreement between the Authority and the selected Respondent would be executed upon approval and award by the Authority's Governing Board.

F. AGREEMENT EXECUTION AND RENEWALS

Following award, the selected Consultant will be required to provide insurance documentation before an agreement is executed. The Consultant will be expected to execute the Authority's standard agreement without modification. A copy of the Agreement is provided in Exhibit B. If the Consultant takes exception(s) to any portion of the agreement, the specific portion of the agreement to which exception(s) is taken shall have been identified and proposed alternative language shall have been provided and explained in the proposal.

All services shall be performed on a time and materials basis in accordance with the standard hourly rates as submitted by the Consultant and the terms of the agreement. Once the agreement is executed by both parties, the Consultant's work will be authorized via a Notice to Proceed (NTP) letter.

E. DISCLAIMER

This RFP does not commit the Authority to enter into an agreement for services, to pay any costs incurred in the preparation of a proposal, or to procure or contract for services or supplies. The Authority reserves the right to accept or reject any or all proposals received as a result of this RFP, to negotiate with any qualified source, or to cancel in part or in its entirety this RFP, if it is in the best interest of the Authority to do so. The Authority shall not be obligated to contract any or all of the requested services to the selected Consultant. Further, even upon execution of the Agreement, the selected Consultant will not be guaranteed any work under the Agreement until an NTP letter is issued by the Authority.

If you have any questions regarding this RFP or the described scope of work, please contact me at edelbosque@sweetwater.org, or 619-409-6750.

Sincerely,

SWEETWATER AUTHORITY

Erick Del Bosque

Erick Del Bosque, P.E.
Director of Engineering and Operations

enclosures: Exhibit A: Seismic Evaluation of Sweetwater Dam Outlet Tower and Conduit
Exhibit B: Standard Agreement for Services Template

EXHIBIT A

SEISMIC EVALUATION OF SWEETWATER DAM OUTLET TOWER AND CONDUIT REPORT 2003

Seismic Evaluation of Sweetwater Main Dam Outlet Tower and Conduit B.P. 01-20E

GEI Consultants, Inc.

2141 Palomar Airport Road, Suite 160
Carlsbad, CA 92009
(760) 929-9136

SUBMITTED TO

Sweetwater Authority
505 Garrett Avenue
Chula Vista, CA 91912-2328

IN ASSOCIATION WITH

Gilles Bureau
Consulting Engineer
140 Wildwood Avenue
Piedmont, CA 94610

Thomas O. Keller, P.E., G.E.
Project Manager

February 2003
022560

February 14, 2003
Project 022560

Mr. T. Kevin Kasner, P.E.
Sweetwater Authority
P.O. Box 2328
Chula Vista, California 91912-2328

Re: Sweetwater Main Dam – Outlet Tower and Conduit Evaluation
B.P. 01-20E

Dear Mr. Kasner:

Attached are three copies of our February 14, 2003 report on an evaluation of the response of Sweetwater Main Dam's outlet tower and conduit to various earthquake loading scenarios. Analyses presented in the report were performed by Gilles Bureau as a subconsultant to GEI Consultants, Inc. The key results of the evaluation were conveyed to Sweetwater Authority at a meeting on December 5, 2002.

Sincerely,

GEI CONSULTANTS, INC.

Thomas O. Keller, P.E., G.E.
Principal

c: Gilles Bureau

Table of Contents

List of Tables

List of Figures

Executive Summary

The Sweetwater Main Dam and Reservoir contains a 100-foot tall outlet tower in the reservoir that is used to control flow of reservoir water to the Robert A. Perdue Water Treatment Plant. The tower is very slender, and consists of stone and mortar with no steel reinforcement. The tower was constructed in 1888, and raised by 20 feet in 1911. Even though the tower is over 100 years old, it appears to be in good condition. However, the slenderness of the tower, combined with the fact that it contains no steel reinforcement, makes it vulnerable to cracking, and possibly toppling, during an earthquake.

GEI Consultants, Inc. was engaged to estimate the level of earthquake loading that could cause the tower to fail, and the probability of that earthquake to occur. A conclusion of the study was that an earthquake causing a peak ground acceleration at the site of about 0.11g (g is the acceleration due to gravity) could cause failure of the tower. The chance of this occurring is about 50 percent in the next 100 years. Hence, within the next century, the Sweetwater Dam outlet tower has a 50 percent chance of remaining stable during an earthquake event. An example of an earthquake that could produce a ground acceleration of 0.11g at the dam site is a Magnitude 5.5 earthquake on the Rose Canyon fault, located about eight miles west of the site.

A stone and mortar conduit is located between the base of the outlet tower and the base of the dam. This conduit is used to convey water from the tower to a pipeline that passes through the dam, which in turn conveys water to a pipeline that leads to the water treatment plant. In general, the conduit between the outlet tower and dam is capable of surviving a much larger earthquake than the tower.

Failure of the outlet tower would not cause failure of Sweetwater Dam itself, which is a massive concrete structure. Therefore, the potential for tower failure is not a dam safety issue. However, tower failure could cause an interruption in water deliveries from Sweetwater Reservoir to the customers of Sweetwater Authority. The findings of this study will be used by the Authority to decide whether the calculated risk of failure of the outlet tower is acceptable for such an essential, but not safety-related, facility and to perform cost-benefit analysis for any major upgrades that might be considered.

Technical Summary

This report presents the results of a seismic evaluation of the Sweetwater Main Dam outlet tower and conduit, owned and operated by Sweetwater Authority (Authority). The primary purpose of this investigation was to evaluate the structural performance of the tower under seismic loads and estimate the characteristics of the most severe ground motion that the tower could withstand without collapse or major failure.

Seismic evaluation criteria were developed as deterministic or probabilistic response spectra. The deterministic response spectra represent mean (50^{th} percentile) levels of ground motion that could be induced at the site by a Maximum Credible Earthquake (MCE) centered along the La Nacion Fault or other upper-bound magnitude events centered along more distant faults, such as the Rose Canyon, Agua Blanca-Coronado, San Miguel-Vallecitos, San Diego Trough, and Elsinore faults. The La Nacion and Rose Canyon faults have low rates of slip. The probabilistic criteria are representative of ground motion levels with 10 or 50 percent probabilities of occurrence during a 50-year period, corresponding to return periods of 144 and 72 years, respectively.

We performed a visual inspection of the tower on August 29, 2002 and found it to be in good condition. Schmidt hammer testing was performed during the inspection to assess the quality of the concrete portion of the tower. The stone masonry was tested near the dam left abutment, which was assumed representative of the tower masonry, most of which was constructed at the same time as the dam. No particular structural deficiencies were observed in the visible portions of the tower.

We performed parametric finite element response analyses of the tower for the specified earthquake loading, and for a range of strength and elastic properties for the stone masonry. The mathematical model was composed of three-dimensional structural beam elements (stick model).

For some of the specified ground motions, our response analyses indicated that the moment-resisting capacity of the Sweetwater outlet tower would be largely exceeded in its lower half. The structure did not meet performance evaluation criteria for the MCE or Rose Canyon events, or for the seismic criteria with 10 percent probability of occurrence in 50 years. Hence, significant earthquake-induced cracking of the masonry and possible collapse of the tower could occur under ground motion similar or more severe than these earthquake scenarios. The tower was also shown to be potentially unstable for global overturning for the La Nacion and Rose Canyon maximum earthquake events.

Based on analyses reported herein, we believe that the tower is not likely to be significantly damaged by ground motions induced by recognized active faults in the greater project vicinity other than the La Nacion and Rose Canyon faults. After estimating earthquake loads and capacities by eliminating some of the necessary conservatism applied in the numerical analysis (this was done by using root-mean-square loads and unfactored capacities), the tower appears capable of safely withstanding ground motion with a horizontal peak ground acceleration (PGA) up to about 0.11g. For the local tectonic environment, this corresponds to a seismic event with a return period of about 144 years, or a probability of occurrence of 29 percent in 50 years or 50 percent in 100 years. Hence, within the next century, the Sweetwater outlet tower has a 50 percent chance of remaining stable during a seismic event.

The outlet conduit was evaluated for global stability and for overstressing potentially caused by seismic waves traveling laterally or longitudinally with respect to its alignment. Masonry cracking is probable under the MCE. The MCE was the only one of four seismic scenarios considered where conduit instability was computed to occur by toppling, ignoring any passive resistance that could be provided by the loose reservoir sediments. Overall, the seismic stability of the outlet conduit is of little concern, compared with that of the tower.

The above findings could be used by the Authority to decide whether such risk is acceptable for this essential, but not safety-related facility, and to perform cost-benefit analysis for structural upgrades that might be considered.

This technical summary presents selected elements of our findings, and interpretations. It does not present crucial details needed for application of our findings and interpretations. These details are provided in the main body of this report.

1. Introduction

1.1 General

This report presents the results of a seismic stability evaluation of Sweetwater Main Dam's outlet tower, as well as a conduit that connects the tower to the dam. The potential behavior of the tower was evaluated for various earthquake scenarios, and considered a range of potential tower properties. Sweetwater Main Dam (referred to as Sweetwater Dam in this report) and Reservoir are owned and operated by the Sweetwater Authority (Authority).

1.2 Purpose

The Sweetwater outlet tower is used to control releases of raw water stored in Sweetwater Reservoir. The raw water is directed through an outlet conduit to the nearby Robert A. Perdue Water Treatment Plant prior to delivery to Authority customers. Therefore, the tower is essential to the management of the Authority's water distribution function, and its failure would represent a major inconvenience to the Authority.

The primary objective of this evaluation was to assess the behavior of the tower and conduit under seismic loading, and to estimate the levels of ground motion they could withstand without collapse or major structural failure. The seismic behavior of the tower is partly dependent on the properties of materials used to construct the tower. Seismic analysis were performed for a range of these material properties to judge their influence on tower behavior. Results of these parametric analyses can be used to judge the need for more detailed investigations of material properties. Knowing the level of risk associated with possible major seismic damage, the Authority will be able to assess if such risk is acceptable for these structures. Such knowledge can be used in the decision-making process before considering any structural upgrades.

1.3 Limitations

The data, information, interpretations and recommendations contained in this report are presented solely as a basis for a preliminary assessment of the seismic performance of the Sweetwater outlet tower and conduit. The conclusions and interpretations contained herein were primarily developed by Gilles Bureau, P.E., G.E. as a subconsultant to GEI Consultants, Inc. (GEI). They are in accordance with generally accepted standards in the geotechnical and structural engineering professions, but rely on old drawings and background data developed by others.

This report was prepared based on a review of full-size or reduced original construction drawings, design data, and previous construction or inspection reports made available to the project team. We performed a brief field inspection and structural audit. Data collected and dimensional checks performed during that inspection, as well as published information found to be applicable, were used to supplement the data retrieved from Authority files.

Unanticipated geologic or foundation conditions, or concealed structural features of the outlet tower, if different from those shown on the drawings or described in previous reports, could affect some of our conclusions. Our field inspection was limited to the visible portion of the outside perimeter of the tower on the day of the inspection. The true existing conditions of concealed elements of the tower may differ from those assumed in this evaluation. Our evaluation relied upon stone masonry elastic parameters and mortar strength properties estimated from the original design data and limited non-destructive in-situ testing performed during our field inspection. Such testing was less complete than would be obtained from a core sampling and testing program. However, more detailed field and laboratory studies were concluded not to be required for the purpose of this initial investigation.

Our conclusions and recommendations only relate to the Sweetwater Dam outlet tower and conduit. This report has been prepared for the sole and exclusive use of the Authority and for possible submission to the State Division of Safety of Dams (DSOD). It may contain information insufficient for the purpose of other parties or other uses.

2. Project Description

2.1 Construction History

Sweetwater Dam is a 127-foot high curved gravity dam located near Chula Vista, California composed of an upstream cyclopean stone masonry thick arch and a downstream concrete gravity section. The lower 50 feet of the masonry portion was designed by F.E. Brown and constructed in 1886. The dam was intended to be a thin-arch. James Schuyler, a renowned dam engineer, revised the design in 1887, and thickened and extended the masonry arch to a height of 60 feet in 1887, and to 90 feet in 1888.

All elevations in this report are in units of feet and are referenced to Sweetwater Authority datum. Sweetwater Authority datum is about four feet lower than National Geodetic Vertical Datum (also referred to as Mean Sea Level Datum). Elevations are commonly referred to by the abbreviation "El."

The original freestanding outlet tower was constructed in 1888 to about El 220 (top platform elevation), and was presumably built of the same masonry as the dam. It is located inside the reservoir, about 40 feet from the base of Sweetwater Dam, and is adjacent to the lower portion of the right abutment slope. A general plan of the dam and tower is shown in Figure 1, and a cross-section through these facilities is shown in Figure 2.

In an 1897 Annual Report of the United States Geological Survey (USGS) and a technical article titled "*Reservoirs for Irrigation,*" Schuyler described the care taken during the original construction of the Sweetwater facilities. Original construction consisted of the best class uncoursed, rough rubble masonry laid in rich mortar of Portland cement and sand. The masonry was carefully set in-place by skilled stone masons, and was mixed one part cement to two parts clean sand (1:2) for the portion of masonry within 4 feet of the reservoir. The stones came from a quarry 800 feet downstream of the dam, had no well-defined joints, and were reported to have a specific gravity between 175 pounds per cubic foot (pcf) and 200 pcf. Based on such records, the quality of the masonry would be expected to be high.

The original tower was equipped with eight inlet elbows with a saucer valve and basket screen, and three outlet pipes near the bottom. Two of the valves, valves 1 and 2, are actually located on the outlet conduit, on either side of the tower, with inlets at El 145.3 and El 155, respectively. The other valves are located along the tower shaft as follows: valve 3 (El 165), valve 4 (El 175), valve 5 (El 185), valve 6 (El 195), valve 7 (El 205) and valve 8 (El 215).

Following floods and overtopping of the dam in 1895 and 1909, Sweetwater Dam was structurally raised 20 feet in 1911, and converted to a curved gravity dam by placing mass cyclopean concrete against the downstream face. The outlet tower was also raised at that time and the top of the shaft extended by about 20 feet (top of platform at El 240) with a masonry sleeve. An October 3, 1911 construction report described the concrete mixture in the stone masonry as follows: One part cement, three parts sand and five parts aggregate. Five to ten percent of the cement in the mixture were replaced by hydrated lime. The sand came from a local quarry, half-a-mile south of the dam. A new saucer valve, valve 9 (El 220), was added. According to the construction drawings, the outside of the new portion of the tower was covered with one coat of 1:2 cement mortar with 10 percent of hydrated lime. It is possible that the entire tower surface was covered with cement mortar at that time. An October 3, 1911 Construction Report by John Covert, Resident Engineer, indicated that 77.2 cubic yards of cement were used for the tower. The old tower platform and roof were raised to the new elevation, as well as the access footbridge.

Sweetwater Dam was again overtopped in 1916 and experienced some damage at the abutments. No damage was reported to the composite masonry section of the dam or to the outlet tower. The dam was repaired and the parapet wall raised, bringing the dam crest to near present maximum height (127 feet). In 1939-1940, the spillway crest wall was replaced with a rounded spillway overflow sill, and the access bridge to the outlet tower was relocated to its lower present elevation (bridge deck at El 237). The old bridge was replaced with a 50.8 foot-long one-span steel footbridge providing access to the tower from the dam crest. The bridge is attached to the tower by four 5/8-inch x 12-inch carriage bolts and, on the dam side, its lower and upper members are supported on two bearing pads indented into the spillway crest.

The present tower is about 100 feet high, from its foundation base to the top of its circular operating platform. The shaft cross-section is hexagonal, with a maximum outside width of 13.4 feet, a maximum inside width of 5.2 feet, and a wall thickness of about 3.55 feet, as scaled from the drawings. The upper platform has a radius of about 21 feet, and a thickness of about 8 inches. Photographs of the tower are contained in Appendix A.

2.2 Geology

Available information regarding the geology of the site was reviewed, as developed in earlier foundation investigations and safety review studies (Dames & Moore, 1994; URS, 2001).

Foundation conditions at Sweetwater Dam and outlet tower consist of competent metavolcanic bedrock of the Jurassic Santiago Peak volcanics. That formation consists of a very hard metamorphosed dacite, with either aphanitic or porphyric texture. Bedrock is typically sound, with only a few feet of surface deterioration. Woodward-Clyde Consultants (1975) reported unconfined compressive strength data for foundation bedrock and masonry stones ranging from 12,000 to 18,000 pounds per square inch (psi), and a unit weight of 168 pcf, hence lower than the 175 to 200 pcf reported in 1897. The spacing between joints in the foundation rock was estimated at three to six feet or more, based on construction photographs and records (USCOLD, 1988). However, erosion of the rock resulting from spillway overflow and observations of exposed rock downstream of the dam indicate that a more closely spaced micro-fracture system appears to exist (URS, 2001). Because of the overall excellent quality of the local bedrock, the bond between the tower base and bedrock, although not described on drill logs, is likely to be good (USCOLD, 1988).

2.3 Field Inspection

2.3.1 General

The project team inspected the Sweetwater outlet tower on August 29, 2002. Kevin Kasner and James Smith, from the Authority, were present. During the inspection, Gilles Bureau performed non-destructive Schmidt hammer testing of the concrete mortar facing along the outside facing of the outlet tower wall, slightly above the reservoir surface, and of the concrete at the top of the tower platform. No masonry was visible at the tower, but Schmidt hammer rebound measurements were also taken near the dam left abutment, where the original dam masonry is exposed. The approximate location of the masonry tested on August 29, 2002 is shown in Photograph 5 in Appendix A. Such measurements may be indicative of the strength of the tower masonry stones and mortar, which are believed to be of the same composition as the dam masonry.

2.3.2 Structural Inspection and Existing Data Review

The August 29, 2002 inspection was limited to observations of the visible portion of the outside faces of the tower walls. The water level was at El 198.9 on the day of the visit. Observation of the concrete mortar facing suggests that the upper part of the tower is in good condition. Neither significant deterioration nor efflorescence was observed. No significant cracks were visible. Thin horizontal cracks were observed where the bolts that anchor the footbridge deck to the tower penetrate the tower wall. These cracks are not structurally significant.

At the top of the upper platform of the tower, there are five two-foot wide, 14-inch tall square concrete pedestals with a one-foot wide central square opening. These pedestals support the winches used to open and close the five upper saucer valves. Authority personnel indicated that reservoir silt has reached a level between valves 3 and 4, or approximately El 170.

The widths of the contact areas of the bridge structure with the dam crest were measured at 14 inches at the lower support pad, and 12 inches at the upper pad. Bridge side horizontal clearance with the vertical concrete surfaces at the support pads is about six inches. Hence, for out-of-phase seismic movements between the top of the tower and the dam spillway crest, the maximum relative displacements that the bridge could experience toward the dam without being compressed between the two structures is about six inches. About 12 inches of relative movements of the bridge could be accommodated by the support pads, if the tower and the dam were moving away from each other, assuming that the four anchor bolts that tie the bridge to the tower side have sufficient capacity. Rupture or pullout of the bolts as a result of excessive oscillations of the tower structure would cause the bridge to fall into the reservoir.

During the field inspection, Gilles Bureau performed Schmidt hammer testing of the concrete mortar facing along the tower outside surface, about four feet above the reservoir level, and of the concrete at the top and side of the operating platform. Schmidt hammer measurements can be correlated to compressive strength. The stone masonry was not visible along the tower shaft. However, stone masonry of the dam structure is exposed at its left abutment. Schmidt hammer testing of the dam masonry mortar and stones was also performed. The dam stones and mortar may be similar to those at the tower, having been built at similar times and presumably with similar materials and techniques.

The results of these tests, which were taken at random locations, are shown in Table 1. Schmidt hammer readings are proportional to the height of instantaneous rebound, after impact on the material tested, of a steel ram and plunger released through the sudden expansion of a loaded spring. Hence, these measurements should be indicative of a “dynamic” strength (rapid loading condition). A Type-N Schmidt hammer was used. The tests were performed on clean flat surfaces, prepared with a grinding stone.

Compressive strengths for the tower concrete facing and mortar were obtained from a correlation between measured rebound and unconfined compressive strength provided by the instrument manufacturer. The estimated compressive strength for the masonry stones was obtained after converting measured rebound values to equivalent rebound values for a Type-L Schmidt hammer. The L-equivalent values were then converted into a compressive strength, based on an assumed unit weight of 168 pcf for the masonry stones and a correlation developed by Deere and Miller (1966) for rock testing with a Schmidt hammer.

Interpretation of the Schmidt hammer testing indicated an average dynamic compressive strength of 3,900 psi for the dam masonry mortar. Twelve tests were performed on that material, with a standard deviation of about 980 psi. Although not very precise (the instrument error, or dispersion, can be significant), these measurements indicate good quality masonry mortar. Eleven tests were performed on the top concrete platform and along the concrete facing of the outlet tower. These tests indicated a higher dynamic compressive strength, averaging about 7,200 psi, with a standard deviation of about 710 psi. Lastly, ten tests were performed on selected dam masonry stones. These tests indicated high rebound values, typically between 60 and 70, which correspond to dynamic compressive strengths ranging from 31,000 psi to 59,000 psi, with an average of about 47,400 psi and a standard deviation of about 10,600 psi.

The Schmidt hammer tests were performed on selected small, clean, hand-ground, uncracked areas of the concrete, mortar or stone surfaces. For two other towers, Bureau and Scawthorn (1986) indicated reasonable consistency between compressive strengths derived from Schmidt hammer tests and laboratory tests on cores of concrete and brick masonry mortar. The same may not be true in the case of the masonry stones, where Schmidt hammer testing could yield higher estimated strengths than large core or block testing. Micro-fissures, joints, foliation, and any sheared or weathered areas would control failure of masonry blocks or stones. Hence, the field tests performed on masonry stones only confirm the hard nature of the local rock and cannot be used to reliably estimate the strength of large specimens. A limited amount of laboratory testing to obtain information on compressive strength of bedrock cores and masonry stones from the dam site was performed in 1975 (WCC). These tests indicated compressive strengths ranging from about 12,000 to 18,000 psi. These compressive strengths are significantly lower than those based on Schmidt hammer testing performed for this evaluation. The differences are due in part to the test method ("static" testing of laboratory samples versus "dynamic" testing in the field using a Schmidt hammer), but may also be due to potential discontinuities in the larger samples used for laboratory testing.

According to construction records (1939), the unconfined compressive strength of the cement mortar facing and concrete of the top platform was specified as 3,000 psi at 28 days, with maximum 2-inch aggregate. The static compressive strength presently estimated from the Schmidt hammer tests is about 6,000 psi. This indicates that such concrete has gained strength with age, and/or that the original specifications were met or exceeded.

In summary, based on the field observation of the visible parts of the tower and limited non-destructive testing, the Sweetwater outlet tower appears to be in good condition.

3. Seismic Analysis Criteria

3.1 General

Failure of the Sweetwater outlet tower would be extremely unlikely to endanger the safety of the dam and its capacity to impound the reservoir. Hence, seismic requirements less demanding than previously used for Sweetwater Dam have been used for evaluation of the tower.

For tower evaluation purposes, it was assumed that the foundation bedrock at the tower site is sufficiently competent to be considered as rigid, compared with the more flexible tower structure. Therefore, seismic criteria applicable to an outcropping bedrock condition were developed. As the tower is a slender structure of relatively light mass, tower-foundation interaction effects were neglected. Ignoring tower-foundation interaction effects is reasonable.

The use of free-field seismic input criteria at the base of the tower is conservative, as no radiation damping is accounted for in such numerical analyses. Overall, we believe that local subsurface conditions should have little or no significance for the dynamic structural evaluation of the tower, and that fixed-base response analysis is appropriate.

Previous geologic and seismicity studies have been performed for Sweetwater Dam and other Authority facilities (Dames & Moore, 1994, 1995; URS, 2001). Most of the following section, which describes the tectonic environment of the site, is based on information contained in these previous studies, and updated as needed.

3.1.1 Tectonic Environment

The greater site area lies within a broad zone of faulting related to interaction between the Pacific and North American tectonic plates. Sweetwater Dam is located within that tectonically active region. Chula Vista and its vicinity have only experienced moderate, rare historic seismicity, compared with other areas of near-coastal California. Major regional and local faults include, from east to west: the San Andreas, San Jacinto, Elsinore, La Nacion, Rose Canyon, Coronado Bank, San Diego Trough and San Clemente fault zones.

Ongoing tectonic activity within the area is reflected by Holocene age (11,000 years old or younger) displacements on major northwest-trending faults and youthful geomorphic features of tectonic origin. Historically, The San Jacinto Fault zone has proved to be the most active system in the region. However, because of its distance from Sweetwater Dam (about 94 km), ground shaking resulting from earthquakes on this fault is not considered to be a significant threat. Earthquakes generated along the San Clemente Fault (about 87 km from the dam) and The San Andreas Fault (about 140 km from the dam) are also considered too distant to have a major impact on this site. The La Nacion and Rose Canyon fault zones are the two closest, most prominent, local fault systems with evidence of Quaternary activity. The six fault zones of greatest potential concern to the dam are described below, in order of increasing distance from the site.

La Nacion Fault Zone

Because of the short distance (4 km) from the La Nacion Fault Zone to the site, this fault system controls the Maximum Credible Earthquake (MCE) for the Sweetwater Dam site. The fault zone is a north-northwesterly trending series of discontinuous, moderate to high angle dip-slip faults, traceable from the U.S.-Mexico border northward through the eastern San Diego Metropolitan area, up to about the latitude of Mission Valley. Because the La Nacion Fault Zone is poorly defined, estimates of its length range from approximately 12 to 17 miles (19 to 28 km). Offset along the La Nacion Fault Zone is primarily dip-slip movement. The fault has displaced Pleistocene deposits (Lindavista Formation) by about 365 feet (Artim and Pinckney, 1973), but evidence for Holocene displacements is lacking. Geologically recent tectonic displacements, reported by Artim and Pinckney (1973) were subsequently concluded not to displace Holocene sediments (Elliot and Hart, 1977). However, the La Nacion Fault Zone must be considered to be potentially active with very long recurrence intervals (Artim and Elder, 1979). MCE magnitude estimates range from 6.5 to 6.7. Previous studies of Sweetwater Dam (Woodward-Clyde Consultants, 1975; and URS, 2001) assigned an upper bound magnitude of 6.7 to the La Nacion Fault Zone. This value represents a conservative estimate and was used in this evaluation.

Rose Canyon Fault

The Rose Canyon Fault Zone is located between 7 to 15 km west of the La Nacion Fault Zone and is composed of numerous subparallel, en échelon and branching sub-faults that generally trend north to northwest. This Fault zone extends south, paralleling the coast offshore from the latitude of Carlsbad, crosses inland along the northeast flanks of Mount Soledad, and continues south along the eastern margins of Mission Bay. Between Mission Bay and downtown San Diego the zone appears to widen and diverge as it continues south across San Diego Bay and Coronado before returning offshore. Offshore traces of the Fault zone extend to the latitude of the International border for an estimated total system length of about approximately 72 km. The closest approach of this fault to the site is about 13 km. The Rose Canyon Fault has been characterized by many authors as having a predominantly right-lateral strike-slip type of movement, but significant dip-slip has occurred on at least two segments: toward the southern end of the fault zone in the shallow continental shelf area (Kennedy and others, 1979), and north of Mount Soledad (Kennedy and others, 1975). Historically, the Rose Canyon Fault has typically been micro-seismically active. However, in 1985 and 1986 a series of earthquakes in the vicinity of San Diego Bay with magnitudes up to 4.7 were attributed to activity along the Rose Canyon Fault Zone. Trenching studies by Anderson and others (1989) within Rose Canyon concluded that Holocene alluvium and modern topsoil ("A" horizon) have been offset by the fault. Rockwell and others (1989) suggested the potential for earthquakes with magnitudes up to 7.0 for the Rose Canyon Fault Zone.

Agua Blanca-Coronado Bank Fault

The Agua Blanca-Coronado Bank fault zone consists of a northwest-trending series of en échelon faults that extend from onshore Baja (Agua Blanca portion) into the offshore Mexico and California inner borderland (Coronado Bank portion). The closest approach of this system to the site is along its offshore segment, about 28 km toward the west. The Agua Blanca-Coronado Bank Fault Zone is characterized as having both right- and left-stepping segments (Kennedy et al, 1980). Offshore, it is shown to cut Quaternary-age sediments in reflection profile records. Its predominant type of displacement is right-lateral (Clark, et al., 1984). An upper bound magnitude of 7.2 for the Agua Blanca-Coronado Bank Fault Zone was used for this evaluation.

San Miguel-Vallecitos Fault

The San Miguel-Vallecitos Fault, located in northern Baja California, is approximately 154 km in total length. The fault is a right-stepping system consisting of three segments (northern, central and southern). The northern segment is approximately 43 km southeast of the site. The San Miguel-Vallecitos Fault Zone has been the most active in Northern Baja California. Six earthquakes of about magnitude 6.8 occurred in 1954 and 1956 along its southern segment. Local studies (Anderson, et al., 1989) estimate the magnitude at 7.0 for this fault system.

San Diego Trough Fault

The San Diego Trough Fault is located offshore approximately 50 km west-southwest of the site and displays concentrated, low-level, seismic activity. The San Diego Trough Fault appears to strain in response to movement along a minor southern strand of the Agua Blanca Fault (Legg, 1985), which slips no more than 1 millimeter/year (Rockwell et al, 1987). Seismic reflection profiles suggest that the San Diego Trough Fault is continuous for approximately 15 km. However, it is presumed to be associated with the Bahia Soledad Fault, onshore Baja California (Legg, 1985). If so, this would yield a total fault length of over 155 miles (250 km). A combined rupture along this fault zone could generate earthquakes of magnitude 7.2, or perhaps greater.

Elsinore-Laguna Salada Fault

The Elsinore Fault is about 60 km away from the site. The Elsinore Fault Zone is considered to be part of the northwest-southeast trending Whittier-Elsinore Fault Zone, and extends nearly continuously for approximately 185 to 255 km from the vicinity of Corona, in the Los Angeles Basin, to southeast of the International Border into Mexico, where it continues southward as a series of subparallel right-stepping segments designated as the Laguna Salada Fault. The Elsinore Fault has been characterized as having both dip-slip (Clark, 1982) and right-lateral displacements (Yerkes, 1972; Lamar et al, 1975). Holocene-age displacements have been revealed in exploratory trenches across the fault, south of Lake Elsinore (Lamar and Swanson, 1981). The Elsinore Fault has been recognized to be composed of five individual active segments, each with a separate history of movement and characteristic type of deformation. The Elsinore-Laguna Salada fault system has experienced several relatively recent earthquakes with magnitudes between 5.0 and 5.9, and has had historic earthquakes of larger magnitude (1812, M 6.75; 1842, M 7.0 to 7.5; and 1910, M 6.0). Numerous paleoseismic events of magnitudes between 6.5 and 7.1 have also been identified (Rockwell, 1989). Magnitude estimates range from 7.0 to 7.3, depending on what length is assumed for the active segments. Simultaneous rupture of two or more segments could yield an earthquake with a magnitude of 7.5. Such large magnitude is now conservatively considered the most representative of the fault, and was used for this evaluation.

3.1.2 Deterministic Seismic Criteria

For Sweetwater Dam, the La Nacion Fault is the controlling geologic feature for the MCE. According to USCOLD (1999), "*the MCE is the largest reasonably conceivable earthquake that appears possible along a recognized fault or within a geographically defined tectonic province, under the presently known or presumed tectonic framework. The MCE is generally defined as an upper bound of expected magnitude, or in less frequent cases, as an upper bound of Modified Mercalli Intensity. Little regard is given to its probability of occurrence, which may vary from less than a hundred to over ten thousand years, depending on the geologic environment considered.*" California dams such as Sweetwater Dam must be evaluated for the MCE, which was also considered for the analysis of the outlet tower reported herein. For many sites near coastal California and dams whose failure could potentially cause extensive loss of life and property, it has been customary and a State Division of Safety of Dams (DSOD) requirement, to define the MCE by response spectra that comply with mean-plus-one-standard-deviation (84th percentile) estimates in the range of periods of interest. On a case-by-case basis, DSOD has sometimes accepted less demanding ground motion criteria for low risk dams and moderately active tectonic environments.

The La Nacion Fault has a very low rate of activity, which reduces its significance to the outlet tower. However, events other than the MCE, occurring along more distant faults and/or with a higher probability of occurrence, are of direct interest to the evaluation of the tower. We have, therefore, estimated the maximum ground motions that could be generated by the MCE and by maximum earthquakes centered along five other well-known faults affecting the project area.

Considerable insight has been gained in recent years regarding the characteristics of ground motion and, especially, its attenuation as a function of distance from fault rupture. For this project, we developed response spectra for the MCE and five other deterministic earthquake scenarios, using several well-accepted sets of attenuation equations for peak ground acceleration (PGA) and spectral accelerations. Significant considerations for these response spectra are the associated margin of error and their probability of being exceeded, as discussed below. Such considerations were not used in the previous dam studies, which relied only on the concept of the MCE.

As flood and earthquake loadings represent extreme conditions, typically assumed not to be concurrent, reservoir spilling was considered not to be occurring at the time of the earthquake. Tower failure would impair reservoir drawdown capacity and water deliveries to the Authority's customers after the earthquake or in case of a subsequent flood, but would not cause sudden, uncontrolled release of the reservoir water. Major structural failure of the tower is unlikely to affect Sweetwater Dam other than by inducing cosmetic impact damage in the upper part of the dam, should the tower collapse toward downstream. Hence, tower failure would primarily represent a severe operational inconvenience and an economic loss to the Authority, rather than an immediate danger to the downstream area and population.

For the above reasons, in our deterministic approach based on largest magnitude and shortest distance assumptions for the MCE and other earthquake scenarios, we believe that mean response spectra (50^{th} percentile) are appropriate to assess the seismic performance of the tower. Therefore, we used such response spectra in our dynamic analyses and performance evaluation.

3.1.3 Probabilistic Seismic Criteria

The two closest faults that could generate the most severe ground motion at the site in case of rupture have experienced very low rates of activity in historic times. The La Nacion Fault is probably more a "capable" than an "active" fault, considering its apparent lack of Holocene activity. The Rose Canyon Fault, although definitively active, has also exhibited low rates of slip. For such reasons, probabilistic seismic hazard analysis (PSHA) is appropriate to consider for an essential but non-safety-related facility such as the Sweetwater outlet tower. Probabilistic ground motion estimates allow additional perspective on the conclusions derived for the MCE and other deterministic scenarios.

PSHA combines the contribution of all recognized faults or seismic zones around the site, including random seismicity, to assess the probability of experiencing various specified levels of local ground motion. The results are expressed as return periods or probabilities of exceedance during a given length of time, for seismic parameters such as the PGA or spectral accelerations. A numerical model of the greater area surrounding the site is normally required to develop probabilistic response spectra at various periods of vibration.

An alternative approach was used for this evaluation consisting of the use of results from the *National Seismic Hazard Mapping Project* of the USGS. That project has been ongoing for many years and includes a public database, accessible from the Internet, for nationwide probabilistic ground motion estimates computed at the nodes of a grid with 0.1 degree latitude/longitude intervals overlaying the entire United States. Gridpoint data are internally interpolated when querying the database to obtain estimates directly applicable to the geographic coordinates of the site. The web site provides PGA, and 0.2 second (s), 0.3s and 1.0s spectral accelerations with 10 percent, 5 percent or 2 percent probability of exceedance in 50 years. Assuming a Poisson's distribution of earthquake events and spectral shapes consistent with the local tectonic environment, these values can then be used to estimate seismic parameters for any return period and to develop approximate probabilistic response spectra. This simplified methodology was used herein. Spectral coefficients in-between the four periods provided were obtained by geometric (logarithmic) interpolation for an "average" magnitude level applicable to the region. Spectral coefficients for periods longer than 1.0s were assumed to be inversely proportional to the period considered.

3.2 Response Spectra

3.2.1 Deterministic Response Spectra

The 50th percentile mean horizontal PGA for the La Nacion MCE (M 6.7, distance 4 km) is 0.49g. The corresponding estimated 84th percentile PGA is 0.77g. These values were obtained by averaging predictions from four well-accepted and well-documented attenuation equations by Abrahamson and Silva, Boore and Joyner, Campbell, and Sadigh (see Seismological Research Letters, BSSA, January 1989). Attenuation equation parameters applicable to hard rock site conditions and strike-slip or normal faulting were used. The results obtained are consistent with ground motion estimates for the Sweetwater Dam site by previous consultants. Two of the aforementioned references also provide equations applicable to vertical ground motion. Deterministic horizontal and vertical PGA estimates for the six fault zones of interest to this evaluation are presented in Table 2.

It should be noted that both mean (50th percentile) and mean-plus-sigma (84th percentile) estimates are listed for completeness of the information provided. However, as previously discussed, we have recommended and used mean criteria for evaluating the tower seismic performance.

We obtained complete five percent damping bedrock response spectra through averaging estimates obtained from the same sets of attenuation relationships as used for the PGA. Both horizontal and vertical spectra were developed. Vertical response spectra at various damping values were based on the same reduction or amplification factors used for horizontal motion.

The magnitude dependent and distance dependent 50th percentile response spectra obtained for the six fault zones considered are presented in Tables 3 and 4. Horizontal and vertical response spectra are shown in Figures 3 and 4, respectively. These spectra represent a uniform level of reliability in the estimated ground motion over the entire range of periods considered. The probability of actual maximum ground motion along these six fault zones either exceeding or being less than these response spectra is exactly 50 percent.

3.2.2 Probabilistic Response Spectra

The web search of the USGS Seismic Hazard Mapping Project database provided probabilistic PGA estimates for the Sweetwater Dam site (latitude: 32.461 degrees, longitude: 117.000 degrees). The PGA with 10 percent probability of exceedance in 50 years is 0.21g. Such a value represents the potential contributions of all of the faults identified in the deterministic approach to the local seismic hazard. It also includes the possibility of random earthquakes (centered outside of well-recognized fault zones) occurring anywhere in the vicinity of the site. Ground motion parameters obtained from the USGS are presented in Table 5.

As mentioned in Section 3.1.3, Table 5 was used to obtain more complete response spectra for these probability levels, based on yearly rates of experiencing any specified acceleration at a given period. An earthquake representing an event with a 50 percent probability of occurring during a 50-year period was also used in analyses. The corresponding PGA is 0.06g and has a 72-year return period. The approximate horizontal response spectra developed for the three USGS probability levels and for the 72-year return period earthquake are shown in Figure 5.

The USGS database does not provide probabilistic vertical ground motion estimates. However, recognizing that the faults that are closest to the site (La Nacion and Rose Canyon) have relatively low rates of activity and are associated with the most demanding deterministic estimates, it can be reasonably concluded that probabilistic horizontal response spectra with long returns periods should be associated with more severe vertical motion, comparatively, than those with short return periods. This line of reasoning was followed to develop approximate probabilistic vertical response spectra for analysis purposes.

3.3 Damping Ratio

Response spectra for damping values other than 5 percent were developed by direct scaling of the 5 percent damping response spectra, using empirical equations (Newmark and Hall, 1982) for horizontal spectrum amplification factors at various damping coefficients. An example of application of this procedure is shown in Figure 6 for the 50th percentile La Nacion MCE. Similar spectral scaling factors were used for all other spectra to obtain spectral coefficients at damping values other than 5 percent.

Stone masonry is not a modern construction material, and little information is available regarding what damping levels should be expected under dynamic excitation. Limited information was found in a recent technical paper (Noret, Da Rin, Modaressi and Carrère, 1998). The authors describe full-size testing of stone masonry at Dardennes Dam, in southern France, using large size vibrating equipment of adjustable amplitude and gradually varying frequency from 1 to 20 Hertz (Hz). The first four modes of vibrations of Dardennes Dam were identified and ranged from 10.4 Hz to 18.4 Hz. The corresponding damping ratios were 13 percent of critical (mode 1), 12 percent (mode 2), 10 percent (mode 3), and 8 percent (mode 4). The authors concluded that the stone masonry infilling material actually resulted in a particularly high level of structural damping, even at a low level of deformation.

Severe earthquake shaking would likely be associated with damping levels higher than measured during forced vibration testing. Based on this consideration, for gross response analysis and "uncracked" initial condition of the Sweetwater outlet tower, we have considered a damping factor of 10 percent for the seismic input. It should be noted that because linear elastic response was considered, responses for damping levels other than 10 percent can be easily estimated through spectral response scaling based on the corresponding spectral ratios. The 10 percent estimate represents, in our opinion, a reasonable, moderately conservative value suitable for estimating the response of a non-safety related structure.

3.4 Response Modification Factor

A response modification factor, $R_{w,}$ is typically used to adjust spectral coefficients to be used for linear response analysis of reinforced concrete structures. This factor normally accounts for possible inelastic action, and is used to scale the elastic spectral amplitudes defining the seismic input to compute the earthquake demand (loads) more realistically than through linear-elastic response analysis.

Based on the classification of the Uniform Building Code (UBC), the intake tower cannot be described as a specific lateral-force-resisting structural system, but could be considered to be a distributed mass cantilever structure. The applicable R_w coefficient, per the UBC, would be 4.0. The use of such a value has been suggested for reinforced concrete intake/outlet towers in the USCOLD *Guidelines for Earthquake Design and Evaluation of Structures Appurtenant to Dams* (1995), but current U.S. Army Corps of Engineers (COE) criteria recommend a more conservative value of 2.0 for R_w in the case of the Maximum Design Earthquake (MDE), which is essentially equivalent to a MCE (Erikson, 1996). For lower levels of motion, such as the 72-year return period earthquake, Erikson used an R_w of 1.0. Because the Sweetwater tower is not steel-reinforced, and since stone masonry would exhibit little or no ductility, we did not use a response modification factor in this study. This is a conservative approach ($R_w = 1.0$).

3.5 Load Combination Factors

Three components of motion were applied simultaneously in the analyses. The same spectral shape was used for the primary and secondary horizontal components of ground motion, as no distinction was made between these two components in the development of the attenuation equations. However, peak loading in one horizontal direction is unlikely to occur simultaneously with peak loading in the other horizontal direction. To take this into account and as recommended by Goyal and Chopra (1989) for the evaluation of intake/outlet towers, we used load combination factors of 100 percent and 50 percent, respectively, for the primary and secondary components of ground motion. As the peak vertical response also occurs at a frequency different than the peak horizontal response, we used a load combination factor of 75 percent for the vertical component of motion, as assumed to occur concurrently with the peak horizontal excitation. The responses to the three components of motion were separately computed, and then combined by the Square-Root-of-the-Sum-of-the-Squares (SRSS) procedure.

The response analysis was used to assess what portion(s) of the tower would be expected to remain intact after occurrence of the specified seismic loads, and to estimate the overall stability of the structure against global overturning and base sliding. As the tower is not reinforced, post-cracking (cracked) response analysis is not applicable, and the tower must be assumed to fail if substantial overstressing of the materials comprising the tower occurs.

4. Parametric Response Analysis

4.1 General

This section describes the numerical analyses performed to evaluate the response of Sweetwater outlet tower to various earthquake ground motions. Two basic types of analyses were performed. The first is referred to as a “stress evaluation” in which the applied loads were compared to the estimated capacity of the tower structure. Capacity is based, in part, on the strength of the tower material. In this type of analysis, the applied loads were based on the assumption that the tower remained “uncracked,” even though computed stresses exceeded material strengths. The second was a “gross stability evaluation” in which the potential for overturning or sliding of the tower as a whole was considered.

Seismic evaluation criteria were discussed in Section 3.0. Masonry strength parameters were developed from a review of previous safety reports for Sweetwater Dam and non-destructive (Schmidt Hammer) testing performed as part of this evaluation.

We performed a parametric response analysis of the tower to account for uncertainties in the estimated strength and elasticity parameters of the tower material. The gross response evaluation was based on intact section properties, assumed to exist prior to the occurrence of any earthquake-induced cracking. The cracking capacity of the tower was developed for ranges of mortar strength and stone masonry stiffness, with average estimates based on the results of the Schmidt hammer testing.

We developed a structural engineering model for the Sweetwater outlet tower. The response of the model to the specified seismic loading was computed. The basic structural analyses involved the following steps:

- Select an appropriate analysis and results interpretation methodology, commensurate with the degree of refinement required,
- Specify analysis and strength parameters,
- Develop a numerical (finite element) model of the tower,
- Perform the analyses, and
- Interpret the results obtained and compare earthquake demand (induced loading) with capacity (ability of tower to withstand loading).

The above steps are discussed in the following sections.

4.2 Analysis Parameters

4.2.1 General

The tower response (uncracked condition) is governed by the stiffness and modulus of elasticity (E) of the stone masonry. The E-modulus of such composite material should be intermediate between those estimated for the cement mortar and the masonry stones.

Without full-size field-testing, it is difficult to assess the global stiffness of the masonry, which directly influences the tower dynamic response. For example, in earlier studies of Sweetwater Dam, Woodward-Clyde Consultants (1975) simply assumed the modulus of elasticity of the masonry portion of the dam to be between one-tenth and ten times that of the concrete gravity portion (3.5 million psi). Dames & Moore (1994) used 2.0 million psi for the static or dynamic modulus of the masonry. As a basis for comparison, Noret, et al. (1998) reported static moduli of elasticity for stone masonry used in dam construction ranging from about 0.3 to 3.9 million psi, with a mean value of about 2.9 million psi. Dynamic E-moduli reported by these same authors ranged from about 1.7 to 4.5 million psi. A best estimate for the E-modulus of the tower material was developed on the basis of our literature review, and considering the apparent good quality of the Sweetwater outlet tower masonry. Analyses were performed using the best estimate of E-modulus, as well as a lower and higher value to assess the sensitivity of results to E-modulus.

The seismic capacity (stress analysis or stability analysis) of the Sweetwater outlet tower will be governed by the shear and tensile strengths of the masonry mortar, and its bond strength with the masonry stones and tower foundation. Therefore, we have focused on developing strength properties for the mortar, as discussed in the following section. As with the E-modulus, analyses were performed using best estimates of strength parameters, as well as lower and higher values to assess the sensitivity of results to strength parameters.

4.2.2 Masonry Mortar

The Schmidt hammer manufacturer lists an instrument error of plus or minus 15 percent for tests performed in accordance with ASTM C-805 guidelines. However, in similar studies of intake/outlet towers, Bureau and Scawthorn (1986) and Bureau (1985, 1993) found less than ten percent difference in estimated average compressive strengths (f'_c) between laboratory and Schmidt hammer tests on concrete. Based on that experience, the strength properties derived from the Schmidt hammer testing of the dam mortar for this investigation (assumed similar to the tower mortar) are judged to be reasonable for this analysis.

Schmidt hammer measurements provide a rapid loading (dynamic) compressive strength for the mortar. From the measured rebound values, the average estimated dynamic compressive strength is about 3,914 psi, with a standard deviation of about 975 psi. Based on the common assumption that the rapid loading compressive strength is about 20 percent higher than the sustained loading compressive strength, this corresponds to an average static compressive strength of about 3,260 psi for long-term, sustained loading.

We successively assumed dynamic compressive strengths of 2,900 psi, 3,900 psi or 4,900 psi in our parametric evaluation of the tower structural capacity. Such dynamic strengths approximately correspond to average-minus-one standard deviation (sigma), average, and average-plus-sigma static compressive strengths of about 2,415 psi, 3,250 psi or 4,085 psi, respectively. The above values likely bound the in-situ compressive strengths of the Sweetwater cement mortar.

Other properties necessary for the analysis, such as Poisson's ratio and modulus of elasticity, were estimated from empirical formulas and values provided in American Concrete Institute (ACI) Standard 318, *Building Code Requirements for Structural Concrete.* An average Poisson's ratio of 0.15 was used, with high and low estimates of 0.12 and 0.18. A unit weight of 150 pcf for the mortar was used. Knowing the compressive strength, ACI-318 and other empirical formulas were used to obtain estimates of the static shear and direct tensile strengths and the modulus of rupture (bending) of the cement mortar. Although established for the design of conventional reinforced concrete buildings, these formulas apply reasonably well to assessment of the outlet tower.

As discussed above, it is well known that concrete cores tested in either tension or compression exhibit higher strength under rapid than slow loading condition. Hence, it is common practice (USCOLD, 1985; 1999) that both concrete strength and modulus of elasticity be increased for earthquake (rapid) loading condition. Such increase factors were assumed applicable to the mortar of the Sweetwater tower. The following dynamic increase factors were selected, based on precedents and a history of approval for similar projects by regulatory authorities, such as the Federal Energy Regulatory Commission (FERC) and the DSOD:

- Compressive Strength: 20 percent increase
- Modulus of Elasticity: 25 percent increase
- Tensile Strength: 40 percent increase
- Shear Strength: 30 percent increase

The mortar dynamic shear strength (v_c) and direct tensile strength, based on ACI and other well-accepted formulas, were used to establish the cracking (gross) capacity of the tower for shear and moment loading, respectively. For reinforced concrete towers, the modulus of rupture is normally used to establish the bending capacity. However, because of the irregular failure surfaces likely to occur in stone masonry, the use of the direct tensile strength was considered more prudent. Our tensile strength estimate is lower than the "apparent" seismic tensile strength and is, therefore, believed to be sufficiently conservative. The concept of "apparent" strength, first introduced by Dungar (1981), was generalized by Raphael (1984) to define what strength value should be used to interpret the results of linear-elastic analysis of structures built with concrete, a material known to behave nonlinearly. The masonry mortar properties developed for the seismic analysis of the Sweetwater tower are listed in Table 6. Detailed information on how these properties were selected are provided in the following paragraphs.

For comparison, three masonry outlet towers of the same vintage (1873 to 1894) as the Sweetwater tower and located in California, (Lower Crystal Springs, Lake Frey and Pilarcitos towers), also exhibited high quality mortar when tested in the field or the laboratory, with shear and tensile strengths greater than 400 psi and 300 psi, respectively. Our estimated mortar strengths, based on the Schmidt hammer data, are consistent with that other experience.

Essential to the evaluation of the Sweetwater tower is the bond strength that can develop at the stone-mortar interface. Failure in response to induced dynamic tensile stresses is likely to occur at the interface between the stones and mortar, rather than through the mortar itself. The quality of the bond between these two materials depends on the care that was given during construction to clean and wet the contact surfaces. Proper preparation will normally achieve most of the strength of intact mortar at the interface. While the quality of construction was reported to be excellent, details of mortar and stone surface preparation are unknown. Hence, it is prudent to assume that the tensile strength at stone-mortar joints is less than that of intact mortar. Based on data available from the literature, concrete lift joints with no prior surface treatment achieve between 31 and 83 percent of the tensile strength of intact concrete. Joints formed by placing new concrete on a dry or wet prepared surface achieve higher strengths. Using data reported by Waters (1954) and Tynes (1959, 1963) at the U.S. Army Corps of Engineers, strength reduction factors for clean-brushed and hand-compacted lifts on hard surfaces with no other prior surface treatment range from 0.55 to 0.74. We took the mean value of these reduction factors for unprepared surfaces, or 0.58, as being applicable to the Sweetwater masonry, in the absence of other information. Therefore, the estimated dynamic tensile strengths of masonry mortar joints, after reduction at stone contact level, are 196 psi, 264 psi and 332 psi, at the average-minus-sigma, average, and average-plus-sigma levels, respectively. These values represent the dynamic direct tensile strength of the mortar, multiplied by the joint strength reduction factor of 0.58.

The above reduction factor corresponds to a perfectly plane horizontal failure surface in direct tension. Two other factors were considered. First, the estimated modulus of rupture, as defined by the ACI, could be a better parameter than the estimated direct tensile strength for defining the capacity to resist moment loading. However, its applicability to masonry mortar is unknown. Secondly, because masonry stones have various shapes and sizes, actual failure along stone-mortar contact surfaces could be irregular and involve an area larger than defined by a horizontal plane. Taking the above factors into account, we have increased by 25 percent the reduced stone-mortar bond strength defined above. Hence, in our analyses, we have taken the dynamic strength of the stone-mortar joints as being 0.725 times (0.58 x 1.25) the dynamic strength of intact mortar. Dynamic tensile strength values for equivalent horizontal joints were successively taken as 245, 330 and 415 psi for average-minus-sigma, average, and average-plus-sigma levels, respectively.

4.2.3 Stone Masonry

Stone masonry can be difficult and potentially very costly to core and test in the laboratory. Therefore, a qualitative assessment of the stone and mortar condition, where visible, and non-destructive testing of these materials were performed for this investigation. The compressive strength of the mortar and stones, where exposed near the dam abutment, could be measured using a Schmidt Hammer, and the quality of the bond between stone and mortar visually assessed. The local stone is dense and very hard based on our field testing. A unit weight of 168 pcf was assumed for the stone masonry. In the absence of specific data, the same estimates of Poisson's ratio used for the mortar were also used for the masonry.

Estimates of the E-modulus of the stone masonry (mortar plus stones) were used in analyses to estimate tower response to dynamic loading. The E-modulus of the masonry is expected to be greater than the E-modulus of the mortar alone because of the influence of very hard stones in the masonry. Based on empirical relationships between E-modulus and compressive strength, and using compressive strength measurements of mortar at the dam site, we estimated a static E-modulus for the mortar alone at about 3.5 million psi. This corresponds to a dynamic E-modulus for the mortar of about 4.4 million psi. For analysis purposes, we assumed a "best estimate" dynamic E-modulus for the masonry (mortar plus stones) of 5.0 million psi. Parametric analyses were performed for dynamic E-moduli ranging from 1.25 to 8.75 million psi. This range is broader than the range of dynamic moduli for stone masonry reported by Noret (1998).

4.2.4 Foundation Bedrock

As the tower is free-standing on a competent, hard foundation, no significant interaction between the bedrock and tower structure would be expected. The shaft is flexible compared with the underlying half-space and the slender tower is a relatively low mass structure. For analysis purposes, we considered the foundation materials as infinitely stiff, compared with the more flexible free-standing tower shaft. Hence, we assumed the tower to be rigidly connected to bedrock at El 139, a conservative assumption as it ignores radiation damping and interaction effects.

To evaluate the gross stability of the tower for overturning and sliding, we assumed a dynamic bond strength of 50 percent of the stone-mortar joints in the tower and a friction angle of 35 degrees at the masonry-bedrock interface. The reduced bond strength accounts for possible weaker contact, in the event the foundation surface was prepared with less care than used for the tower walls. This friction angle is believed to be conservative because the hard foundation substratum should be capable of developing significant dilatancy resulting from uneven contact surfaces and rock asperities at the masonry-bedrock interface.

4.3 Methodology and Analysis

4.3.1 Methodology

Dynamic, three-dimensional (3-D), finite element response spectrum analysis was used to calculate the structural demand (imposed loading) on the tower under the specified input motion. Analyses were performed using the computer program SAP2000 (1999). Facilities such as the Sweetwater outlet tower primarily behave as vertical cantilever beams in resisting earthquake motion. However, as the Sweetwater tower appears not to be anchored to the substratum, bottom uplift or overturning may represent a potential concern for this structure under severe earthquake loading. The analysis steps followed the basic approach described in *Section 5 - Intake/Outlet Towers* of the *Guidelines for Earthquake Design and Evaluation of Structures Appurtenant to Dams* (USCOLD, 1995), and included:

- Develop a numerical model for the structure,
- Define the significant modes and frequencies of vibration,
- Calculate induced loads (moment and shear) as a result of the specified earthquake shaking,
- Combine static and dynamic loads, and
- Compare these loads with gross (uncracked) shear and moment capacities of the tower shaft (stress evaluation).

The modal characteristics of the tower and its response were computed using a finite element numerical model. A three-dimensional system of flexural beam elements and lump masses was used to represent the tower, as shown in Figure 7. The tower shaft was assumed cantilevered at El 139, which represents a point of fixity (Node 1) when the tower vibrates.

A limited number of beam elements (20 or less) is amply sufficient to model this type of structure (USCOLD, 1995; Bureau, 1993), due to its relatively simple vibration characteristics. The tower does not contain equipment other than the saucer valves and their winches at the top platform. The masses of the walls and external appendages (top platform, valve operators and saucer valves) were lumped to the appropriate nodal points of the mathematical model.

In the calculations, areas and moments of inertia were adjusted to account for the encroachment of valves 7, 8 and 9 into the tower hexagonal section. Key nodes of the model were placed at the center of the valve inlets. Nodal point mass assignments were adjusted to account for the valve inlet wall openings and the weight of the steel valves. A small portion of the outlet conduit, between the tower and valve 2, was assumed to form an integral part (no joint) of the tower base. The top node of the numerical model was placed at the center of gravity of the roof concrete platform. The masses of the platform, winches, winch pedestals and roof assembly were lumped to that node.

A steel footbridge connects the operating platform of the tower and the crest of the dam. The footbridge has no intermediate supports and is a very light structure (1,788 lbs), compared with the tower itself. Based on other experience and comparative studies performed with or without including the bridge for other similar towers, structural interaction between the bridge and the tower should be negligible. This is because the mass of the bridge (steel structure) is very low, compared with that of the tower (concrete and masonry structure). For the above reason, the entire mass of the footbridge was simply lumped to the applicable nodal point near the top of the tower (Node 20). This corresponds to the assumption that the bridge would move in phase with the tower by sliding on the dam crest support pads.

4.3.2 Basis for Modal Analysis

For seismic analysis purposes, it is customary not to combine flood with earthquake loading. A high water level is the most critical in the case of outlet towers. Because the reservoir water elevation normally fluctuates, we have assumed for analysis purposes that the reservoir elevation would be at its maximum normal operating level, defined by the south spillway crest level at El 237. The modal characteristics of the tower under empty reservoir condition were not defined, because in the case of a slender tower such as this one, the full reservoir case is the most critical.

Authority personnel indicated that the water level inside the tower shaft is frequently at a high level. Therefore, for analyses we assumed the inside water level as the same level as the reservoir. We verified that assuming the tower full of water resulted in larger response than if the inside shaft was empty. Based on our calculations, the filled tower has a fundamental period 1.8 percent longer, and computed moments for the case of the 475-year earthquake were about 3 percent higher, than when dewatered. Hence all the results discussed in this report assume the inside and outside water levels to be at El 237.

Loose, compressible sediments (reservoir siltation) surround the lower part of the tower up to a level between valves 3 and 4. Reservoir sediments are typically denser (e.g., 81 to 83 pcf, measured at Searsville Dam, CA) and have lower compressive wave velocities (about 1,000 feet/second, also measured at Searsville Dam) than water (4,800 feet/second). Hence, the reservoir silt slightly restrains the lower part of the tower and perhaps dampens traveling compressive waves near the tower, thereby reducing potential hydrodynamic pressures on the tower wall. However, such effects are difficult to quantify, and we ignored the presence of the silt in our hydrodynamic equivalent masses calculations. We simply assumed the tower to be submerged in water from its base to the assumed reservoir elevation.

The following tables summarize the analysis model properties developed for the uncracked tower (assumed existing condition):

- Table 7: Nodal Points Coordinates
- Table 8: Cross Section Areas
- Table 9: Sections Moment of Inertias
- Table 10: Nodal Point Masses

4.3.3 Parametric Gross Response Analysis Methodology

The parametric gross (uncracked) response analysis is intended to determine the factors of most significance to the tower response. Following identification of such factors, the tower model can be fine-tuned, as needed, or conclusions derived based on other assumptions. The deterministic and probabilistic seismic criteria were successively applied.

Typical parametric analyses include the influence on response, and/or gross capacity, of the concrete or masonry strength, reservoir water level, and factors such as the structure and foundation flexibility or the presence of appurtenances, such as access bridges or heavy equipment. The Sweetwater tower contains no equipment other than the external saucer valves, and was assumed surrounded by water at spillway crest level. Therefore, the principal factors of significance to its gross response are the seismic criteria and modulus of elasticity of the stone masonry. As previously stated, the light footbridge was not separately considered in the analyses, but its influence on the computed tower response was accounted for.

4.4 Influence of Masonry Stiffness and Strength

4.4.1 Frequencies of Vibration

The frequencies of the significant modes of vibration of the Sweetwater outlet tower were calculated for the average stiffness estimated for the tower masonry, as well as for the low or high estimates. The tower was assumed cantilevered at foundation level (El 139). For the "best estimate" of the dynamic modulus of elasticity (5 million psi), the following frequencies were calculated:

Bonding Mode	Frequency, Hz		Period, Seconds	
	X Direction	Y Direction	X Direction	Y Direction
Fundamental	2.20	1.95	0.45	0.51
Second	11.5	10.6	0.09	0.09
Third	28.9	27.4	0.03	0.04

Notes: X-Direction is upstream/downstream
Y-Direction is cross-valley

The first vertical mode frequency was calculated to be 25.3 Hz. Modal frequencies are summarized in Table 11 for the range of masonry stiffnesses considered. Except for its first few modes of vibration, the dewatered tower behaves as a rigid structure (frequencies higher than 33 Hz). However, the first bending modes in the upstream/downstream (X) and cross-valley (Y) directions, which have the largest mass participation factors, occur at frequencies lower than the frequency at which the peak acceleration of the specified response spectrum occurs (about 5 Hz, see Figure 5). Hence, when the strength (and stiffness) estimates of the mortar increase, the response of the tower also increases, which makes the computed demand-to-capacity (D/C) ratios less sensitive to the uncertainty regarding the masonry parameters.

4.4.2 Uncracked Tower Capacities

The two critical modes of response of typical intake/outlet towers are for shear and bending loads, bending being generally the most critical by a substantial margin. Hoop (horizontal) tensile loading is of no concern for towers. Hence, only bending and shear loads were considered in this study.

Intact Masonry Mortar

For the average compressive strength estimated for intact mortar from the Schmidt Hammer measurements, we estimated a <u>static</u> direct tensile strength of 325 psi and a <u>dynamic</u> tensile strength of 455 psi (40 percent increase). This value is less than the ACI modulus of rupture or the apparent dynamic tensile strength (746 psi) suggested by Raphael (1984) for linear-elastic analysis of mass concrete structures. In this study, we also used lower and upper bounds of 338 psi and 572 psi for the dynamic tensile strength of intact mortar, based on the range (plus or minus one standard deviation) of compressive strengths estimated from the field measurements. The average tensile strength and this range of values were used to calculate “best” and upper- and lower-bound estimates for the gross moment capacity (cracking moment) at various elevations along the tower shaft.

Normal compressive loads, such as dead load (gravity), increase the moment-resisting capacity of individual tower cross-sections, based on their elevation within the shaft. Upward or downward earthquake accelerations can increase or reduce the initial vertical loads acting across tower sections, and affect their moment-resisting capacity. Instead of successively combining the (+) or (-) vertical earthquake loading with the two horizontal components of loading, it is equivalent in the simple model considered to reduce the effective section capacity by subtracting the most critical (upward) computed vertical dynamic loads from the static gravity loads in the cracking moment capacity calculations. This simplified procedure is appropriate, as there is essentially no contribution of the vertical accelerations to the overturning moment in this near-axisymmetric structure. Hence, instead of combining vertical with horizontal loads in the demand calculations, we simply adjusted the gross section bending capacities, taking into account the calculated maximum dynamic upward loads. We combined the two components of horizontal loading to compare earthquake demand with the available gross capacity of individual sections, adjusted for axial loading.

We computed the factored capacity of the tower wall (cracking of intact mortar) using a strength reduction factor in bending of 0.90, as recommended in the American Concrete Institute (ACI-318) requirements. This strength reduction factor provides a factor of safety in the calculation of the gross capacity of the structure. Table 12 shows the calculated factored gross moment capacities (cracking moment M_{cr}) of the Sweetwater outlet tower, based on the assumed range of strengths for intact mortar.

Stone-Mortar Joints

As previously discussed, a potential concern is how the presence of cracks and irregular mortar joints affects the tensile strength. We assumed that potentially weak and irregular joints in the tower would develop a bond strength of 72.5 percent of the strength of intact mortar. Hence, the mean dynamic tensile strength of "equivalent horizontal" stone-mortar joints was assumed to range from 245 psi to 415 psi, with an "average" estimate of 330 psi. Moment capacities based on the bond strength of mortar-stone joints are shown in Table 13.

The factored gross shear capacity of the uncracked tower was also calculated for a range of values assumed for the masonry mortar compressive strength. We also followed the principles described in ACI-318 to compute shear capacities, and used a shear strength reduction factor of 0.85. As assumed for moment loading, joints would potentially affect the shear capacity of the Sweetwater outlet tower. A stress increase factor of 1.855 (approximately circular section) was used to account for the fact that the tower walls are not uniformly stressed. The calculated uncracked shear capacities of the Sweetwater outlet tower, through intact mortar or along stone-mortar joints, are presented in Tables 14 and 15, respectively.

4.4.3 Response Spectrum Analysis

The first 40 modes of vibration of the tower were included in the analysis, which corresponds to a combined mass participation greater than 98 percent and, therefore, sufficient accuracy. For the analysis of the tower in an uncracked condition, we used response spectra, including those shown on Figures 3 to 6, as a basis to define the peak horizontal and the vertical components of ground motion. We used 10 percent damping for the structural response, as previously discussed.

The vertical static stresses are well below the allowable compressive strength of the tower wall. Hoop stresses in the tower wall cannot be calculated in a stick (3-D beams) model, but have been shown to be essentially negligible in detailed studies of other intake/outlet towers. Static and dynamic hoop stresses would also be very small. Therefore, under an earthquake loading condition, the most critical dynamic loads for the tower are bending (overturning) moments and shear forces. Based on other experience (Bureau, 1986, 1993), earthquake-induced compressive and torsion loads do not represent a potential problem for this type of structure. Therefore, our dynamic response analyses and interpretation was focused on induced bending moments and shear forces at various elevations along the shaft.

We used the principles described in ACI-318 as guidelines to evaluate the performance of the Sweetwater outlet tower. ACI-318 criteria are based on the strength design method, and normally use load factors (greater than 1.0) and strength reduction factors (smaller than 1.0) to compare induced stresses with the available structural capacity of concrete (or cement mortar). Several combinations of static and dynamic loads and a l33 percent dynamic overstress allowance are normally used.

In the absence of live loads, as is the case for outlet towers, conventional application of ACI-318 requirements would define combined loads for earthquake loading as the most critical of the following:

$$\textit{Total Load} = 1.05 \textit{ Static Load} + 1.40 \textit{ Dynamic Load} \qquad [4\text{-}1]$$

or

$$\textit{Total Load} = 0.90 \textit{ Static Load} + 1.43 \textit{ Dynamic Load} \qquad [4\text{-}2]$$

where *Load* represents either bending, axial or shear loads.

Instead of using the above equations, we selected static and dynamic load factors equal to 1.0. The reason for this modification of the code formulas is that, in ACI-318, the earthquake load factors provide an additional margin of safety when pseudo-static earthquake forces are computed through code formulas (seismic zoning of the Uniform Building Code and applicable source factors N_A and N_V). The load factors defined by equations [4-1] and [4-2] would be required if a new structure were to be designed. For response spectrum analysis and the evaluation of an existing, older structure, load factors equal to 1.0 are appropriate. This is because a rigorous definition of the earthquake demand (response spectra) has been used and an estimate of the true seismic vulnerability is desired. We also used the dynamic overstress allowance factors described previously, rather than ACI's 133 percent.

Since the orientations of the specified directions of earthquake loading are unknown, we vectorially combined peak shear forces and bending moments calculated in the X and Y directions of shaking to obtain upper bound estimates of the peak forces and overturning moments at various elevations along the shaft, a conservative approach. As discussed earlier in this report, the assumed spectral shape of the secondary horizontal component was the same as that representing the primary component, but was used with a vectorial combination factor of 0.50. The vertical response spectrum was assigned a combination factor of 0.75.

4.5 Results of Analyses

4.5.1 Stress Analysis

Gross Moment Response

- For the La Nacion MCE, the largest D/C (demand/capacity) ratio for moment loading at stone-mortar joints was 6.61 for the average ("best estimate") stiffness and strength properties (dynamic modulus of elasticity of 5.0 million psi). D/C ratios higher than 1.0 indicate non-compliance with the specified performance criteria. Lower D/C ratios than for the average condition were computed for the lower-bound assumptions (D/C = 4.70) and upper bound assumptions (D/C = 6.26). Hence, the "best estimates" represent the most critical combination of analysis parameters and strength properties for the three combinations considered. D/C ratios for moment loading for the La Nacion MCE are presented in Table 16 for intact mortar or stone mortar joints. These D/C ratios indicate unacceptable performance, based on the performance evaluation criteria discussed in this report.

- For the Rose Canyon maximum earthquake, the most critical D/C ratio was 3.72, again for the "best estimate" conditions at joint level. Computed D/C ratios are presented in Table 17.

- Complete results are presented for the 475-year probabilistic seismic criteria. Table 18 shows the computed moment response of the Sweetwater tower for the three assumptions regarding the E-modulus of the masonry. As previously discussed, overturning moments induced by the two horizontal components of motion (X and Y directions) were combined by the SRSS procedure. Induced moments and factored gross capacities (at mortar joints level) are graphically compared in Figure 8. D/C ratios are listed in Table 19. Two conclusions can be drawn from Figure 8 and Table 19. First, and for this postulated earthquake loading, the gross capacity of the shaft is exceeded for a significant extent (from about El 139 to about El 198), for the average conditions considered. The most critical location is located immediately above the outlet conduit (D/C = 2.95). Secondly, masonry strength and stiffness have limited influence on the available capacity and moment response. The most critical condition corresponds to the "best estimates" of stiffness and strength properties.

- Moment response was also calculated for the 72-year earthquake. Depending on the assumed properties, D/C ratios at the most critical location ranged from 0.59 to 0.83, indicated that the tower should be capable of withstanding moments resulting from such an event, based on the specified performance criteria.

The Sweetwater outlet tower is not capable of withstanding an MCE (La Nacion Fault) or a maximum earthquake along the Rose Canyon fault. It appears not capable of withstanding the probabilistic event with 10 percent probability in 50 years, which may better represent the seismic hazard at this site than the two deterministic events, because of the low slip rates of the La Nacion and Rose Canyon faults.

Gross Shear Response

- For the La Nacion MCE, the maximum D/C ratio was 1.63 for the average analysis conditions. Although this is unacceptable performance (D/C greater than 1.0), it is considerably less critical than the moment loading case. D/C ratios for "low" and "high" mortar property estimates are 1.28 and 1.66, also indicating potential overstressing, but less critical than for moment loading. Therefore, moment loading controls the seismic performance of the Sweetwater outlet tower.

- For the Rose Canyon maximum earthquake, the highest computed D/C ratio for shear loading was 1.05, indicating questionable performance, but a probably stable tower for this mode of failure, because a strength reduction factor of 0.85 for shear was used.

- For the 475-year probabilistic earthquake, the highest D/C ratio for shear loading was 0.85, indicating compliance with our performance criteria for the three assumptions regarding the strength of the stone-mortar joints. Average induced shear forces are shown in Table 20, and D/C ratios in Table 21. A graphical comparison between shear forces induced by this probabilistic earthquake and the shear capacity is shown in Figure 9, at stone-mortar joint level. Compared with the bending capacity, the shear capacity of the tower is considerably less critical.

- The maximum D/C ratio computed for the 72-year earthquake was 0.21, indicating satisfactory performance.

Overall, shear response is considerably less critical than the moment response for the Sweetwater outlet tower.

4.5.2 Gross Stability Analysis

In addition to possible overstressing, we evaluated the stability of the tower against global overturning (toppling) and sliding along its base. This was done by comparing the moment at the bottom of the tower with the overturning capacity, and the base shear with the frictional resistance along the tower bottom. Tower toppling was assumed possible around a rotation point along the tower bottom perimeter, perpendicularly to the outlet conduit. Resistance to overturning is provided by the resultant of the moments of the buoyant weight of the tower masonry and inside water and the total bond force available at the masonry-foundation interface. Resistance to sliding is provided by frictional and bond forces along the tower-foundation contact.

The highest computed instantaneous D/C ratio for global overturning, based on the computed peak moment at the bottom of the tower, was 2.12 (La Nacion MCE). For this earthquake scenario, D/C ratios for global overturning ranged from 1.43 to 2.12, for the three assumptions regarding the masonry dynamic stiffness and assumed dynamic bond strength at the foundation contact (122.5 psi, 165 psi or 207.5 psi). As in the stress evaluation, the worst condition was for the assumed "best estimate" properties.

The highest D/C ratios computed for the other earthquake loading assumptions were 1.25 (Rose Canyon MCE), 1.00 (475-year earthquake) and 0.28 (72-year earthquake). Hence, the tower could become unstable for global overturning for the two most demanding deterministic events (La Nacion and Rose Canyon), and is marginally stable for global overturning under probabilistic criteria with a return period of 475 years.

It is conservative to use a peak dynamic moment to compute a factor of safety against overturning using equivalent static moment equilibrium considerations. The peak dynamic loading would be applied only for a short instant of time, and the direction of loading application would constantly change during the duration of the earthquake shaking. Conceivably, the tower might uplift or oscillate from one side to the other under load reversals, without being out of plumb, if the bending capacity of the shaft were not exceeded. However, failure of the masonry through overstressing is likely to occur before failure by overturning and would control the seismic performance of the tower.

For the maximum applied base shear (V_{max}), and assuming a masonry to bedrock friction angle of 35 degrees and bond strength of 50 percent of that used for mortar joints, the factors of safety against sliding are considerably higher than for overturning. Therefore, sliding at the base of the tower was not considered further.

4.6 Interpretation of Results

Based on the results presented in Tables 16 through 20 and depicted in Figures 8 and 9, the earthquake-induced moment demand substantially exceeds the available cracking capacity of the Sweetwater outlet tower for the La Nacion MCE, Rose Canyon maximum earthquake, and the probabilistic 475-year earthquake. Induced shear forces, as well as global overturning and sliding stability are considerably less critical. While the lower or upper bounds of estimated masonry strength influenced capacity, as expected, the loads induced on the tower were affected by corresponding changes in stiffness, and the average (best estimate) assumptions regarding the masonry properties turned out to be the most critical. Overall, computed D/C ratios exhibited moderate sensitivity to the wide range of postulated strength and stiffness of the stone masonry.

Stresses were calculated assuming a stable tower, cantilevered at foundation level. Under severe seismic moment loading, the tower could become unstable. Hence, the assumption of the base being fixed would no longer apply. Base uplift and masonry mortar cracking above the outlet conduit would make the response highly nonlinear and significantly affect the actual loads. Loads, but also the capacity, should decrease considerably if the response became non-linear. However, as the gross moment capacity of the shaft was largely exceeded, we concluded that, as a minimum, the masonry mortar would crack extensively.

Partial or complete collapse of the Sweetwater outlet tower as a result of extensive cracking and possible instability against overturning are probable, under the postulated MCE, Rose Canyon or 475-year earthquake events. The structure response to any of these three scenarios would be inelastic and the tower, in its assumed existing condition, would experience major cracking of the masonry mortar, likely resulting in partial to complete collapse due to excessive earthquake-induced bending moments.

Out-of-phase movements between the tower and the crest of Sweetwater Dam appear to be of limited concern under elastic response, as long as the four bolts that tie the access bridge to the tower are not pulled out. Maximum tower platform elastic displacements (Node 21) under the most demanding MCE earthquake loading condition ranged from 1.3 to 4.2 inches for the three assumptions regarding the E-modulus of the masonry. Out-of-phase elastic displacements of the dam crest would be less than the displacements noted above. However, extensive cracking (inelastic response) is likely for the MCE, resulting in partial or complete collapse of the tower, rendering the footbridge performance a moot point.

5. Maximum Sustainable Earthquake Loads

We believe the analyses reported herein are conservative, although not excessively. Such conservatism was required to compensate for the uncertainties regarding the seismic exposure of the site and the masonry properties. In this section, we provide an opinion on the maximum earthquake loading that the tower could withstand without experiencing unacceptable cracking or major structural failure. We assessed more realistically what the expected performance of the tower might be, essentially without using any implied "factors of safety" in the performance evaluation.

First, we recomputed the D/C ratios for moment loading, using an assumed root-mean-square moment (M_{rms}) equal to 0.7 times the peak moment (M_{max}). We then eliminated the strength reduction factor (0.9) required by the code in the moment capacity calculations. This reduced the previously calculated D/C ratios by 37 percent, but still indicated insufficient overturning capacity for the MCE, Rose Canyon and 475-year probabilistic events.

Based on the recomputed "realistic" D/C ratios, the tower appears capable of safely withstanding ground motion with a peak ground acceleration of about 0.11g. For the local tectonic environment, this corresponds to a seismic event with a probability of occurrence of approximately 29 percent in 50 years, or 50 percent in 100 years. Hence, within the next century, the Sweetwater outlet tower has a 50 percent chance of either experiencing major failure or remaining stable during a seismic event.

6. Evaluation of Outlet Conduit

6.1 General

The outlet conduit consists of a masonry structure of rectangular section (17.3 feet x 6.5 feet) with a short wall (2.3 feet high by 1.5 feet wide) at its top on the right-abutment side. It contains three unlined water lines, 40-inches, 14-inches and 18-inches in diameter, respectively. The top of the conduit is at about El 155.

According to drawings provided to us, clay filling was placed on the right-abutment side of the conduit (see Figure 2). No details are available regarding the clay filling properties and extent. The Authority indicated that the surface of loose reservoir sediments is between valves 3 and 4, or about El 170. Hence, the outlet conduit should be buried in about 15 feet of loose sediments above the top of the conduit.

6.2 Gross Stability of Outlet Conduit

The presence of the reservoir sediments should help to stabilize the outlet tunnel for global overturning or sliding along its base. However, the extent of these sediments and their physical properties are unknown. A worst-case assumption would be if there are no sediments or if they are so loose that they behave essentially like a fluid. In that case, the conduit would be exposed to horizontal and vertical earthquake forces, plus hydrodynamic pressures under full reservoir head (El 237). This scenario was used to assess the gross stability of the conduit, assuming sliding along its base or toppling around the edge of its base would become possible.

Horizontal and vertical upward earthquake loads contribute to the overturning moment, as well as hydrodynamic pressures. Because the outlet conduit is a short, massive structure, it should respond as a rigid body (fundamental frequency of 33 Hz or greater to earthquake motion). However, for the purpose of taking a conservative approach, we assumed that the peak horizontal and vertical ground accelerations would be amplified by a factor of 2.0 at the top of the approximate center of gravity of the conduit (taken as the center of its rectangular section). Hydrodynamic pressures were assumed to be exerted along the conduit structure side, and along the side of its small crest wall. Resisting forces consist of the buoyant weight of the conduit and bond forces at foundation-conduit level. As previously discussed, the bond strength at foundation level was assumed to be half of that of the stones and mortar in the masonry. An additional analysis was also performed, using an assumed lower-bound bond strength of 50 psi. The D/C ratios for moment loading under the various assumptions regarding the bond strength are summarized in Table 22 for the four earthquake loading conditions considered.

Based on the above results, the outlet conduit should be stable for overturning for any earthquake event other than the postulated MCE along the La Nacion Fault, and with a lower bound estimate of foundation-conduit bond strength. In any case, the outlet tower would fail well before the conduit, whose overturning capacity should be of limited concern. As in the case of the tower, the sliding stability of the outlet conduit is less critical than its overturning stability and was not considered further.

6.3 Seismic Wave Passage Considerations

6.3.1 General

The outlet conduit structure and opening could be sensitive to seismic wave passage through the foundation and reservoir water or sediments. The conduit structure was built of stone masonry, and the diameter of its largest pipe is 40 inches. The influence of the two smaller pipes is negligible. Outlet conduit response to wave passage can be estimated based on two simplified bounding assumptions: (1) taking the largest pipe as an unlined 40-inch diameter circular opening in a hard medium (stone masonry) or, (2) assuming a lined opening (the lining being the conduit masonry itself) in very soft ground consisting of the reservoir sediments. The smallest side thickness of the conduit wall (15 inches) defines the most critical lining thickness, in the case of the second assumption. The reservoir hydrostatic pressure and gravity loads define the initial state of stress of the masonry for both of these assumptions.

For practical purposes, the 40-inch diameter conduit opening can be considered as near-rigid and, therefore, will not experience significant induced seismic stresses as a result of earthquake-induced "racking." It should not significantly amplify ground motions, and the only requirement is that it does not experience excessive strains as a result of seismic wave passage, which would cause potential cracking in both the transverse and longitudinal directions. Depending on their directions of travel, seismic waves could induce axial, bending or hoop strains that might affect the conduit opening perimeter. As the conduit structure is built of unreinforced masonry, under assumption (2), it will act as a "rigid inclusion" and will tend to resist surrounding reservoir sediment movements.

For assumption (2), the properties and, especially, the elastic and bulk moduli of the reservoir sediments are unknown. Such materials are expected to be extremely loose and transmit compressive waves at velocities close to, or substantially lower than that of water (4,720 feet/second). For this study, we considered two extreme compressive wave velocities through the Sweetwater Reservoir sediments, 500 feet/second (fps) or 4,800 fps, recognizing that somewhere between 1,000 fps and 2,000 fps is the most likely value.

Among other factors, induced seismic strains will depend on the velocity of the traveling waves (V_s or V_c), on the particle velocity (V) of the earthquake-induced ground motion, and on the ratios of the shear and compression moduli of the masonry and surrounding sediments. Seismic performance can be estimated by computing stresses and strains induced by wave passage using simplified mathematical solutions, and by comparing such stresses and the corresponding strains with threshold values representative of typical concrete seismic performance, such as “cracking” or “crushing” strain limits. For concrete, the “cracking limit” is typically taken as 0.04 percent strain, and the “crushing limit” is typically taken as 0.4 percent strain. The same limits were used to assess the masonry mortar, even though stone masonry may behave differently and experience locally larger strains than mass concrete. However, taking the masonry as a homogeneous material was considered sufficient for preliminary estimates of its behavior under seismic wave passage.

Another way to assess conduit performance is to compare the specified ground motion parameters (such as peak ground acceleration and velocity) with damage limits established from empirical correlations between these parameters and the observed performance of soil and rock tunnels and underground openings. The application of the above two methods, while extremely simplified, provided a useful basis to assess the performance of the outlet conduit.

6.3.2 Analysis Methodology

Various simplified solutions are available to simulate the effects of seismic waves on buried conduits. These simplified solutions assimilate dynamic transient stresses induced by the seismic wave passage to an equivalent-static stress field, superimposed on the pre-existing confining stresses. Analytical procedures rely on numerical formulations derived from the work of various researchers (Mow and Mente, 1963; Newmark, 1968; Pao and Mo, 1973; and others) and decouple several types of wave loading, such as induced by transversely- or longitudinally-propagating compressive, shear or Rayleigh waves. While earthquake effects result from the combination of all wave types, we reduced the uncertainty resulting from the decoupling of wave effects by taking an upper-bound approach in our calculations. In the estimation of induced stresses and strains, peak static stresses were combined with peak induced seismic stresses obtained using the most unfavorable combination of stress concentration factors and wave travel orientation, thereby resulting in a conservative assessment. These simplified analysis procedures are briefly summarized below:

Simplifying Assumptions Related to Ground Motion

- Particle velocity is the same in shear or compression (conservative),
- Peak loads occur simultaneously in the horizontal and vertical directions,
- Linear elastic medium was assumed,
- The most critical direction of propagation of wave fronts was taken, and
- Detailed calculations were only performed for the case of the 475-year probabilistic earthquake.

As induced seismic stresses in an elastic medium are proportional to the specified peak ground acceleration, conclusions can be derived for other scenarios.

Longitudinal Waves

The simplified solutions assume that the axial and shear modes of deformation of the conduit wall, hence the stresses, are the same as would exist in the absence of the conduit opening. This implies that displacements of the conduit structure and surrounding medium are assumed to be the same, a conservative assumption as it leads to possibly overestimating actual movements of the conduit walls.

Transverse Waves

The dynamic problem is reduced to an equivalent static solution by assuming that a planar wave front imposes a transient uniform stress field on the materials surrounding the conduit opening. If one further assumes a state of plane-strain, elastic solutions provide the seismic stresses around the opening. The induced seismic stresses modify the initial elastic stresses, which depend on the depth of overburden and on the shape of the opening and its wall thickness. Under assumption (2), stresses around the perimeter of the conduit were estimated through the use of stress concentration factors applicable to buried structures, as developed by Chen, Deng and Birkmyer (1979). These factors approximately take into account the influence of the assumed different rigidities of the conduit wall and surrounding sediments and clay fill.

There is a fundamental difference in the case of transverse waves, compared with the case of longitudinal waves. While no relative movements between the conduit and the surrounding materials are assumed to occur, the conduit pipe perimeter can deform in shape as a result of transverse wave passage. Newmark (1968) provided expressions to estimate strains induced by a planar front traveling at a certain angle with respect to the centerline of a long buried structure. These expressions were used to provide upper bound estimates of dynamic strains and curvatures imposed on the outlet conduit.

6.3.3 Results

Longitudinal Wave Passage

Under assumption (1), maximum induced tensile strains in the conduit were estimated to range from 0.002 percent to 0.006 percent for shear waves, and 0.03 to 0.007 percent for compression waves. These values are less than the assumed cracking limit of 0.04 percent strain, indicating satisfactory performance. Under assumption (2), shear wave tensile strains were computed to range from 0.07 to 0.35 percent, and compressive wave tensile strains ranged from 0.009 to 0.04 percent. These values indicate possible cracking, but are below the assumed crushing limit of 0.4 percent strain.

Transverse Wave Passage

Under assumption (1), computed strains remained below the cracking limit. Maximum tensile strains ranged from 0.009 to 0.03 percent and maximum compressive strains ranged from 0.01 to 0.03 percent. Under assumption (2), these ranges become 0.008 to 0.002, and 0.003 to 0.009 percent, respectively. Overall, these calculated values do not represent a concern, as they remain below the postulated cracking limit.

Empirical Considerations

As a supplement to the above analyses, the ground motion specified for the 475-year earthquake was compared with ground motions known to have caused damage in tunnels and underground facilities. Such a comparison would apply to a conduit buried in somewhat consolidated sediments or if the clay fill was well compacted, which may or may not be the case. Historically, below-ground facilities such as tunnels, pipelines and conduits have performed satisfactorily during earthquakes, if they were not directly intersected by a fault rupture or surrounded with liquefied soils. Most of the applicable literature is related to tunnels (Dowding and Rozen, 1978; Sharma and Judd, 1991; Geomatrix Consultants; 1998). Most of this information is applicable to bored tunnels, however, and does not readily apply to the Sweetwater outlet conduit. However, pipelines and buried conduits have generally performed well if they were not surrounded by soft soils and potentially liquefiable materials. For PGA's of 0.20g or less, shaking causes very little or no damage in tunnels. For PGA's between 0.20 and 0.50g, there has been limited occurrences of slight to moderate damage. However, at that level of ground motion, the most extensive damage has been related to cases of landsliding at tunnel portals (e.g. 1923 Kanto Earthquake, Japan), and therefore would be irrelevant to the outlet conduit. Relatively few instances, but some cases of severe damage in tunnels, have been reported for PGA's greater than 0.50g, which is about the PGA of the MCE (0.49g). Overall, for PGA's between 0.20g and 0.50g, no or minor damage would be expected in buried pipes or conduit structures not intersected by fault movement.

7. Conclusions

Based on analyses of the Sweetwater outlet tower reported herein, we believe that significant cracking (major structural damage) of the tower would probably occur under several of the postulated earthquake loads. In addition, the stability of the tower against global overturning could not be demonstrated. The tower would likely collapse under moment loading from an earthquake with a 475-year return period, or under maximum earthquakes occurring along the La Nacion or Rose Canyon faults, due to masonry overstressing resulting from large overturning moments. However, the La Nacion or Rose Canyon events have a very low probability of occurrence, due to the low slip rates of these two faults. Based on a parametric analysis, we concluded that the tower is likely to survive a ground motion at the site having a peak ground acceleration (PGA) up to 0.11g. The return period of an earthquake that can cause a PGA of 0.11 at the site was estimated to be 144 years. For ground motions with return periods between 144 and 475 years, various degrees of cracking, or partial or total failure, could conceivably occur. The tower appears to be capable of resisting maximum earthquakes generated by more distant faults such as the San Miguel-Vallecitos, San Diego Trough and Elsinore faults, and perhaps the Agua Blanca-Coronado fault. Such faults are the most active in the greater San Diego area.

It is possible that the tower could sustain significant cracking, and still maintain its stability. However, uncertainties in the way loads would be redistributed after the onset of cracking make assessment of post-cracking behavior of the masonry tower virtually impossible to predict, especially since the tower contains no reinforcing steel.

The response of the outlet conduit to seismic wave passage was evaluated based on simplified analyses and empirical considerations. No significant damage other than some cracking of the conduit wall would be expected under the 475-year earthquake or lesser ground motions. The parameters used in the analyses were conservative, meaning that the actual performance of the conduit would probably be better than that obtained from the analyses. Overall, the conduit is considerably less vulnerable to earthquake motion than the tower itself.

8. References

American Concrete Institute (1995). **Building Code Requirements for Structural Concrete**, Standard ACI 318-95 and Commentary (ACI 318R-95).

Artim, E.R.; Elder, D.L. (1979). **Late Quaternary Deformation along the La Nacion Fault System, San Diego, California**, Geological Society of America, Abstracts with Prog., Vol. 11, p. 381.

Anderson, J.G., Rockwell, T.K., and Agnew, D.C. (1989). **Past and Possible Future Earthquakes of Significance to the San Diego Region**, Earthquake Spectra, Vol. 5, No. 2. pp 299-335.

Artim, E.R.; Pinckney, C.J. (1973). **La Nacion Fault System, San Diego**, Geological Society of America Bulletin, Vol. 84, pp. 1075-1080.

Bureau, G. (1985). **Seismic Safety Analysis and Rehabilitation of Dam Inlet/Outlet Structures**, XVth International Congress on Large Dams, Lausanne, Switzerland, Proceedings, Q.59, R.17, pp. 297-319.

Bureau G. (1993). **Seismic Safety of Intake/Outlet Towers**, International Workshop on Dam Safety Evaluation, Grindelwald, Switzerland, 26-28 April, Proceedings, Vol. 2, pp. 229-240.

Bureau, G. (1995). **Intake/Outlet Towers**, Section 5 in USCOLD *Guidelines for Earthquake Design and Evaluation of Structures Appurtenant to Dams,* USCOLD Committee on Earthquakes, pp. 41-62.

Bureau, G.; Scawthorn, C. (1986). **Seismic Reevaluation of Lower Crystal Springs Outlet System** in Seismic Evaluation of Lifeline Systems - Case Studies, ASCE Publisher, October, pp. 16-27.

Chen, P.C.; Deng, D.Z.F.; Birkmyer, A.J. (1970). **Considerations of Dynamic Stress Concentrations in the Seismic Analysis of Buried Structures,** 2nd U.S. National Conference on Earthquake Engineering, Stanford, CA, Proceedings.

Clark, M.M. (1982). **Map Showing Recently Active Breaks Along the Elsinore and Associated Faults between Lake Henshaw and Mexico**, USGS Map I-1329, 1:24,000

Clark, M.M.; Harms, K.K.; Lienkaemper, J.J.; Harwood, D.S.; Lajoie, K.R.; Matti, J.C.; Perkins, J.A.; Rymer, N.J.; Sarna- Wojcicki, A.M.; Sharp, R.V.; Sims, J.D.; Tinsley, J.C., III; and Ziony, J.I.; (1984). **Preliminary Slip-Rate Table and Map of Late-Quaternary Faults of California**, U.S. Geological Survey Open-File Report 84-106, p. 12, pl. 1-5.

Construction Specifications (1939). **Constructing Unit No. 1 of the Enlargement of Sweetwater Dam and Spillways**, November, in Sweetwater Authority files.

Dames & Moore (1994). **Final Report, Control of Water Surface at Sweetwater Dam, Spring Valley, CA**, Sweetwater Authority File B.P. 93-41E, Report to Sweetwater Authority, September.

Dames & Moore (1995). **Seismic Analysis of Various Steel Tanks**, Report to Sweetwater Authority, Chula Vista, CA, April.

Deere, D.U.; Miller, R.P. (1966). **Engineering Classification and Index Properties for Intact Rock**, Figure 6-20.

Dowding, C.; Rozen, A. (1978). **Damage to Rock Tunnels from Earthquake Shaking**, ASCE Journal of the GT Division, Vol. 104, No. GT2, February.

Dungar, R. (1981). **El Cajon Hydroelectric Power Plan, Arch Dam Final Design, Static and Dynamic Stress Analysis**, Report by Motor Columbus Consulting Engineers, Baden, May, p. 25.

Elliot, W.J. and Hart, M.W. (1977). **New Evidence Concerning Age of Movement of the La Nacion Fault, Southwestern San Diego County**, in *Geology of Southwestern San Diego County, California and Northwestern Baja California*, San Diego Association of Geologists Guidebook, Farrand G.T. Editor, pp. 53-60.

Erikson, D.L. (1996). **Seismic Evaluation of the Blue River Reservoir Intake Tower**, Proceedings, USCOLD Annual Meeting, Los Angeles, CA, July, pp. 155-173

Geomatrix Consultants (1998). **Summary and Evaluation of Procedures for the Seismic Design of Tunnels**, By Power, M.S., et al., Technical Report to MCEER, FHWA Contract No. DTFH61-92-C-00112, September 9, Rev. 1.

Goyal, A.; Chopra, A.K. (1989). **Earthquake Analysis and Response of Intake-Outlet Towers**, University of California, Earthquake Engineering Research Center, Report UBC/EERC-89/04, 427 pp.

International Conference of Buildings Officials (1997). **Uniform Building Code** [UBC], Volumes 1 to 3.

Kennedy, M.P.; Clarke, S.H., Greene, H.G.; Legg, M.R. (1980). **Recency and Character of Faulting, Offshore Metropolitan San Diego, California**, California Division of Mines and Geology, Map Sheet No. 42.

Kennedy, M.P.; and others (1975). **Recency and Character of Faulting, San Diego Metropolitan Area, California**, CDMG Special report No. 123, 33 pp.

Kennedy, M.P.; and others (1979). **Recency and Character of Faulting Offshore from Metropolitan San Diego, California**, USGS, Menlo Park, CA.

Lamar, D.L.; et al (1975). **Application of Skylab and ERTS Imagery to Fault Tectonics and Earthquake Hazards of Peninsular Ranges, Southwestern California**, Earth Sciences Corporation, Technical Report 75-2, Santa Monica, California

Lamar, D.L.; Swanson, S.C. (1981). **Study of Seismic Activity by Selective Trenching along the Elsinore Fault Zone, Southern California**, USGS Open File Report 81-0882

Legg, M.R. (1985). **Geological Structure and Tectonics of the Inner Continental Borderland Offshore Northern Baja California Mexico**, Ph.D. Thesis, University of California, Santa Barbara.

Mow, C.C.; Mente, L.J. (1963). **Dynamic Stresses and Displacements Around Cylindrical Discontinuities Due to Planar Harmonic Shear Waves**, Journal of Applied Mechanics, Transactions ASME, 30(4):598, December, pp. 598-604.

Newmark, N. M.; Hall, W. J. (1982). in **Earthquake Spectra and Design**, Earthquake Engineering Research Institute (EERI), Monograph Series, Vol. 3, 103 pp.

Newmark (1968). **Problems in Wave Propagation in Soil and Rock**, International Symposium on Wave Propagation and Dynamic Properties of Earth Materials, August 23035, U. of New Mexico, Albuquerque, Proceedings, pp. 7-26.

Noret, C.; Da Rin, E.M.; Modaressi, A.; Carrère, A. (1998). **Seismic Vulnerability Assessment of Masonry Gravity Dams**, DG12 Workshop on *Reducing Earthquake Risks to Structures and Monuments in the European Union*, Cambridge, Great Britain, Nov. 26-27, 13 pp.

Pao, Y.H.; Mow, C.C. (1973). **Diffraction of Elastic Waves and Dynamic Stress Concentrations**, Rand Corporation research Study, Crane-Russak, Publisher, 694 pp.

Raphael, J.M. (1984). **Tensile Strength of Concrete**, ACI Journal, Technical Paper, March-April Issue, pp. 158-165.

Rockwell, T.K. (1989). **Behavior of Individual Fault Segments along the Elsinore-Laguna Salada Fault Zone and Northern Baja California: Implications for the Characteristic Earthquake Model in Fault Segmentation and Control of Rupture Initiation and Termination**, USGS Open File Report 89-315.

Rockwell, T.K., Lindvall, S.C., Haraden, C.C., Hibayashi, C.K., and Baker, E., (1989). **Minimum Holocene Slip Rate for the Rose Canyon Fault in San Diego, California** in Abbott, P.L. and Elliot, W.J., Eds., "Environmental Perils, San Diego Region," published for the Geological Society of American Annual Meeting by the San Diego Association of Geologists.

Rockwell, T.K.; Hatch, M.E.; Schug, D.L. (1987). **Late Quaternary Rates: Agua Blanca and Borderland Faults**, Final Technical Report, U.S. Geol. Survey, 122 pp.

Rockwell, T.K.; Loughman, C.; Merrifield, T. (1989). **Late Quaternary rate of Slip Along the San Jacinto Fault Zone, Southern California**, Journal Geophys. Research.

Sadigh, K.R. (1989). **Attenuation Relationships for Horizontal Peak Ground Acceleration and Response Spectral Accelerations for Rock Sites**, Seismological Research Letters, Vol. 60, No. 1, January- March, p. 19

SAP2000 (1999). Version 7.10, computers and Structures, Inc., Berkeley, California.

Seismological Research Letters, Seismological Society of America [SSA] (1989). Vol. 60, No. 1, January-March, papers by Abrahamson and Silva; Boore, et al.; Sadigh and Campbell.

Sharma, S.; Judd, W.R. (1991). **Underground Opening Damage from Earthquakes**, Engineering Geology, Volume 30.

Tynes, W.O. (1963). **Investigation of Methods of Preparing Horizontal Construction Joints in Concrete**, Technical Report No. 6-518, Report 2, *Test of Joints in Large Blocks*, U.S. Army Engineer Waterways Experimental Station, Vicksburg, Miss., August.

Tynes, W.O. (1959). **Investigation of Methods of Preparing Horizontal Construction Joints in Concrete**, Technical Report No. 6-518, Report 1, US Army Engineer Waterways Experimental Station, Vicksburg, Miss., July.

United States Committee on Large Dams (1985). **Guidelines for Selecting Seismic Parameters for Dam Projects**, USCOLD Publication, Committee on Earthquakes, October, 39 pp.

United States Committee on Large Dams [USCOLD] (1999). **Updated Guidelines for Selecting Seismic Parameters for Dam Projects**, USCOLD Publication, Committee on Earthquakes, April, 63 pp.

United States Committee on Large Dams [USCOLD] (1995). **Guidelines for Earthquake Design and Evaluation of Structures Appurtenant to Dams**, USCOLD Publication, Committee on Earthquakes, May, 75 pp.

United States Committee on Large Dams [USCOLD] (1988). **Development of Dam Engineering in the United States**, Published for XVIth ICOLD Congress, San Francisco, USA, *"No. 3-1 Sweetwater Dam,"* Kollgaard, Chadwick, Editors, Pergamon Press, pp. 319-330.

URS (2001). **Safety Inspection Report, Sweetwater Dam**, Report to Sweetwater Authority, Sweetwater Authority File B.P. 99-36E, February.

Waters, T. (1954). **A Study of the Tensile Strength of Concrete Across Construction Joints**, Magazine of Concrete Research, Cement and Concrete Association, London, England, Vol. 6, No. 18, Dec., pp. 151-153.

Woodward-Clyde Consultants (1975). **Study of the Behavior and Stability of the Sweetwater Arch Dam, San Diego County, CA**.

Yerkes, R.F. (1972). **Geology and Oil Resources of the Western Puente Hills Area, Southern California**, USGS Professional Paper 420-C, p. 63

Tables

Table 1

Schmidt Hammer Rebound Measurements

SWEETWATER MAIN DAM OUTLET TOWER, CA

MEASUREMENTS AT VARIOUS TOWER AND DAM ABUTMENT LOCATIONS

(Random locations)

Test Number	Material Tested	Hammer Position	Rebound Value (R)	Compressive Strength (psi)	Comments
1	Concrete	Horizontal	50	7,416	Side of top platform
2	Concrete	Vertical	49	7,889	Top of operating platform
3	Concrete	Vertical	48	7,667	Top of operating platform
4	Concrete	Vertical	46	7,222	Top of operating platform
5	Concrete	Vertical	52	8,556	Top of operating platform
6	Concrete	Vertical	45	7,000	Top of operating platform
7	Concrete	Vertical	44	6,778	Top of operating platform
8	Concrete	Horizontal	42	5,683	Side of tower, from boat
9	Concrete	Horizontal	48	6,983	Side of tower, from boat
10	Concrete	Horizontal	50	7,416	Side of tower, from boat
11	Concrete	Horizontal	46	6,550	Side of tower, from boat
12	Mortar	Horizontal	30	3,083	At dam left abutment
13	Mortar	Horizontal	32	3,516	At dam left abutment
14	Mortar	Horizontal	33	3,733	At dam left abutment
15	Mortar	Horizontal	44	6,116	At dam left abutment
16	Mortar	Horizontal	30	3,083	At dam left abutment
17	Mortar	Horizontal	29	2,866	At dam left abutment
18	Mortar	Horizontal	30	3,083	At dam left abutment
19	Mortar	Horizontal	36	4,383	At dam left abutment
20	Mortar	Horizontal	38	4,816	At dam left abutment
21	Mortar	Horizontal	40	5,250	At dam left abutment
22	Mortar	Horizontal	31	3,300	At dam left abutment
23	Mortar	Horizontal	33	3,733	At dam left abutment
24 [4]	Stone	Horizontal	62	35,000 *	At dam left abutment
25 [4]	Stone	Horizontal	62	35,000 *	At dam left abutment
26 [4]	Stone	Horizontal	60	31,000 *	At dam left abutment
27 [4]	Stone	Horizontal	65	45,000 *	At dam left abutment
28 [4]	Stone	Horizontal	70	59,000 *	At dam left abutment
29 [4]	Stone	Horizontal	70	59,000 *	At dam left abutment
30 [4]	Stone	Horizontal	65	45,000 *	At dam left abutment
31 [4]	Stone	Horizontal	67	47,000 *	At dam left abutment
32 [4]	Stone	Horizontal	70	59,000 *	At dam left abutment
33 [4]	Stone	Horizontal	70	59,000 *	At dam left abutment

NOTES

[1] All surfaces were grounded smooth prior to measurements. Uncorrected N-type rebounds are listed.
[2] Masonry stone and mortar were tested at dam left abutment, as masonry is not visible at tower.
[3] Dynamic strength is measured, as it is based on instantaneous hammer rebound.
[4] Different correlations between rebound and compressive strength are used for stone or concrete.

f'c CONCRETE:	**7,196**	**psi**	**STD. DEV.**	**714**	**psi**
f'c MASONRY MORTA	**3,914**	**psi**	**STD. DEV.**	**976**	**psi**
f'c MASONRY STONE:	**47,400**	**psi ***	**STD. DEV.**	**10,613**	**psi**

* Corrected from N- to L-type hammer and use of Deere and Miller (1966) rock R-f'c (L-type) correlation.

SCHMIDT.wq!

Table 2

Deterministic Peak Ground Acceleration Estimates

SWEETWATER MAIN DAM OUTLET TOWER, CA

Fault Name	M_{max}	Distance (km)	Peak Ground Acceleration, (g)			
			50th Percentile		84th Percentile	
			Horizontal	Vertical	Horizontal	Vertical
La Nacion	6.7	4	0.49	0.51	0.77	0.90
Rose Canyon	7.0	13	0.28	0.24	0.43	0.41
Agua Blanca-Coronado	7.2	28	0.15	0.11	0.24	0.20
San Miguel-Vallecitos	7.0	43	0.09	0.06	0.14	0.10
San Diego Trough	7.2	50	0.08	0.05	0.13	0.10
Elsinore-Laguna Salada	7.5	60	0.08	0.05	0.13	0.10

Mmax = Earthquake magnitude
50th Percentile = mean value
84th Percentile = mean-plus-one standard deviation

Table 3
Deterministic Horizontal Response Spectra

FAULTS NAME:

[1] LA NACION	**M = 6.7 [Mw]**	**d = 4 km**
[2] ROSE CANYON	**M = 7.0 [Mw]**	**d = 13 km**
[3] AGUA BLANCA - CORONADO BANK	**M = 7.2 [Mw]**	**d = 28 km**
[4] SAN MIGUEL - VALLECITOS	**M = 7.0 [Mw]**	**d = 43 km**
[5] SAN DIEGO TROUGH	**M = 7.2 [Mw]**	**d = 50 km**
[6] ELSINORE - LAGUNA SALADA	**M = 7.5 [Mw]**	**d = 60 km**

COMPONENT: **HORIZONTAL (Rock)** PERCENTILE: **50th** **Average of 4**

T (sec)	[1] La Nacion	[2] Rose Canyon	[3] Coronado	[4] San Miguel	[5] San Diego	[6] Elsinore
0.010	0.49	0.28	0.15	0.09	0.08	0.08
0.020	0.49	0.28	0.15	0.09	0.08	0.08
0.030	0.55	0.31	0.17	0.09	0.09	0.09
0.040	0.62	0.35	0.19	0.10	0.10	0.09
0.050	0.69	0.38	0.20	0.11	0.10	0.10
0.060	0.75	0.41	0.22	0.12	0.11	0.11
0.075	0.83	0.45	0.23	0.13	0.12	0.11
0.090	0.91	0.49	0.26	0.14	0.13	0.12
0.100	0.96	0.52	0.27	0.15	0.14	0.13
0.120	1.02	0.56	0.29	0.16	0.15	0.14
0.150	1.08	0.60	0.31	0.17	0.16	0.15
0.170	1.09	0.61	0.32	0.18	0.17	0.16
0.200	1.09	0.61	0.33	0.18	0.17	0.16
0.240	1.04	0.59	0.32	0.18	0.17	0.16
0.300	0.96	0.55	0.30	0.17	0.16	0.16
0.360	0.86	0.50	0.28	0.16	0.15	0.15
0.400	0.81	0.47	0.27	0.15	0.15	0.15
0.460	0.73	0.43	0.25	0.14	0.14	0.14
0.500	0.68	0.40	0.23	0.13	0.13	0.13
0.600	0.58	0.35	0.21	0.11	0.11	0.12
0.750	0.47	0.28	0.17	0.10	0.10	0.10
0.850	0.41	0.25	0.15	0.09	0.09	0.09
1.000	0.34	0.21	0.13	0.07	0.08	0.08
1.500	0.21	0.14	0.09	0.05	0.05	0.06
2.000	0.14	0.10	0.07	0.04	0.04	0.04
3.000	0.08	0.06	0.04	0.02	0.02	0.03
4.000	0.05	0.03	0.02	0.01	0.01	0.02
5.000	0.03	0.02	0.02	0.01	0.01	0.01

Note: geometric average is used

Table 4
Deterministic Vertical Response Spectra

FAULTS NAME:

[1] LA NACION	**M = 6.7 [Mw]**	**d = 4 km**
[2] ROSE CANYON	**M = 7.0 [Mw]**	**d = 13 km**
[3] AGUA BLANCA - CORONADO BANK	**M = 7.2 [Mw]**	**d = 28 km**
[4] SAN MIGUEL - VALLECITOS	**M = 7.0 [Mw]**	**d = 43 km**
[5] SAN DIEGO TROUGH	**M = 7.2 [Mw]**	**d = 50 km**
[6] ELSINORE - LAGUNA SALADA	**M = 7.5 [Mw]**	**d = 60 km**

COMPONENT: **VERTICAL (Rock)** PERCENTILE: **50th** **Average of 4**

T (sec)	[1] La Nacion	[2] Rose Canyon	[3] Coronado	[4] San Miguel	[5] San Diego	[6] Elsinore
0.010	0.51	0.24	0.11	0.06	0.05	0.05
0.020	0.51	0.24	0.11	0.06	0.05	0.05
0.030	0.60	0.27	0.12	0.06	0.06	0.06
0.040	0.80	0.35	0.15	0.08	0.07	0.07
0.050	1.00	0.43	0.19	0.09	0.09	0.08
0.060	1.13	0.49	0.22	0.10	0.10	0.09
0.075	1.27	0.55	0.24	0.12	0.11	0.10
0.090	1.28	0.55	0.24	0.12	0.11	0.10
0.100	1.28	0.55	0.24	0.11	0.11	0.10
0.120	1.18	0.51	0.23	0.11	0.10	0.10
0.150	1.05	0.47	0.21	0.11	0.10	0.10
0.170	0.94	0.43	0.20	0.10	0.09	0.09
0.200	0.80	0.37	0.18	0.09	0.08	0.08
0.240	0.65	0.31	0.15	0.08	0.07	0.07
0.300	0.50	0.24	0.12	0.07	0.06	0.06
0.360	0.41	0.20	0.10	0.05	0.05	0.05
0.400	0.36	0.18	0.09	0.05	0.05	0.05
0.460	0.30	0.15	0.08	0.04	0.04	0.04
0.500	0.28	0.14	0.07	0.04	0.04	0.04
0.600	0.23	0.12	0.07	0.04	0.04	0.04
0.750	0.19	0.10	0.06	0.03	0.03	0.03
0.850	0.17	0.09	0.05	0.03	0.03	0.03
1.000	0.15	0.08	0.04	0.02	0.02	0.03
1.500	0.11	0.06	0.03	0.02	0.02	0.02
2.000	0.07	0.04	0.02	0.01	0.01	0.01
3.000	0.05	0.03	0.02	0.01	0.01	0.01
4.000	0.03	0.02	0.01	0.01	0.01	0.01
5.000	0.02	0.01	0.01	0.00	0.00	0.01

Note: geometric average is used

Table 5

USGS Probabilistic Ground Motion Estimates

SWEETWATER MAIN DAM OUTLET TOWER, CA

Probability of occurrence in 50 years	Return Period (years)	Peak Ground Acceleration (g)	0.2 Second Spectral Acceleration (g)	0.3 Second Spectral Acceleration (g)	1.0 Second Spectral Acceleration (g)
10%	475	0.21	0.49	0.44	0.18
5%	975	0.28	0.64	0.61	0.25
2%	2,475	0.38	1.06	0.99	0.36

Table 6

Analysis Properties for Masonry Mortar and Stone Masonry

SWEETWATER MAIN DAM OUTLET TOWER, CA

Analysis Parameter	Average - sigma	Average	Average + sigma
Masonry Mortar Only			
Sustained Static Compressive Strength (psi)	**2,417**	**3,250**	**4,083**
Dynamic Compressive Strength (psi)	2,900	3,900	4,900
Dynamic Tensile Strength (psi)	338	455	572
Dynamic Shear Strength (psi)	128	148	166
Equivalent Horizontal Joint Between Stone and Masonry			
Dynamic Tensile Strength (psi)	245	330	415
Dynamic Shear Strength (psi)	93	107	120
Stone Masonry			
Dynamic Modulus of Elasticity (million psi)	1.25	5.0	8.75
Poisson's ratio	0.12	0.15	0.18

Table 7
Nodal Point Coordinates

Sweetwater Main Dam Outlet Tower
Sweetwater Authority, Chula Vista, CA

Node	X (ft)	Y (ft)	Z (ft)	Z (in)	DZ (in)
21	0.000	0.000	239.30	2,871.6	
					42.6
20	0.000	0.000	235.75	2,829.0	
					108.0
19	0.000	0.000	226.75	2,721.0	
					108.0
18	0.000	0.000	217.75	2,613.0	
					30.0
17	0.000	0.000	215.25	2,583.0	
					30.0
16	0.000	0.000	212.75	2,553.0	
					60.0
15	0.000	0.000	207.75	2,493.0	
					60.0
14	0.000	0.000	202.75	2,433.0	
					60.0
13	0.000	0.000	197.75	2,373.0	
					60.0
12	0.000	0.000	192.75	2,313.0	
					60.0
11	0.000	0.000	187.75	2,253.0	
					60.0
10	0.000	0.000	182.75	2,193.0	
					60.0
9	0.000	0.000	177.75	2,133.0	
					60.0
8	0.000	0.000	172.75	2,073.0	
					60.0
7	0.000	0.000	167.75	2,013.0	
					60.0
6	0.000	0.000	162.75	1,953.0	
					46.5
5	0.000	0.000	158.88	1,906.5	
					46.5
4	0.000	0.000	155.00	1,860.0	
					58.2
3	0.000	0.000	150.15	1,801.8	
					58.2
2	0.000	0.000	145.30	1,743.6	
					75.6
1	0.000	0.000	139.00	1,668.0	

Table 8

Cross-Section Areas

Sweetwater Main Dam Outlet Tower
Sweetwater Authority, Chula Vista, CA

NODE / SECTION NUMBER	NODE Z (ft)	NODE Z (in)	SECTION Z (in)	SECTION Ri (in) [Note 1]	SECTION Ro (in) [Note 1]	A-GROSS (Ag) (in**2)	A-SHEAR (As) (in**2)
21	239.30	2,871.6					
20			2,850.3	31.2	80.4	13,575	11,810
20	235.75	2,829.0					
19			2,775.0	31.2	80.4	13,575	11,810
19	226.75	2,721.0					
18			2,667.0	31.2	80.4	13,575	11,810
18	217.75	2,613.0					
17			2,598.0	31.2	80.4	13,723	11,939
17	215.25	2,583.0					
16			2,568.0	31.2	80.4	13,723	11,939
16	212.75	2,553.0					
15			2,523.0	31.2	80.4	13,954	12,140
15	207.75	2,493.0					
14			2,463.0	31.2	80.4	13,954	12,140
14	202.75	2,433.0					
13			2,403.0	31.2	80.4	14,265	12,411
13	197.75	2,373.0					
12			2,343.0	31.2	80.4	14,265	12,411
12	192.75	2,313.0					
11			2,283.0	31.2	80.4	14,265	12,411
11	187.75	2,253.0					
10			2,223.0	31.2	80.4	14,265	12,411
10	182.75	2,193.0					
9			2,163.0	31.2	80.4	14,265	12,411
9	177.75	2,133.0					
8			2,103.0	31.2	80.4	14,265	12,411
8	172.75	2,073.0					
7			2,043.0	31.2	80.4	14,265	12,411
7	167.75	2,013.0					
6			1,983.0	31.2	80.4	14,265	12,411
6	162.75	1,953.0					
5			1,929.8	31.2	80.4	14,265	12,411
5	158.88	1,906.5					
4			1,883.3	31.2	80.4	14,265	12,411
4	155.00	1,860.0					
3			1,830.9	31.2	80.4	18,945	16,482
3	150.15	1,801.8					
2			1,772.7	31.2	80.4	13,185	11,471
2	145.30	1,743.6					
1			1,705.8	31.2	80.4	21,474	18,682
1 [fixed]	139.00	1,668.0					

Note 1: Ri and Ro designate maximum hexagonal radii.
Note 2: For tower bottom, Ro is minimum distance from hexagon center to side of equivalent section.
Note 3: For sections 14 to 20, areas were reduced for encroachment of valves 7, 8 and 9.

Table 9

Sections Moments of Inertia

Sweetwater Main Dam Outlet Tower
Sweetwater Authority, Chula Vista, CA

NODE / SECTION NUMBER	Z (ft)	Z (in)	Z (ft)	Ri (in)	Ro (in)	Ixx (in**4)	Iyy (in**4)	J (in**4)
21	239.30	2871.6						
20			237.53	31.2	80.4	2.136E+07	2.137E+07	4.273E+07
20	235.75	2829.0						
19			231.25	31.2	80.4	2.136E+07	2.137E+07	4.273E+07
19	226.75	2721.0						
18			222.25	31.2	80.4	2.136E+07	2.137E+07	4.273E+07
18	217.75	2613.0						
17			216.50	31.2	80.4	2.136E+07	2.137E+07	4.274E+07
17	215.25	2583.0						
16			214.00	31.2	80.4	2.136E+07	2.137E+07	4.274E+07
16	212.75	2553.0						
15			210.25	31.2	80.4	2.168E+07	2.169E+07	4.337E+07
15	207.75	2493.0						
14			205.25	31.2	80.4	2.168E+07	2.169E+07	4.337E+07
14	202.75	2433.0						
13			200.25	31.2	80.4	2.208E+07	2.209E+07	4.416E+07
13	197.75	2373.0						
12			195.25	31.2	80.4	2.208E+07	2.209E+07	4.416E+07
12	192.75	2313.0						
11			190.25	31.2	80.4	2.208E+07	2.209E+07	4.416E+07
11	187.75	2253.0						
10			185.25	31.2	80.4	2.208E+07	2.209E+07	4.416E+07
10	182.75	2193.0						
9			180.25	31.2	80.4	2.208E+07	2.209E+07	4.416E+07
9	177.75	2133.0						
8			175.25	31.2	80.4	2.208E+07	2.209E+07	4.416E+07
8	172.75	2073.0						
7			170.25	31.2	80.4	2.208E+07	2.209E+07	4.416E+07
7	167.75	2013.0						
6			165.25	31.2	80.4	2.208E+07	2.209E+07	4.416E+07
6	162.75	1953.0						
5			160.81	31.2	80.4	2.208E+07	2.209E+07	4.416E+07
5	158.88	1906.5						
4			156.94	31.2	80.4	2.208E+07	2.209E+07	4.416E+07
4	155.00	1860.0						
3			152.58	31.2	80.4	3.157E+07	7.413E+07	1.057E+08
3	150.15	1801.8						
2			147.73	31.2	80.4	2.557E+07	3.778E+07	6.335E+07
2	145.30	1743.6						
1			142.15	0.0	80.4	3.208E+07	7.466E+07	1.062E+08
1 [fixed]	139.00	1668.0						

Note 1: Ri and Ro designate maximum hexagonal radii.
Note 2: For all elements, averaged properties are being used and include adjustment for openings.
Note 3: Inertias adiusted for valve encroachment of valves 7. 8. 9.

Table 10
Nodal Point Masses

Sweetwater Main Dam Outlet Tower
Sweetwater Authority, Chula Vista, CA

NODE NUMBER	SECTION NUMBER	NP MASS [dry] (lbs.s2/in)	OUTSIDE WATER MASS	INSIDE WATER MASS	NP MASS [w/water] (lbs.s2/in)
21		179.91	0.00	0.00	179.91
	20				
20		266.67	20.51	15.45	302.62
	19				
19		369.18	83.08	15.45	467.71
	18				
18		243.65	26.50	4.29	274.44
	17				
17		103.67	26.64	4.29	134.60
	16				
16		164.47	53.56	8.58	226.62
	15				
15		210.82	92.57	14.67	318.07
	14				
14		220.40	109.41	17.16	346.97
	13				
13		215.53	110.54	17.16	343.23
	12				
12		222.75	111.00	17.16	350.91
	11				
11		215.53	111.23	17.16	343.92
	10				
10		222.75	111.46	17.16	351.37
	9				
9		215.53	111.68	17.16	344.37
	8				
8		222.75	112.03	17.16	351.94
	7				
7		215.53	112.25	17.16	344.94
	6				
6		198.50	99.83	15.23	313.56
	5				
5		167.03	87.44	13.30	267.77
	4				
4		222.34	98.74	14.97	336.05
	3				
3		235.44	109.88	16.65	361.97
	2				
2		301.02	126.43	19.14	446.59
	1				
1		fixed	fixed	fixed	0.00

Note 1: Tower inside is assumed to be dewatered during postulated occurrence of earthquake.
Note 2: Node masses have been adjusted to account for presence of valve openings, saucer valves, footbridge, operating platform and equipment.

Table 11
Principal Modes of Vibration

Sweetwater Main Dam Outlet Tower
Sweetwater Authority, Chula Vista, CA

with inside of tower full of water

RESERVOIR ELEVATION: 237.00 feet

CASE Number	MASONRY f'c (psi)	Mode 1 (Hz) Y, X	Mode 2 (Hz) Y, X	Mode 3 (Hz)	Mode 4 (Hz) Y, X
===>	*VALUES BELOW ARE FOR UNCRACKED TOWER SHAFT [Gross section properties were used]*				
1	2,417	0.96, 1.08	5.22, 5.67	12.40	13.54, 14.27
2	3,250	1.95, 2.20	10.62, 11.53	25.32	27.45, 28.91
3	4,083	2.59, 2.92	14.07, 15.27	33.59	36.33,38.24

Note :
Mode 1 = First Bending Mode of Vibration (Y and X)
Mode 2 = Second Bending Mode of Vibration (Y and X)
Mode 3 = First Vertical Mode of Vibration (Z)
Mode 4 = Third Bending Mode of Vibration (Y and X)

Table 12
Gross Section Moment Capacity
Intact Masonry Mortar

Sweetwater Main Dam Outlet Tower
Sweetwater Authority, Chula Vista, CA
475-year earthquake

PGA (H)	0.21	g
PGA (V) =	0.15	g

Rw = 1.0

Strength reduction factor for bending: **0.90**
The dynamic tensile strength is considered
Calculations include vertical [static - dynamic] loads
% of peak axial dynamic load used **75**

NODE NUMBER	SECTION NUMBER	Ro (in)	Igross (in**4)	INTACT MASONRY MORTAR FACTORED GROSS MOMENT CAPACITY [in Kips.in x 1,000]			
				f'c >>	2,417	3,250	4,083
				ft >>	338	455	572
21							
	20	80.4	2.14E+07		81.7	109.7	137.7
20							
	19	80.4	2.14E+07		83.7	111.9	140.0
19							
	18	80.4	2.14E+07		85.8	114.1	142.2
18							
	17	80.4	2.14E+07		86.3	114.7	142.8
17							
	16	80.4	2.14E+07		86.9	115.3	143.4
16							
	15	80.4	2.17E+07		89.2	118.1	146.7
15							
	14	80.4	2.17E+07		90.3	119.3	148.0
14							
	13	80.4	2.21E+07		93.0	122.6	151.8
13							
	12	80.4	2.21E+07		94.2	123.9	153.1
12							
	11	80.4	2.21E+07		95.3	125.1	154.4
11							
	10	80.4	2.21E+07		96.5	126.4	155.6
10							
	9	80.4	2.21E+07		97.7	127.6	156.9
9							
	8	80.4	2.21E+07		98.9	128.9	158.2
8							
	7	80.4	2.21E+07		100.0	130.2	159.5
7							
	6	80.4	2.21E+07		101.2	131.4	160.8
6							
	5	80.4	2.21E+07		102.1	132.4	161.8
5							
	4	80.4	2.21E+07		103.0	133.4	162.8
4							
	3	80.4	7.41E+07		333.5	434.4	532.9
3							
	2	80.4	3.78E+07		183.7	236.2	286.7
2							
	1	80.4	7.47E+07		336.9	438.7	537.9
1							

Note: Ro designate distance to extreme fiber for hexagonal section

Table 13
Gross Section Moment Capacity
Stone-Mortar Joints

Sweetwater Main Dam Outlet Tower
Sweetwater Authority, Chula Vista, CA
475-year earthquake

PGA (H)	0.21	g
PGA (V) =	0.15	g

Rw = 1.0

Strength reduction factor for bending: **0.90**
The dynamic modulus of rupture is being considered
Calculations include vertical [static - dynamic] loads
% of peak axial dynamic load used 75
Lift joint strength reduction factor: 0.725

NODE NUMBER	SECTION NUMBER	Ro (in)	Igross (in**4)	f'c >> / ftj >>	MORTAR-STONE JOINTS FACTORED GROSS MOMENT CAPACITY [in Kips.in x 1,000] 2,417 / 245	3,250 / 330	4,083 / 415
21							
	20	80.4	2.14E+07		59.4	79.8	100.1
20							
	19	80.4	2.14E+07		61.5	82.0	102.3
19							
	18	80.4	2.14E+07		63.5	84.2	104.6
18							
	17	80.4	2.14E+07		64.0	84.7	105.2
17							
	16	80.4	2.14E+07		64.6	85.3	105.8
16							
	15	80.4	2.17E+07		66.6	87.7	108.5
15							
	14	80.4	2.17E+07		67.8	89.0	109.8
14							
	13	80.4	2.21E+07		70.0	91.7	112.9
13							
	12	80.4	2.21E+07		71.2	92.9	114.2
12							
	11	80.4	2.21E+07		72.4	94.2	115.5
11							
	10	80.4	2.21E+07		73.5	95.4	116.8
10							
	9	80.4	2.21E+07		74.7	96.7	118.0
9							
	8	80.4	2.21E+07		75.9	98.0	119.3
8							
	7	80.4	2.21E+07		77.0	99.2	120.6
7							
	6	80.4	2.21E+07		78.2	100.5	121.9
6							
	5	80.4	2.21E+07		79.1	101.5	122.9
5							
	4	80.4	2.21E+07		80.0	102.4	123.9
4							
	3	80.4	7.41E+07		256.3	330.6	402.4
3							
	2	80.4	3.78E+07		144.4	183.2	220.2
2							
	1	80.4	7.47E+07		259.2	334.1	406.4
1							

Note: Ro designate distance to extreme fiber for hexagonal section

Table 14
Gross Section - Shear Capacity
Intact Masonry Mortar

Sweetwater Main Dam Outlet Tower
Sweetwater Authority, Chula Vista, CA
475-year earthquake

PGA (H)	0.21	g
PGA (V) =	0.15	g

Strength reduction factor for shear = **0.85**
Small capacity increase from vertical load is ignored
Stress increase factor for shape = 1.855

Rw = 1.0

NODE NUMBER	ELEV. (ft)	SECTION NUMBER	Shear Area (in**2)	INTACT MASONRY MORTAR FACTORED GROSS CAPACITY [in Kips]			
				f'c >>	2,417	3,250	4,083
				vc >>	128	148	166
21	239.30						
		20	11810		692	802	899
20	235.75						
		19	11810		692	802	899
19	226.75						
		18	11810		692	802	899
18	217.75						
		17	11939		699	811	909
17	215.25						
		16	11939		699	811	909
16	212.75						
		15	12140		711	825	924
15	207.75						
		14	12140		711	825	924
14	202.75						
		13	12411		727	843	945
13	197.75						
		12	12411		727	843	945
12	192.75						
		11	12411		727	843	945
11	187.75						
		10	12411		727	843	945
10	182.75						
		9	12411		727	843	945
9	177.75						
		8	12411		727	843	945
8	172.75						
		7	12411		727	843	945
7	167.75						
		6	12411		727	843	945
6	162.75						
		5	12411		727	843	945
5	158.88						
		4	12411		727	843	945
4	155.00						
		3	16482		965	1119	1255
3	150.15						
		2	11471		672	779	873
2	145.30						
		1	18682		1094	1269	1422
1	139.00						

Table 15
Gross Section - Shear Capacity
Stone-Mortar Joints

Sweetwater Main Dam Outlet Tower
Sweetwater Authority, Chula Vista, CA
475-year earthquake

PGA (H)	0.21	g
PGA (V) =	0.15	g

Rw = 1.0

Strength reduction factor for shear =	**0.85**
Small capacity increase from vertical load is ignored	
Stress increase factor for shape =	1.855
Mortar joint strength reduction factor:	**0.73**

				MORTAR-STONE JOINTS FACTORED GROSS CAPACITY [in Kips]			
NODE NUMBER	ELEV. (ft)	SECTION NUMBER	Shear Area (in**2)	f'c >>	2,417	3,250	4,083
				vcj >>	93	107	120
21	239.30						
		20	11810		501	582	652
20	235.75						
		19	11810		501	582	652
19	226.75						
		18	11810		501	582	652
18	217.75						
		17	11939		507	588	659
17	215.25						
		16	11939		507	588	659
16	212.75						
		15	12140		515	598	670
15	207.75						
		14	12140		515	598	670
14	202.75						
		13	12411		527	611	685
13	197.75						
		12	12411		527	611	685
12	192.75						
		11	12411		527	611	685
11	187.75						
		10	12411		527	611	685
10	182.75						
		9	12411		527	611	685
9	177.75						
		8	12411		527	611	685
8	172.75						
		7	12411		527	611	685
7	167.75						
		6	12411		527	611	685
6	162.75						
		5	12411		527	611	685
5	158.88						
		4	12411		527	611	685
4	155.00						
		3	16482		700	812	910
3	150.15						
		2	11471		487	565	633
2	145.30						
		1	18682		793	920	1031
1	139.00						

Table 16
Gross Response - Demand/Capacity (D/C) Ratios
SWEETWATER MAIN DAM OUTLET TOWER
SWEETWATER AUTHORITY, Chula Vista, CA

Reservoir water level: 237
Eqk load condition: La Nacion MCE

MOMENT LOADING

NODAL POINT	SECTION NUMBER	Elevation (ft)	INTACT MORTAR MORTAR STRENGTH (psi)			STONE-MORTAR JOINTS BOND STRENGTH (psi)		
			338	455	572	245	330	415
21		239.30	0.00	0.00	0.00	0.00	0.00	0.00
	20	237.53	0.02	0.02	0.02	0.03	0.03	0.02
20		235.75	0.04	0.04	0.03	0.06	0.05	0.04
	19	231.25	0.16	0.15	0.13	0.23	0.21	0.18
19		226.75	0.29	0.26	0.23	0.40	0.36	0.32
	18	222.25	0.47	0.45	0.40	0.64	0.62	0.55
18		217.75	0.65	0.64	0.57	0.89	0.88	0.78
	17	216.50	0.70	0.70	0.62	0.97	0.96	0.86
17		215.25	0.75	0.76	0.68	1.04	1.05	0.94
	16	214.00	0.81	0.82	0.74	1.11	1.13	1.02
16		212.75	0.85	0.88	0.79	1.18	1.21	1.09
	15	210.25	0.95	1.01	0.91	1.31	1.39	1.25
15		207.75	1.06	1.14	1.03	1.46	1.57	1.43
	14	205.25	1.16	1.29	1.17	1.61	1.78	1.62
14		202.75	1.26	1.42	1.30	1.74	1.96	1.79
	13	200.25	1.35	1.56	1.44	1.86	2.16	1.99
13		197.75	1.45	1.72	1.59	2.00	2.37	2.19
	12	195.25	1.55	1.88	1.75	2.13	2.60	2.41
12		192.75	1.64	2.05	1.90	2.27	2.82	2.63
	11	190.25	1.74	2.22	2.07	2.40	3.06	2.86
11		187.75	1.84	2.39	2.24	2.54	3.30	3.09
	10	185.25	1.94	2.57	2.41	2.68	3.55	3.33
10		182.75	2.04	2.75	2.59	2.82	3.79	3.57
	9	180.25	2.15	2.94	2.77	2.97	4.05	3.82
9		177.75	2.26	3.12	2.95	3.11	4.31	4.07
	8	175.25	2.37	3.32	3.14	3.28	4.57	4.33
8		172.75	2.49	3.51	3.32	3.44	4.84	4.58
	7	170.25	2.62	3.71	3.51	3.62	5.11	4.84
7		167.75	2.75	3.90	3.70	3.79	5.38	5.10
	6	165.25	2.89	4.11	3.89	3.99	5.66	5.37
6		162.75	3.04	4.31	4.09	4.19	5.95	5.64
	5	160.81	3.16	4.47	4.24	4.36	6.17	5.85
5		158.88	3.28	4.63	4.39	4.52	6.39	6.05
	4	156.94	3.41	4.79	4.54	4.70	6.61	6.26
4		155.00	1.65	2.31	2.19	2.27	3.19	3.02
	3	152.58	1.13	1.58	1.49	1.55	2.17	2.05
3		150.15	1.55	2.14	2.03	2.13	2.96	2.80
	2	147.73	2.34	3.21	3.04	3.23	4.43	4.20
2		145.30	1.69	2.31	2.18	2.33	3.18	3.01
	1	142.15	1.36	1.85	1.75	1.88	2.55	2.41
1		139.00	1.44	1.94	1.83	1.99	2.67	2.52

Table 17

Gross Response - Demand/Capacity (D/C) Ratios

SWEETWATER MAIN DAM OUTLET TOWER

SWEETWATER AUTHORITY, Chula Vista, CA

Reservoir water level: 237

Eqk load condition: Rose Canyon Earthquake

MOMENT LOADING

NODAL POINT	SECTION NUMBER	Elevation (ft)	INTACT MORTAR MORTAR STRENGTH (psi)			STONE-MORTAR JOINTS BOND STRENGTH (psi)		
			338	455	572	245	330	415
21		239.30	0.00	0.00	0.00	0.00	0.00	0.00
	20	237.53	0.01	0.01	0.01			0.01
20								0.02
	19							0.10
19								0.18
	18							0.31
18								0.45
	17							0.49
17								0.53
	16							0.58
16								0.62
	15							0.71
15								0.81
	14							0.92
14								1.02
	13							1.13
13								1.24
	12							1.37
12								1.49
	11							1.62
11		187.75	1.06	1.36	1.27			1.75
	10	185.25	1.12	1.46	1.37			1.89
10		182.75	1.18	1.56	1.47			2.02
	9	180.25	1.24	1.67	1.57			2.16
9		177.75	1.30	1.77	1.67			2.30
	8	175.25	1.37	1.88	1.78			2.45
8								2.59
	7							2.74
7								2.88
	6							3.03
6								3.18
	5							3.30
5		158.88	1.87	2.61	2.47	2.57		3.41
	4	156.94	1.93	2.69	2.56	2.67		3.53
4		155.00	0.94	1.31	1.24	1.30		1.71
	3							1.16
3		150.15	0.88	1.21	1.14	1.21	1.67	1.58
	2	147.73	1.30	1.79	1.70	1.79	2.47	2.35
2		145.30	0.95	1.30	1.23	1.32	1.79	1.69
	1							1.36
1								1.42

Table 18
Gross (uncracked) Moment Response
475-year earthquake
Sweetwater Main Dam Outlet Tower
Sweetwater Authority, Chula Vista, CA

Reservoir water level: 237 ft

BENDING MOMENT [Kips.in x 1000]

NODAL POINT	Elevation (ft)	SECTION NUMBER	Elevation (ft)	DYN. ELASTICITY MODULUS (psi)		
				1.25 E6	5.0 E6	8.75 E6
21	239.30			0	0	0
		20	237.53			
20	235.75			2	2	2
		19	231.25			
19	226.75			11	14	15
		18	222.25			
18	217.75			26	33	37
		17	216.50			
17	215.25			30	40	44
		16	214.00			
16	212.75			34	46	52
		15	210.25			
15	207.75			44	61	69
		14	205.25			
14	202.75			53	78	88
		13	200.25			
13	197.75			63	96	109
		12	195.25			
12	192.75			73	116	132
		11	190.25			
11	187.75			83	136	157
		10	185.25			
10	182.75			94	158	182
		9	180.25			
9	177.75			105	181	209
		8	175.25			
8	172.75			117	205	237
		7	170.25			
7	167.75			130	230	266
		6	165.25			
6	162.75			145	255	295
		5	160.81			
5	158.88			157	275	318
		4	156.94			
4	155.00			169	296	342
		3	152.58			
3	150.15			185	322	371
		2	147.73			
2	145.30			203	348	401
		1	142.15			
1	139.00			226	383	441

Table 19
Gross Response - Demand/Capacity (D/C) Ratios
Sweetwater Main Dam Outlet Tower
Sweetwater Authority, Chula Vista, CA

Reservoir water level: 237
Eqk load condition: 475-year earthquake

MOMENT LOADING

NODAL POINT	SECTION NUMBER	Elevation (ft)	INTACT MORTAR MORTAR STRENGTH (psi)			STONE-MORTAR JOINTS BOND STRENGTH (psi)		
			338	455	572	245	330	415
21		239.30	0.00	0.00	0.00	0.00	0.00	0.00
	20	237.53	0.01	0.01	0.01	0.01	0.01	0.01
20		235.75	0.02	0.02	0.01	0.03	0.02	0.02
	19	231.25	0.08	0.07	0.06	0.10	0.09	0.08
19		226.75	0.13	0.12	0.10	0.18	0.17	0.14
	18	222.25	0.21	0.21	0.18	0.29	0.28	0.25
18		217.75	0.30	0.29	0.26	0.41	0.40	0.36
	17	216.50	0.32	0.32	0.28	0.44	0.44	0.39
17		215.25	0.35	0.34	0.31	0.48	0.48	0.42
	16	214.00	0.37	0.37	0.33	0.51	0.52	0.46
16		212.75	0.39	0.40	0.36	0.54	0.55	0.49
	15	210.25	0.44	0.46	0.41	0.60	0.63	0.57
15		207.75	0.49	0.52	0.47	0.67	0.71	0.65
	14	205.25	0.54	0.58	0.53	0.74	0.81	0.73
14		202.75	0.58	0.65	0.59	0.80	0.89	0.81
	13	200.25	0.62	0.71	0.65	0.86	0.98	0.90
13		197.75	0.67	0.78	0.72	0.93	1.08	0.99
	12	195.25	0.72	0.86	0.79	0.99	1.18	1.09
12		192.75	0.77	0.93	0.86	1.06	1.28	1.19
	11	190.25	0.82	1.01	0.94	1.13	1.39	1.29
11		187.75	0.86	1.08	1.01	1.19	1.50	1.39
	10	185.25	0.91	1.17	1.09	1.26	1.61	1.50
10		182.75	0.96	1.25	1.17	1.33	1.72	1.61
	9	180.25	1.02	1.33	1.25	1.40	1.83	1.72
9		177.75	1.07	1.41	1.33	1.47	1.95	1.83
	8	175.25	1.12	1.50	1.41	1.55	2.07	1.94
8		172.75	1.18	1.58	1.49	1.62	2.18	2.06
	7	170.25	1.24	1.67	1.58	1.70	2.30	2.17
7		167.75	1.29	1.76	1.66	1.79	2.42	2.29
	6	165.25	1.36	1.84	1.74	1.87	2.54	2.40
6		162.75	1.42	1.93	1.83	1.96	2.67	2.52
	5	160.81	1.47	2.00	1.90	2.03	2.76	2.61
5		158.88	1.53	2.07	1.96	2.10	2.86	2.70
	4	156.94	1.58	2.14	2.03	2.18	2.95	2.80
4		155.00	0.77	1.04	0.98	1.07	1.44	1.35
	3	152.58	0.53	0.71	0.67	0.73	0.98	0.92
3		150.15	0.72	0.96	0.91	0.99	1.32	1.25
	2	147.73	1.06	1.42	1.35	1.46	1.95	1.86
2		145.30	0.78	1.03	0.97	1.07	1.42	1.34
	1	142.15	0.64	0.83	0.78	0.88	1.15	1.08
1		139.00	0.67	0.87	0.82	0.92	1.20	1.13

Table 20
Gross (Uncracked) Shear Response
475-year earthquake
Sweetwater Main Dam Outlet Tower
Sweetwater Authority, Chula Vista, CA

Reservoir water level: 237 ft

SHEAR FORCE [Kips]

NODAL POINT	Elevation (ft)	SECTION NUMBER	Elevation (ft)	DYN. ELASTICITY MODULUS (psi)		
				1.25 E6	5.0 E6	8.75 E6
21	239.30			0	0	0
		20	237.53			
20	235.75			37	43	47
		19	231.25			
19	226.75			88	108	118
		18	222.25			
18	217.75			135	183	204
		17	216.50			
17	215.25			149	216	243
		16	214.00			
16	212.75			154	231	262
		15	210.25			
15	207.75			163	255	291
		14	205.25			
14	202.75			173	284	327
		13	200.25			
13	197.75			183	313	361
		12	195.25			
12	192.75			196	339	392
		11	190.25			
11	187.75			212	364	420
		10	185.25			
10	182.75			230	386	444
		9	180.25			
9	177.75			250	407	466
		8	175.25			
8	172.75			270	425	485
		7	170.25			
7	167.75			291	441	500
		6	165.25			
6	162.75			308	454	512
		5	160.81			
5	158.88			322	463	521
		4	156.94			
4	155.00			332	469	526
		3	152.58			
3	150.15			341	475	532
		2	147.73			
2	145.30			348	479	535
		1	142.15			
1	139.00			352	482	538

Table 21

Gross Response - Demand/Capacity (D/C) Ratios

Sweetwater Main Dam Outlet Tower

Sweetwater Authority, Chula Vista, CA

Reservoir water level: 237

Eqk load condition: 475-year earthquake

SHEAR LOADING

NODAL POINT	SECTION NUMBER	Elevation (ft)	INTACT MORTAR MORTAR STRENGTH (psi)			STONE-MORTAR JOINTS BOND STRENGTH (psi)		
			128	148	166	93	197	120
21		239.30	0.00	0.00	0.00	0.00	0.00	0.00
	20	237.53	0.03	0.03	0.03	0.04	0.04	0.04
20		235.75	0.05	0.05	0.05	0.07	0.07	0.07
	19	231.25	0.09	0.09	0.09	0.12	0.13	0.13
19		226.75	0.13	0.14	0.13	0.18	0.19	0.18
	18	222.25	0.16	0.18	0.18	0.22	0.25	0.25
18		217.75	0.19	0.23	0.23	0.27	0.31	0.31
	17	216.50	0.20	0.25	0.25	0.28	0.34	0.34
17		215.25	0.21	0.27	0.27	0.29	0.37	0.37
	16	214.00	0.22	0.28	0.28	0.30	0.38	0.38
16		212.75	0.22	0.28	0.29	0.30	0.39	0.39
	15	210.25	0.22	0.29	0.30	0.31	0.41	0.41
15		207.75	0.23	0.31	0.31	0.32	0.43	0.43
	14	205.25	0.24	0.33	0.33	0.33	0.45	0.46
14		202.75	0.24	0.34	0.35	0.33	0.47	0.48
	13	200.25	0.25	0.35	0.36	0.34	0.49	0.50
13		197.75	0.25	0.37	0.38	0.35	0.51	0.53
	12	195.25	0.26	0.39	0.40	0.36	0.53	0.55
12		192.75	0.27	0.40	0.41	0.37	0.56	0.57
	11	190.25	0.28	0.42	0.43	0.39	0.58	0.59
11		187.75	0.29	0.43	0.44	0.40	0.60	0.61
	10	185.25	0.30	0.44	0.46	0.42	0.61	0.63
10		182.75	0.32	0.46	0.47	0.44	0.63	0.65
	9	180.25	0.33	0.47	0.48	0.46	0.65	0.66
9		177.75	0.34	0.48	0.49	0.47	0.67	0.68
	8	175.25	0.36	0.49	0.50	0.49	0.68	0.69
8		172.75	0.37	0.50	0.51	0.51	0.70	0.71
	7	170.25	0.39	0.51	0.52	0.53	0.71	0.72
7		167.75	0.40	0.52	0.53	0.55	0.72	0.73
	6	165.25	0.41	0.53	0.54	0.57	0.73	0.74
6		162.75	0.42	0.54	0.54	0.59	0.74	0.75
	5	160.81	0.43	0.54	0.55	0.60	0.75	0.75
5		158.88	0.44	0.55	0.55	0.61	0.76	0.76
	4	156.94	0.45	0.55	0.55	0.62	0.76	0.76
4		155.00	0.39	0.48	0.48	0.54	0.66	0.66
	3	152.58	0.35	0.42	0.42	0.48	0.58	0.58
3		150.15	0.42	0.50	0.50	0.57	0.69	0.69
	2	147.73	0.51	0.61	0.61	0.71	0.85	0.84
2		145.30	0.39	0.47	0.47	0.54	0.65	0.64
	1	142.15	0.32	0.38	0.38	0.44	0.52	0.52
1		139.00	0.32	0.38	0.38	0.44	0.52	0.52

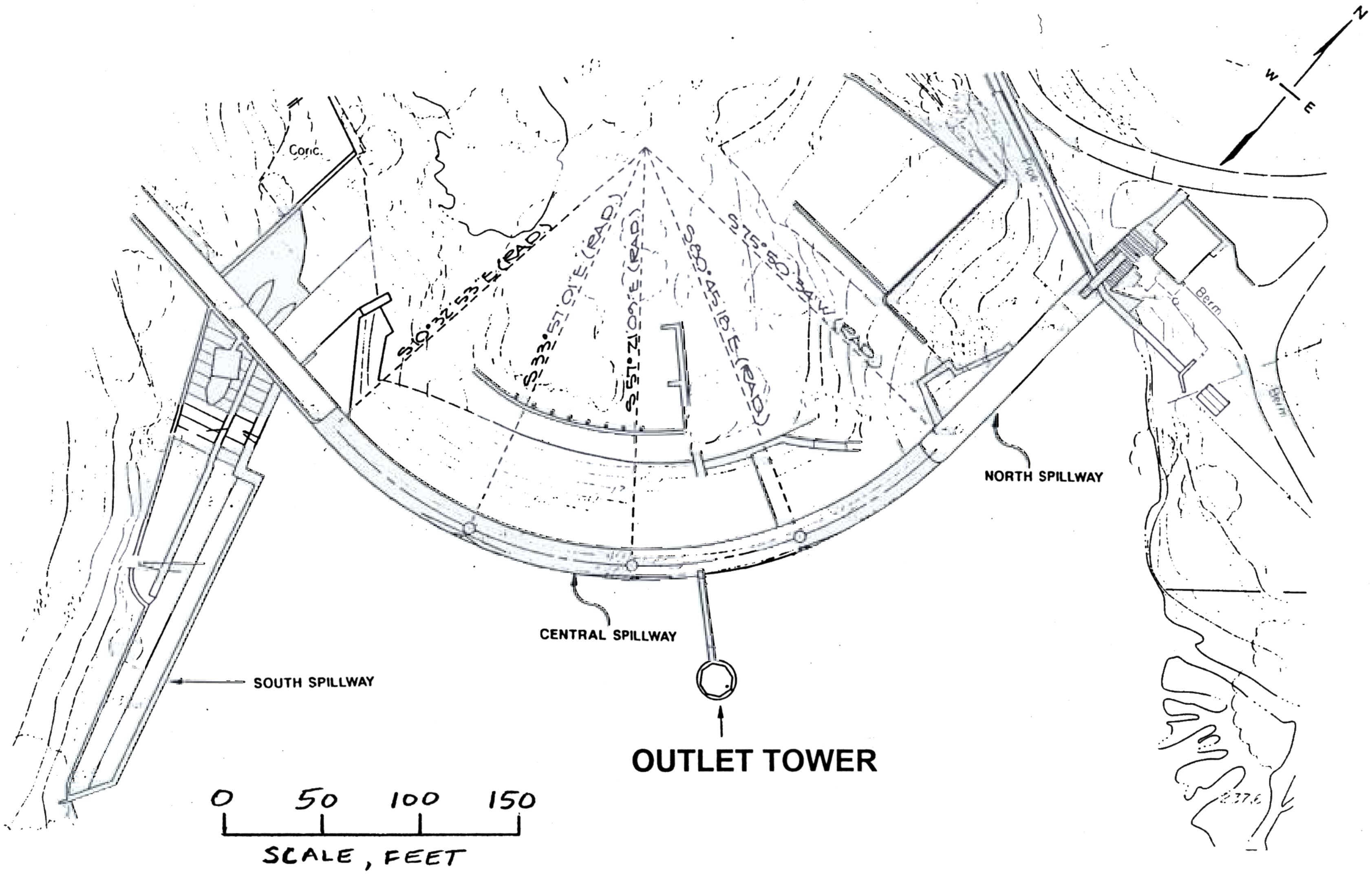
Table 22

Outlet Conduit – Moment Loading D/C ratios

SWEETWATER MAIN DAM OUTLET TOWER, CA

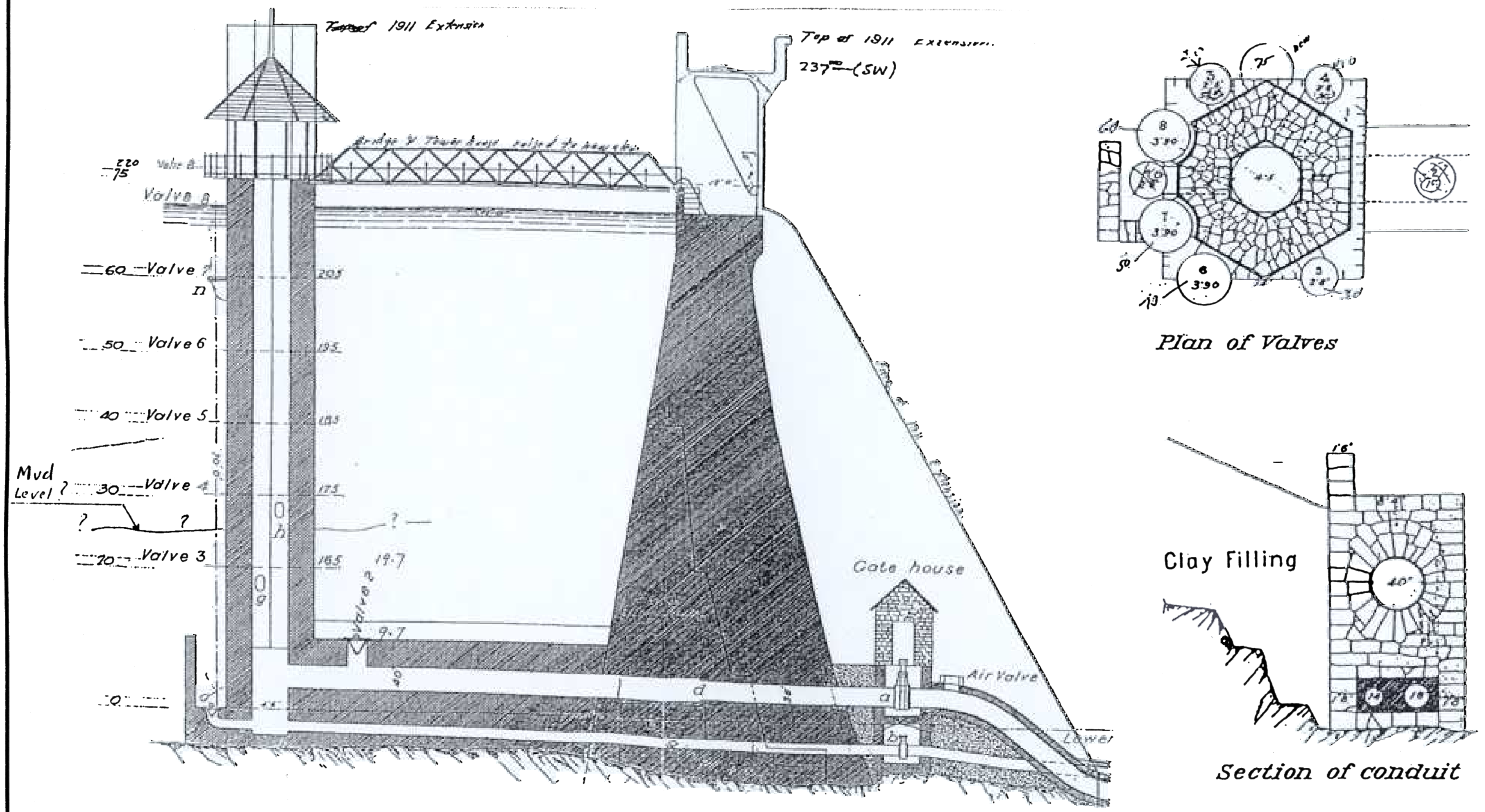
Bond Strength at Foundation Level (psi)	Demand/Capacity Ratio			
	La Nacion Earthquake [MCE]	Rose Canyon Earthquake	475-year Earthquake	72-year Earthquake
50.0	1.73	0.97	0.72	0.20
122.5	0.79	0.44	0.33	0.09
165.0	0.59	0.33	0.25	0.07
207.5	0.48	0.27	0.20	0.06

Figures

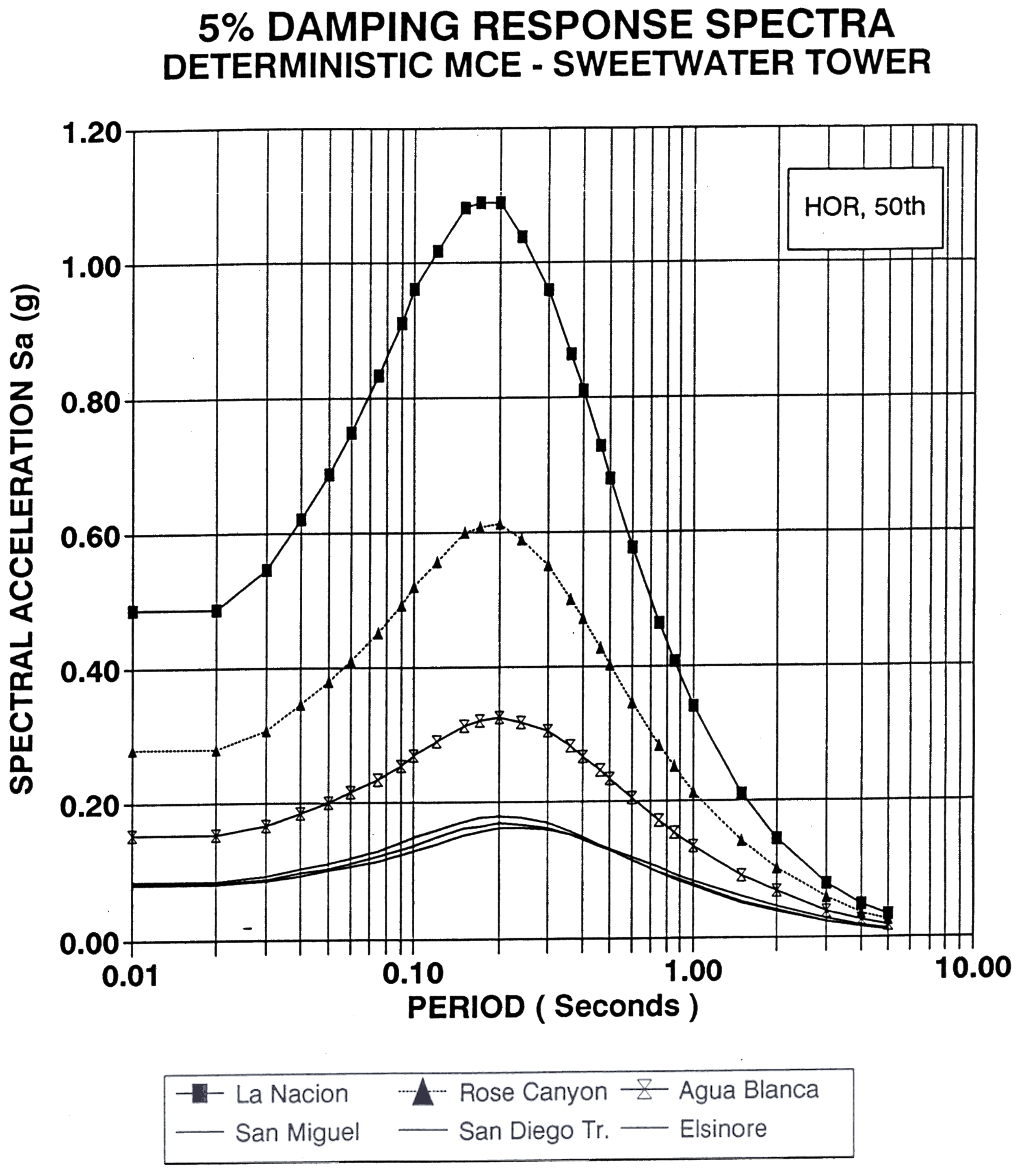


REFERENCE:
REVISED BASED ON THE ORIGINAL PLAN FROM
PHILLIPS - REYNOLDS ENGINEERING, INC.
OCTOBER 16, 1984

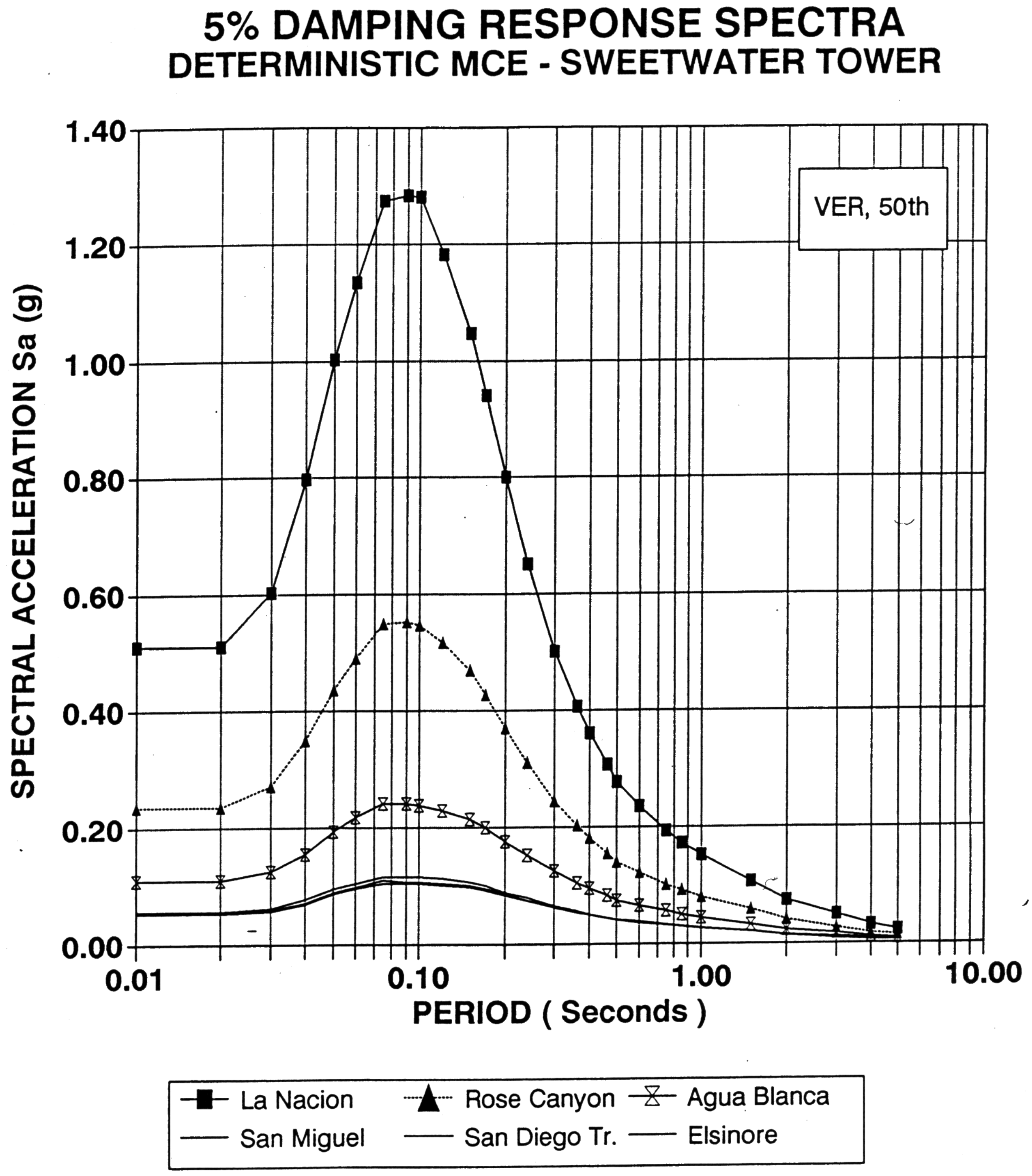
Sweetwater Authority Chula Vista, CA	Sweetwater Dam Outlet Tower California	**PLAN VIEW OF SWEETWATER DAM AND OUTLET TOWER**
GEI Consultants, Inc. Gilles Bureau	Project 022560	December 2002 Figure 1



Sweetwater Authority Chula Vista, CA	Sweetwater Dam Outlet Tower California	**OUTLET TOWER & CONDUIT DRAWINGS**
GEI Consultants, Inc. Gilles Bureau	Project 022560	December 2002 Figure 2

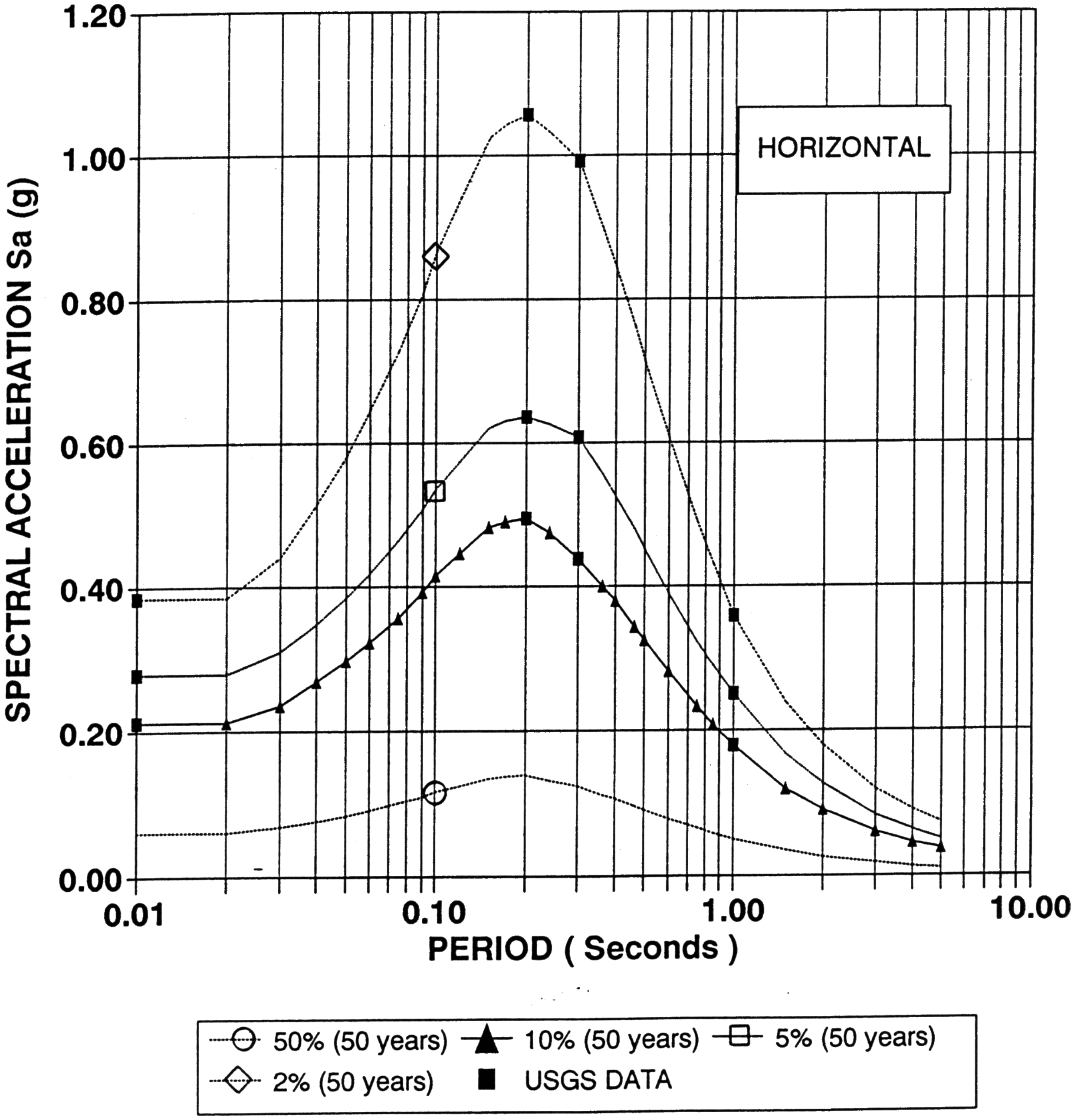


Sweetwater Authority Chula Vista, CA	Sweetwater Dam Outlet Tower California	HORIZONTAL RESPONSE SPECTRA - DETERMINISTIC EARTHQUAKE SCENARIOS
GEI Consultants, Inc. Gilles Bureau	Project 022560	December 2002 — Figure 3



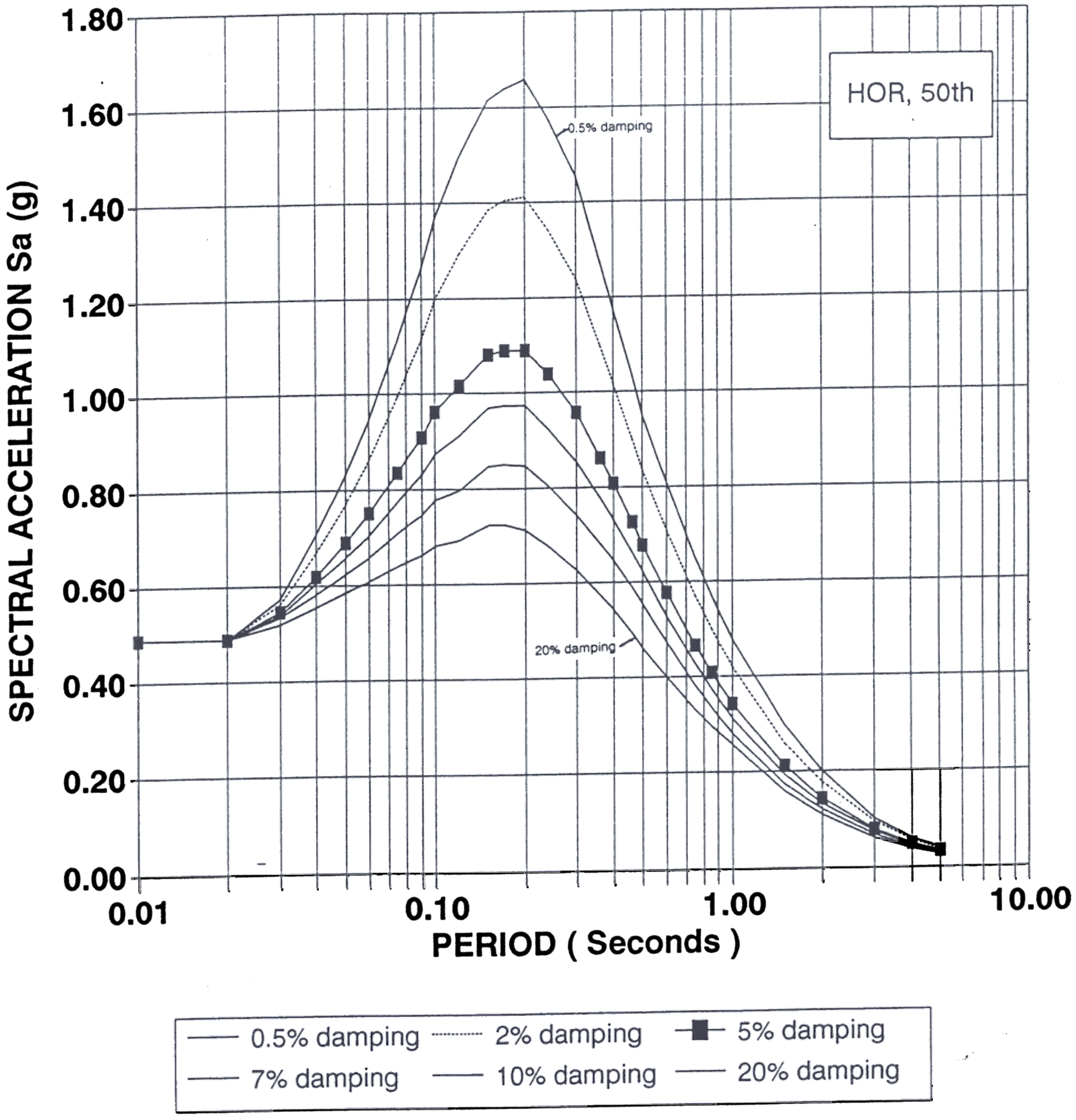
Sweetwater Authority Chula Vista, CA	Sweetwater Dam Outlet Tower California	**VERTICAL RESPONSE SPECTRA - DETERMINISTIC EARTHQUAKE SCENARIOS**
GEI Consultants, Inc. Gilles Bureau	Project 022560	December 2002 / Figure 4

5% DAMPING RESPONSE SPECTRA
USGS PROBABILISTIC - SWEETWATER TOWER

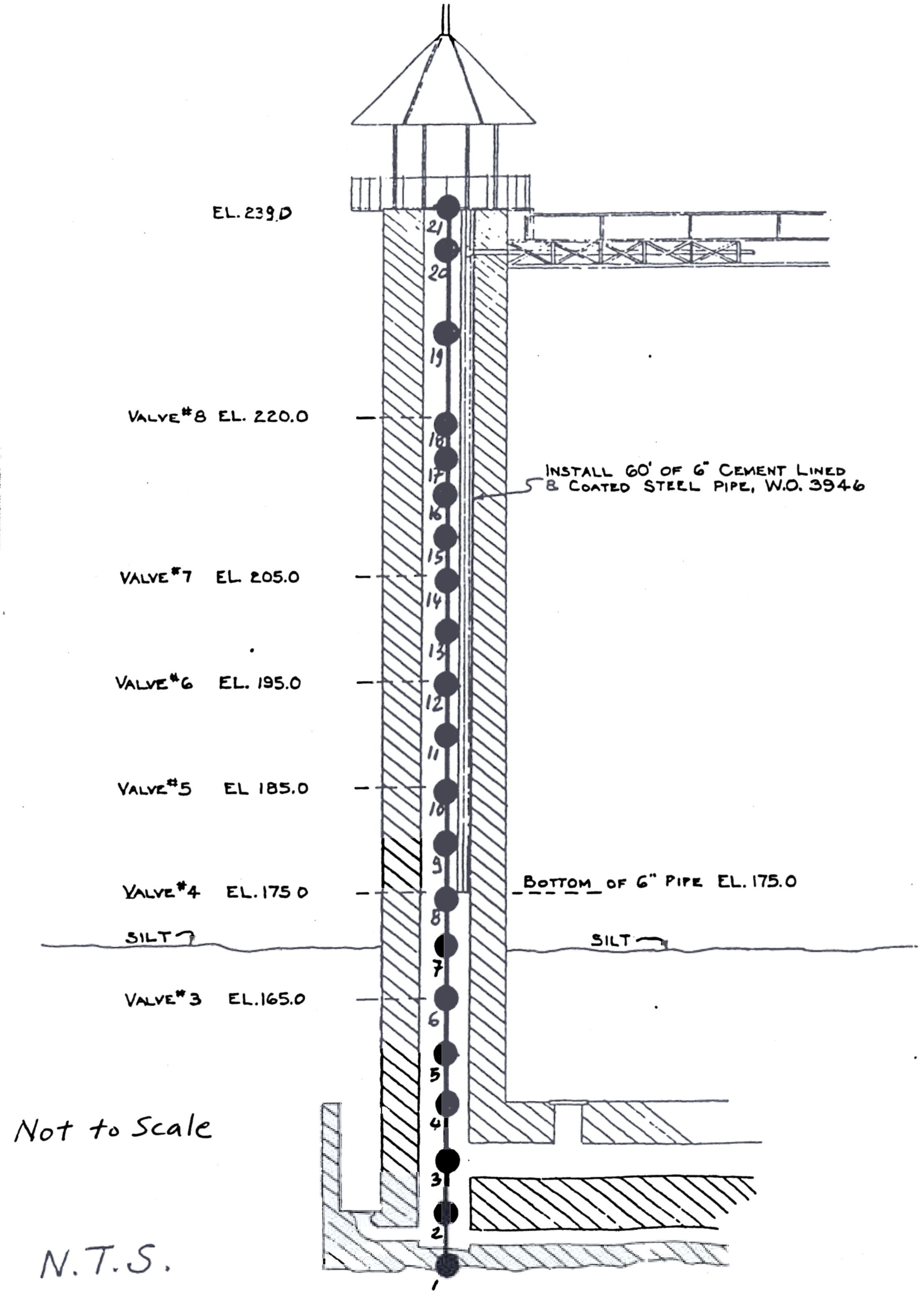


Sweetwater Authority Chula Vista, CA	Sweetwater Dam Outlet Tower California	HORIZONTAL RESPONSE SPECTRA - PROBABILISTIC EARTHQUAKE SCENARIOS
GEI Consultants, Inc. Gilles Bureau	Project 022560	December 2002 — Figure 5

LA NACION FAULT RESPONSE SPECTRA
DETERMINISTIC MCE - SWEETWATER TOWER

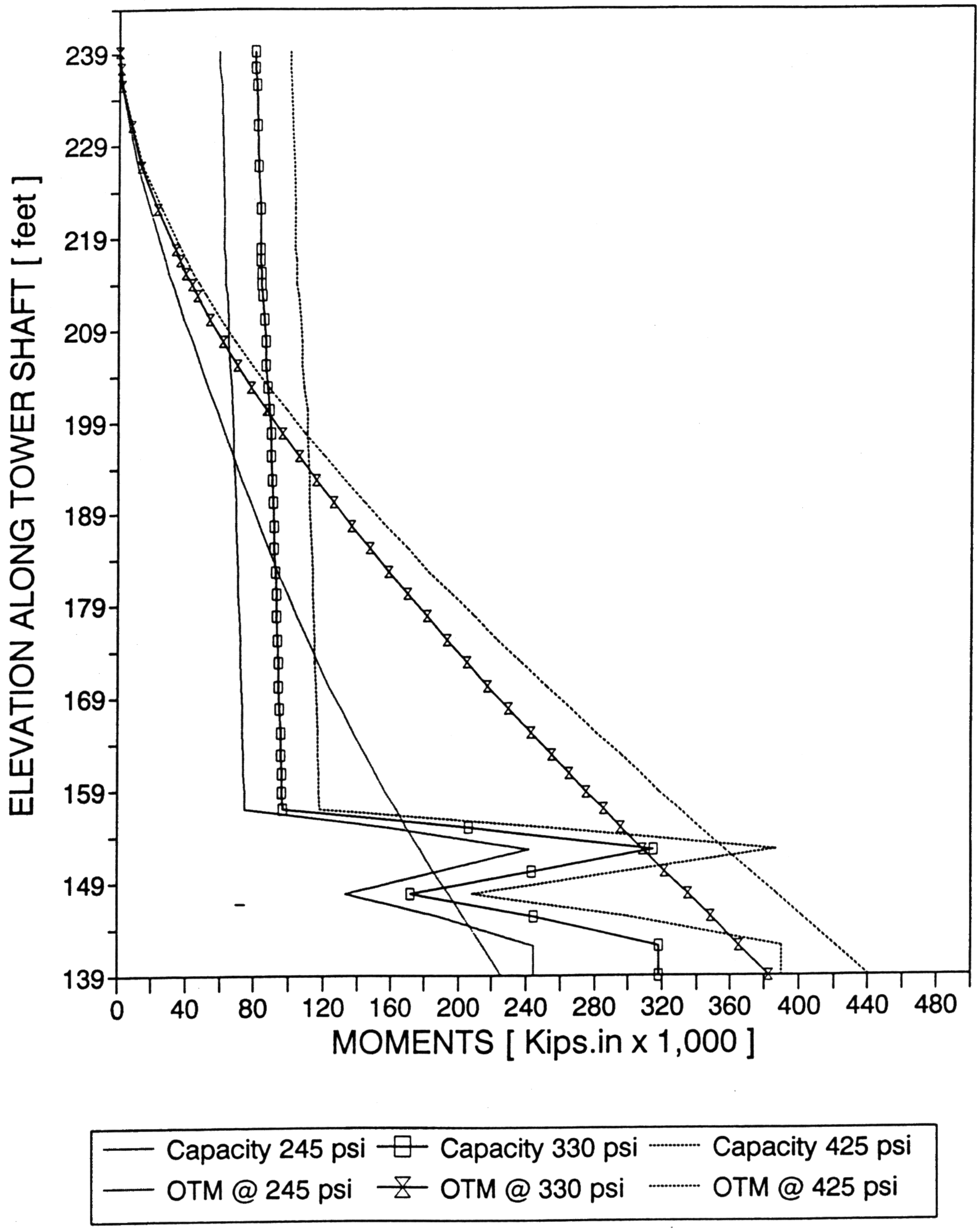


Sweetwater Authority Chula Vista, CA	Sweetwater Dam Outlet Tower California	HORIZONTAL RESPONSE SPECTRA - DETERMINISTIC MCE ON LA NACION FAULT
GEI Consultants, Inc. Gilles Bureau	Project 022560	December 2002 — Figure 6

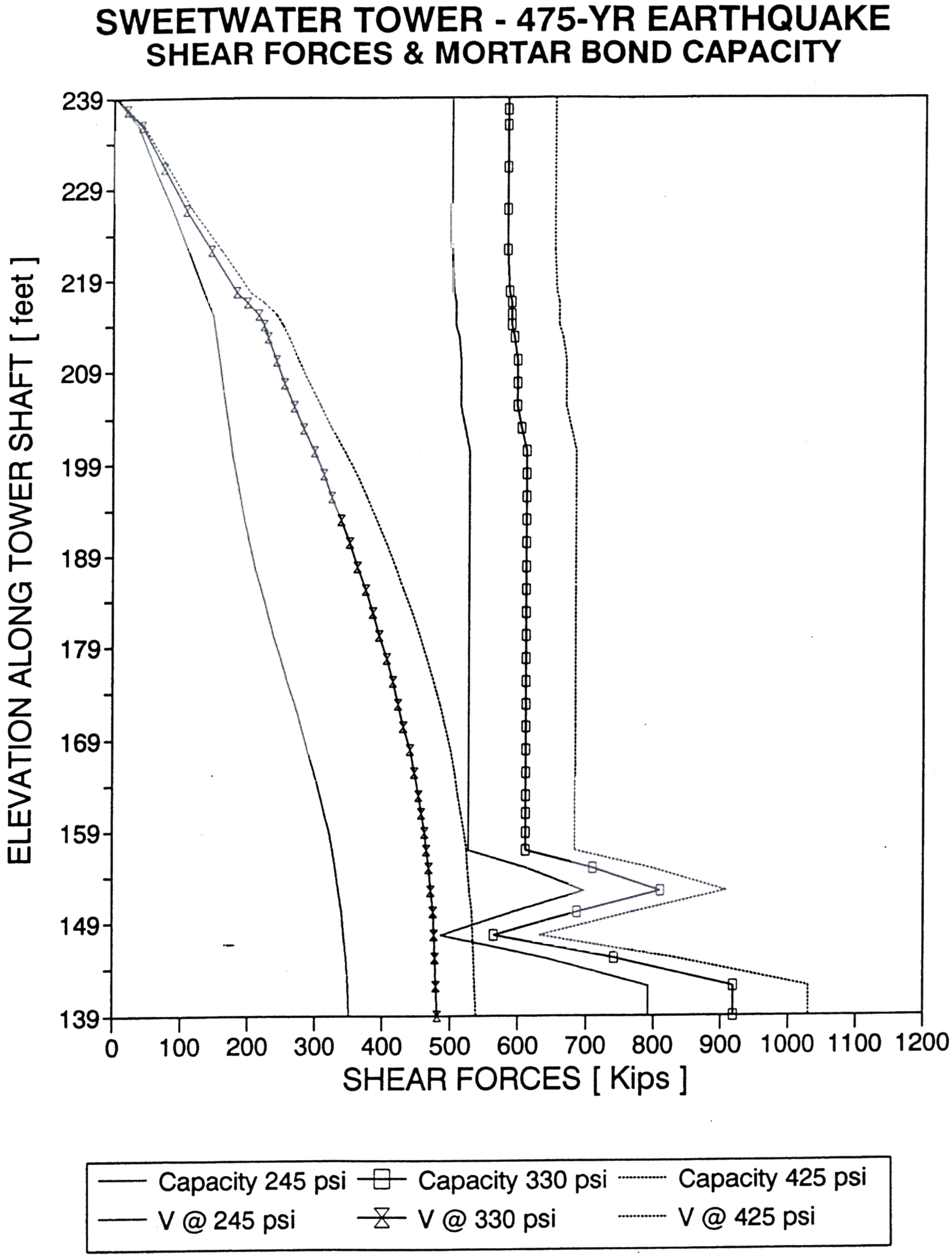


Sweetwater Authority Chula Vista, CA	Sweetwater Dam Outlet Tower California	ONE-DIMENSIONAL MODEL OF TOWER WITH NODE LOCATIONS
GEI Consultants, Inc. Gilles Bureau	Project 022560	December 2002 — Figure 7

SWEETWATER TOWER - 475-YR EARTHQUAKE
GROSS OT MOMENT & MORTAR BOND CAPACITY



Sweetwater Authority Chula Vista, CA GEI Consultants, Inc. Gilles Bureau	Sweetwater Dam Outlet Tower California	**OVERTURNING MOMENT AND CAPACITY VS. ELEVATION - 475 YEAR EARTHQUAKE**
	Project 022560	December 2002 / Figure 8



Sweetwater Authority Chula Vista, CA	Sweetwater Dam Outlet Tower California	**SHEAR FORCE & MORTAR BOND CAPACITY VS. ELEVATION - 475 YEAR EARTHQUAKE**
GEI Consultants, Inc. Gilles Bureau	Project 022560	December 2002 / Figure 9

Appendix A - Photographs

Photographs of Sweetwater Main Dam and Outlet Tower

Photo 1 – Upstream face of dam, and outlet tower, as viewed from left rim of reservoir (taken June 3, 2002).

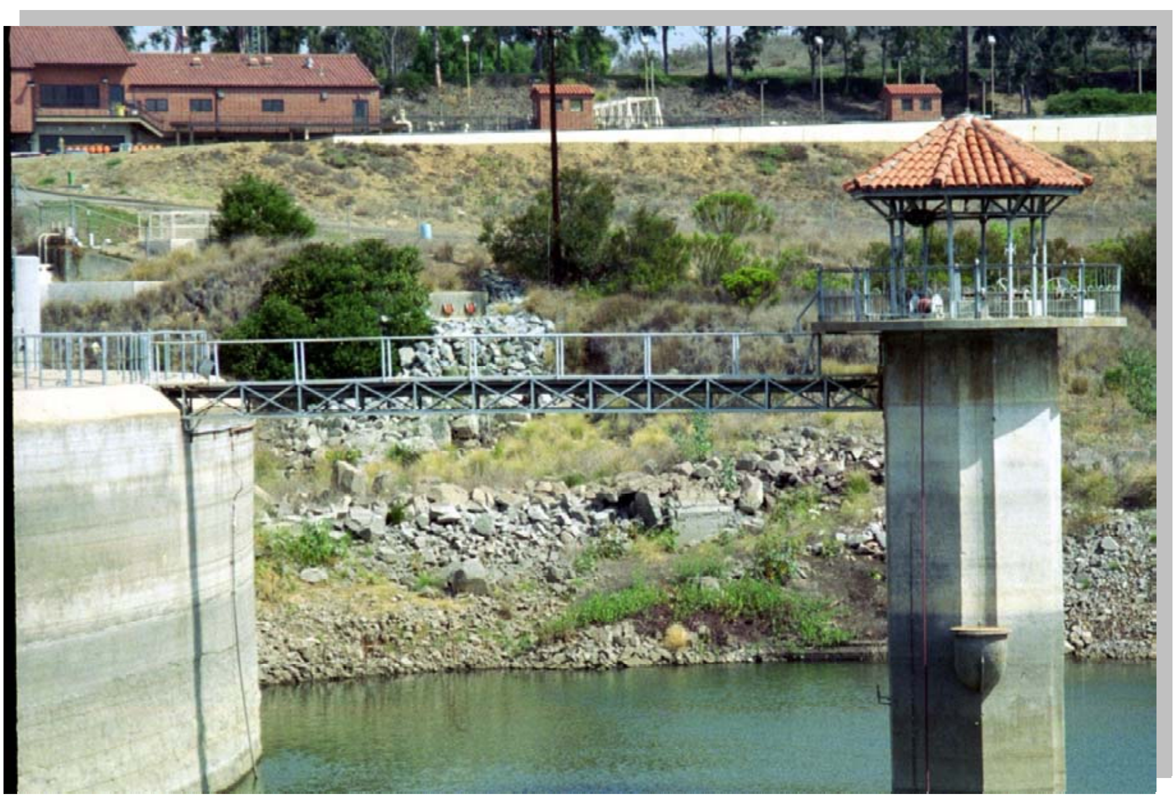


Photo 2 – Outlet tower and bridge as viewed from left abutment (taken June 3, 2002).

Photo 3 – Outlet tower as viewed from left abutment (taken June 3, 2002).

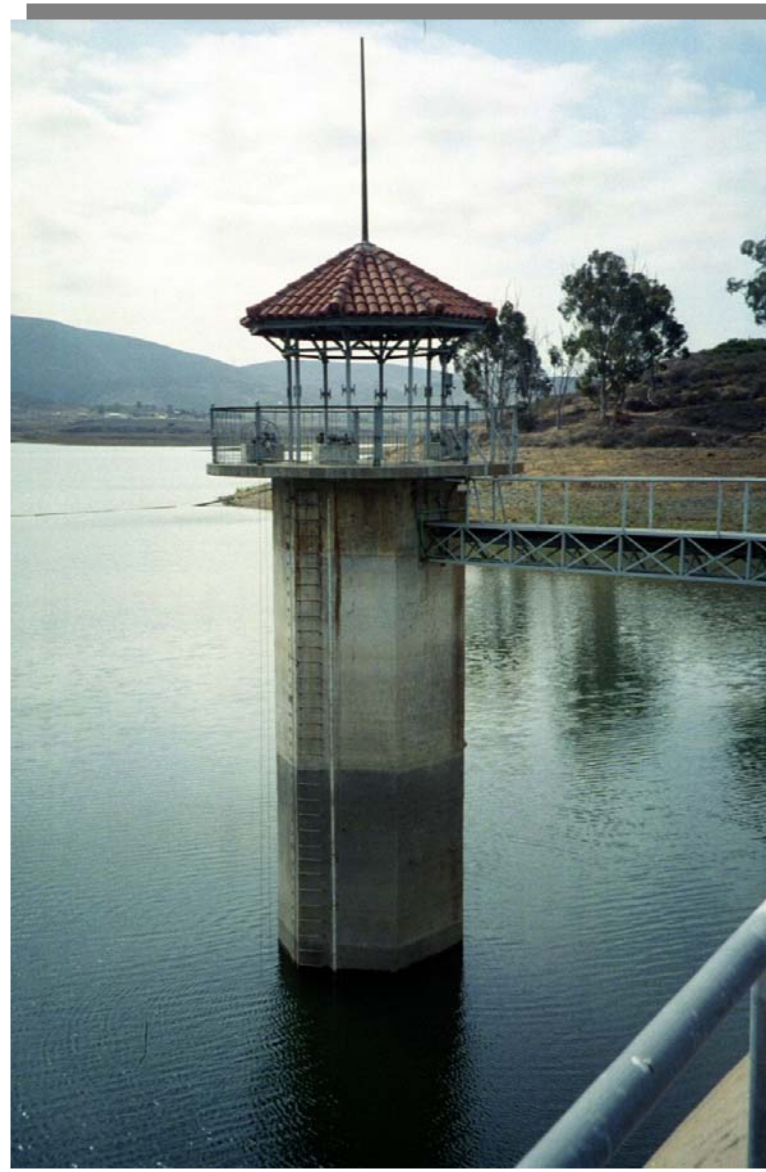


Photo 4 – Oulet tower as viewed from right side of dam crest (taken June 3, 2002).

Photo 5 – Stone masonry on downstream side of dam, on right side of south spillway discharge channel (taken June 3, 2002).

Photo 6 – Close-up of stone masonry shown in Photo 5, showing stones and mortar between stones (taken June 3, 2002).

EXHIBIT B

STANDARD AGREEMENT FOR SERVICES TEMPLATE

AGREEMENT FOR SERVICES
BETWEEN SWEETWATER AUTHORITY
AND
[*CONSULTANT NAME*]

This Agreement is made and entered into this __ day of ______________ 20__ by and between SWEETWATER AUTHORITY (hereinafter referred to as the “Authority”), a joint powers agency operating under the Irrigation District Law, Water Code § 20500 et seq., and **[*CONSULTANT NAME*]** (hereinafter referred to as “Consultant”).

RECITALS

A. The Authority is a public agency of the State of California and is in need of professional services for the following project: **[*PROJECT NAME*]** (hereinafter referred to as “the Project”).

B. Consultant is duly licensed and has the necessary qualifications to provide such services.

C. The parties desire by this Agreement to establish the terms for the Authority to retain Consultant to provide the services described herein.

AGREEMENT

NOW, THEREFORE, IT IS AGREED AS FOLLOWS:

1. Services

1.1 Consultant shall provide the Authority with the services described in the Scope of Services attached hereto as Exhibit “A” and by this reference incorporated herein (“Services”). Consultant warrants that it will perform the Services as set forth herein in a competent, professional and satisfactory manner.

1.2 At any time during the term of this Agreement, the Authority may request changes in the Scope of Services, and any such change shall be processed by the Authority in the following manner: a letter outlining the changes shall be forwarded to the Authority by Consultant with a statement of estimated changes in fee or time schedule. An amendment to the Agreement shall be prepared by the Authority and executed by both parties before performance of such services or the Authority will not be required to pay for the changes in the scope of work. Such amendment shall not render ineffective or invalidate unaffected portions of this Agreement.

2. Compensation

2.1 Subject to paragraph 2.2 below, the Authority shall pay for such Services in accordance with the Schedule of Charges set forth in Exhibit “B” and by this reference incorporated herein.

2.2 Unless otherwise provide herein, Consultant will perform services on a time and material basis. In no event shall the total amount paid for services rendered by Consultant pursuant to Exhibit “A” exceed the sum of $ **[*AMOUNT*]**. Periodic payments shall be made within thirty (30) days of receipt of an undisputed statement for services rendered. Payments to Consultant for work performed will be made on a monthly billing basis.

2.3 Payment shall not constitute acceptance of any work completed by Consultant.

3. Time of Performance

3.1 Consultant shall perform its services hereunder in a prompt and timely manner, in accordance with the Activity Schedule shown in Exhibit "C," and shall commence performance upon receipt of the written Notice to Proceed from the Authority. The Notice to Proceed shall set forth the date of commencement of work. Consultant shall confer as requested with Authority representatives to review progress of work elements, adherence to work schedule, coordination of work, scheduling of review and resolution of problems which may develop.

3.2 Neither the Authority nor Consultant shall be considered in default of this Agreement for delays in performance caused by circumstances beyond the reasonable control of the non-performing party. For purposes of this Agreement, such circumstances include, but are not limited to, abnormal weather conditions, floods, earthquakes, fire, epidemics, war, riots, and other civil disturbances; strikes, lockouts, work slowdowns, and other labor disturbances, sabotage, or judicial restraint.

3.3 Should such circumstances occur, the non-performing party shall, within a reasonable time of being prevented from performing, give written notice to the other party describing the circumstances preventing continued performance and the efforts being made to resume performance of this Agreement.

4. California Labor Code Requirements

4.1 Consultant is aware of the requirements of California Labor Code Sections 1720 et seq and 1770 et seq., which require the payment of prevailing wage rates and the performance of other requirements on certain "public works" and "maintenance" projects. If the services are being performed as part of an applicable "public works" or "maintenance" project, as defined by the Prevailing Wage Laws, and if the total compensation is $1,000 or more, Consultant agrees to fully comply with such Prevailing Wage Laws, if applicable. Consultant shall defend, indemnify and hold the Authority, its elected officials, officers, employees and agents free and harmless from any claims, liabilities, costs, penalties or interest arising out of any failure or alleged failure to comply with the Prevailing Wage Laws. It shall be mandatory upon Consultant and all subconsultants to comply with all California Labor Code provisions, which include but are not limited to prevailing wages, employment of apprentices, hours of labor and debarment of contractors and subcontractors.

4.2 If the services are being performed as part of an applicable "public works" or "maintenance" project, in addition to the foregoing, then pursuant to Labor Code sections 1725.5 and 1771.1, Consultant and all subconsultants must be registered with the Department of Industrial Relations ("DIR"). Consultant shall maintain registration for the duration of the Project and require the same of any subconsultants. This Project may also be subject to compliance monitoring and enforcement by the DIR. It shall be Consultant's sole responsibility to comply with all applicable registration and labor compliance requirements, including the submission of payroll records directly to the DIR.

5. Standard of Care

Consultant's services will be performed in accordance with generally accepted professional practices and principles and in a manner consistent with the level of care and skill ordinarily exercised by members of the profession currently practicing under similar conditions.

6. Insurance

6.1 Minimum Insurance Requirements: Consultant shall procure and maintain for the duration of the contract and for a minimum of twenty-four (24) months following the date of the Project completion and acceptance by the Authority, insurance against claims for injuries or death to persons or damages to property which may arise from or in connection with the performance of the work hereunder and the results of that work by the Consultant, his agents, representatives, employees or sub-contractors.

6.2 Coverage: Coverage shall be at least as broad as the following:

6.2.1 Commercial General Liability (CGL): Insurance Services Office (ISO) Commercial General Liability Coverage (Occurrence Form CG 00 01) including products and completed operations, property damage, bodily injury, personal and advertising injury with limit of at least two million dollars ($2,000,000) per occurrence or the full per occurrence limits of the policies available, whichever is greater. If a general aggregate limit applies, either the general aggregate limit shall apply separately to this project/location (coverage as broad as the ISO CG 25 03, or ISO CG 25 04 endorsement provided to the Authority) or the general aggregate limit shall be at least twice the required occurrence limit or four million dollars ($4,000,000).

(a) **Required Provisions:** The General Liability policy must contain, or be endorsed to contain, the following provisions:

(i) **Additional Insured Status:** Authority, its directors, officers, employees, and authorized volunteers are to be given insured status (at least as broad as ISO Form CG 20 10 10 01), with respect to liability arising out of work or operations performed by or on behalf of the Consultant including materials, parts, or equipment furnished in connection with such work or operations.

(ii) **Primary Coverage:** For any claims related to this project, the Consultant's insurance coverage shall be primary at least as broad as ISO CG 20 01 04 13 as respects to the Authority, its directors, officers, employees and authorized volunteers. Any insurance or self-insurance maintained by the Authority its directors, officers, employees and authorized volunteers shall be excess of the Consultant's insurance and shall not contribute with it.

6.2.2 Automobile Liability: Insurance Services Office (ISO) Business Auto Coverage (Form CA 00 01), covering Symbol 1 (any auto) or if Consultant has no owned autos, Symbol 8 (hired) and 9 (non-owned) with limit of one million dollars ($1,000,000) for bodily injury and property damage each accident.

6.2.3 Workers' Compensation Insurance: As required by the State of California, with Statutory Limits, and Employer's Liability Insurance with limit of no less than $1,000,000 per accident for bodily injury or disease. By his/her signature hereunder, Consultant certifies that he/she is aware of the provisions of Section 3700 of the California Labor Code which require every employer to be insured against liability for workers' compensation or to undertake self-insurance in accordance with the provisions of that code, and he/she will comply with such provisions before commencing the performance of the work of this agreement.

(a) **Waiver of Subrogation:** The Workers' Compensation Policy shall be endorsed with a waiver of subrogation in the favor of the Authority for all work performed by Consultant, its employees, agents and sub-consultants. The Insurer(s) agree to waive all rights of subrogation against the Authority, its elected or appointed officers, officials, agents, authorized volunteers and employees for losses paid under the terms of the policy which arise from work performed by the Consultant; but this provision applies regardless of whether or not the Authority has received a Waiver of Subrogation from the insurer.

6.2.4 <u>Professional Liability (also known as Errors and Omissions)</u>: Insurance appropriate to the Consultant profession, with limits no less than $1,000,000 per occurrence or claim, and $2,000,000 policy aggregate.

(a) **If Claims Made Policies:**

(i) The Retroactive Date must be shown and must be before the date of the contract or the beginning of contract work.

(ii) Insurance must be maintained and evidence of insurance must be provided **for at least five (5) years after completion of the contract of work.**

(iii) If coverage is canceled or non-renewed, and not **replaced with another claims-made policy form with a Retroactive Date** prior to the contract effective date, the Consultant must purchase "extended reporting" coverage for a minimum of **five (5)** years after completion of contract work.

6.2.5 <u>Cyber Liability Insurance (Technology Professional Liability – Errors and Omissions)</u>: Limits not less than $2,000,000 per occurrence or claim, and $2,000,000 aggregate or the full per occurrence limits of the policies available, whichever is greater. Coverage shall be sufficiently broad to respond to the duties and obligations as is undertaken by Consultant in this Agreement and shall include, but not be limited to, claims involving infringement of intellectual property, including but not limited to infringement of copyright, trademark, trade dress, invasion of privacy violations, information theft, damage to or destruction of electronic information, release of private information, alteration of electronic information, extortion and network security. The policy shall provide coverage for breach response costs as well as regulatory fines and penalties as well as credit monitoring expenses with limits sufficient to respond to these obligations.

6.3 <u>Other Required Provisions:</u>

6.3.1 If the Consultant maintains broader coverage and/or higher limits than the minimums shown above, the Authority requires and shall be entitled to the broader coverage and/or higher limits maintained by the Consultant. Any available insurance proceeds in excess of the specified minimum limits of insurance and coverage shall be available to the Authority.

6.3.2 Policy limits shall not be less than the minimum limits described above. The limits of insurance required by this Agreement may be satisfied by a combination of primary, and umbrella or excess insurance. Each umbrella or excess policy shall follow the same provisions as the primary policy.

6.3.3 Any failure to comply with reporting or other provisions of the policies including breaches of warranties shall not affect coverage provided to the Authority its Board and each member of the Board, its officers, employees, agents, and the Authority's designated volunteers.

6.3.4 Consultant's insurance shall apply separately to each insured against whom claim is made or suit is brought, except with respect to the limits of the insurer's liability.

6.3.5 Each insurance policy required above shall provide that coverage shall not be canceled, except with notice to the Authority.

6.4 Deductibles and Self-Insured Retentions: Insurance deductibles or self-insured retentions must be declared to and approved by the Authority. The Authority may require the Consultant to provide proof of ability to pay losses and related investigations, claim administration, and defense expenses within the retention.

6.4.1 At the election of the Authority, Consultant shall either 1) reduce or eliminate such deductibles or self-insured retentions, or 2) procure a bond which guarantees payment of losses and related investigations, claims administration, and defense costs and expenses.

6.4.2 Policies containing any self-insured retention (SIR) provision shall provide or be endorsed to provide, that the self-insured retention may be satisfied by either the named insured or Authority.

6.5 Acceptability of Insurers: Any insurance carrier providing insurance coverage required by the Contract Documents shall be admitted to and authorized to do business in the State of California and maintain an agent for process within the state, unless waived, in writing, by the Authority Risk Manager. Carrier(s) shall have an A.M. Best rating of not less than an A: VII or better, or as otherwise approved by the Authority Risk Manager.

6.6 Verification of Coverage: Consultant shall furnish the Authority with certificates (Acord Form 25 or equivalent) and amendatory endorsements, declarations page(s) listing all policy endorsements or copies of the applicable policy language effecting coverage required by this Agreement. Blanket endorsements are accepted with language that states "as required by contract". All certificates and endorsements are to be received and approved by the Authority before work commences.

6.6.1 Such evidence shall include the following:

(a) Additional insured endorsements with primary & non-contributory wording for each policy providing General Liability coverage

(b) Workers' Compensation waiver of subrogation

6.6.2 All of the insurance shall be provided on policy forms and through companies satisfactory to the Authority. However, failure to obtain the required documents prior to the work beginning shall not waive the Consultant's obligation to provide them. The Authority reserves the right to obtain complete, certified copies of all required insurance policies, at any time.

6.7 Continuation of Coverage: Consultant shall, upon demand of the Authority deliver evidence of coverage showing continuation of coverage for not less than 24 months for all policies, and not less than (5) years for claims made policies, following the termination or completion of this Agreement. Consultant further waives all rights of subrogation under this agreement. When any of the required coverages expire during the term of this agreement, Consultant shall deliver the renewal certificate(s) including the general liability additional insured endorsement and evidence of waiver of rights of subrogation against the Authority to the Authority at least ten (10) days prior to the expiration date. Failure to continually satisfy the Insurance requirements is a material breach of contract.

6.8 Sub-Consultants: In the event that Consultant employs other consultants (sub-consultants) as part of the work covered by this agreement, it shall be Consultant's responsibility to require, verify and confirm that each sub-consultant meets the minimum insurance requirements specified above. Consultant shall, upon demand of the Authority, deliver to the Authority copies such policy or policies of insurance and the receipts for payment of premiums thereon.

6.9 The Authority reserves the right to modify these insurance requirements, including limits, based on the nature of the risk, prior experience, insurer, coverage or other circumstances.

7. Indemnification

7.1 To the fullest extent permitted by law, Consultant shall defend (with counsel of the Authority's choosing), indemnify and hold the Authority, its officials, officers, employees, volunteers, and agents free and harmless from any and all claims, demands, causes of action, costs, expenses, liability, loss, damage or injury of any kind, in law or equity, to property or persons, including wrongful death, in any manner arising out of, pertaining to, or incident to any acts, errors or omissions, or willful misconduct of Consultant, its officials, officers, employees, subcontractors, consultants or agents in connection with the performance of Consultant's Services, the Project or this Agreement, including without limitation the payment of all damages, expert witness fees and attorneys' fees and other related costs and expenses. Consultant's obligation to indemnify shall not be restricted to insurance proceeds, if any, received by Consultant, the Authority, its officials, officers, employees, agents, or volunteers.

7.2 To the extent required by Civil Code section 2782.8, which is fully incorporated herein, Consultant's obligations under the above indemnity shall be limited to claims that arise out of, pertain to, or relate to the negligence, recklessness, or willful misconduct of Consultant, but shall not otherwise be reduced. If Consultant's obligations to defend, indemnify, and/or hold harmless arise out of Consultant's performance as a "design professional" (as that term is defined under Civil Code section 2782.8), then upon Consultant obtaining a final adjudication that liability under a claim is caused by the comparative active negligence or willful misconduct of the Authority, Consultant's obligations shall be reduced in proportion to the established comparative liability of the Authority and shall not exceed Consultant's proportionate percentage of fault.

8. Termination or Abandonment

8.1 The Authority has the right to terminate or abandon any portion or all of the work under this Agreement by giving ten (10) calendar days written notice to Consultant. In such event, the Authority shall be immediately given title and possession to all original field notes, drawings and specifications, written reports, and other documents produced or developed for that portion of the work completed, and/or being abandoned. The Authority shall pay Consultant the reasonable value of services rendered for any portion of the work completed prior to termination. If said termination occurs prior to completion of any task for the Project for which a payment request has not been received, the charge for services performed during such task shall be the reasonable value of such services, based on an amount mutually agreed to by the Authority and Consultant of the portion of such task completed but not paid prior to said termination. The Authority shall not be liable for any costs other than the charges or portions thereof, which are specified herein. Consultant shall not be entitled to payment for unperformed services, and shall not be entitled to damages or compensation for termination of work.

8.2 Consultant may terminate its obligation to provide further services under this Agreement upon thirty (30) calendar days' written notice to the Authority only in the event of substantial failure by Authority to perform in accordance with the terms of this Agreement through no fault of Consultant.

9. Compliance with All Laws

9.1 Consultant shall comply with all applicable laws, ordinances, codes, and regulations of the federal, state, and local government.

9.2 Consultant will use its best professional efforts to interpret all applicable federal, state and local laws, rules and regulations with respect to access, including those of the Americans with Disabilities Act ("ADA"). All documents (including but not limited to plans, specifications, and other technical documents, if applicable) prepared by Consultant pursuant to this Agreement shall be compliant with all applicable requirements of the ADA.

9.3 Consultant shall assist the Authority in obtaining and maintaining all permits required by federal, state, and local regulatory agencies.

9.4 Consultant is responsible for all costs of clean up and/or removal of hazardous and toxic substances spilled as a result of its services or operations performed under this Agreement.

10. Organization

Consultant shall assign **"[*PM NAME*]"** as the Project Manager. The Project Manager shall not be removed from the Project or reassigned without the prior written consent of the Authority.

11. Maintenance of Records

Books, documents, papers, accounting records, and other evidence pertaining to costs incurred shall be maintained by Consultant and made available at all reasonable times during the

Agreement period and for four (4) years from the date of final payment under the Agreement for inspection by the Authority.

12. Job Site Responsibility

If the services covered by this Agreement involve a construction phase of the Project, the Authority agrees that in accordance with generally accepted construction practices, the construction contractor will be required to assume sole and complete responsibility for job site conditions during the course of construction of the Project, including safety of all persons and property, and that this requirement shall be made to apply continuously and not be limited to normal working hours. Consultant shall not have control over or charge of, and shall not be responsible for, construction means, methods, techniques, sequences, or procedures, as these are solely the responsibility of the construction contractor.

13. Assignment and Subconsultants

Consultant shall not assign, sublet, or transfer this Agreement or any rights under or interest in this Agreement without the written consent of the Authority, which may be withheld for any reason. Nothing contained herein shall prevent Consultant from employing independent associates, and subconsultants as Consultant may deem appropriate to assist in the performance of services hereunder.

14. Conflicts of Interest

Identify all existing and past financial relationships (including consulting agreements) between **[*CONSULTANT NAME*]** and members of the Authority's Governing Board, and entities for which said members are employed, or have an interest, both past and present.

15. General Provisions

15.1 Independent Consultant. Consultant is retained as an independent consultant and is not an employee of Authority. No employee or agent of Consultant shall become an employee of the Authority. The work to be performed shall be in accordance with the work described in Exhibit "A," subject to such directions and amendments from the Authority as herein provided.

15.2 Notice. All notices permitted or required under this Contract shall be given at the following address, or at such other address as the parties may provide in writing for this purpose:

Authority:	Consultant:
SWEETWATER AUTHORITY	**[*COMPANY*]**
505 Garrett Ave	[*ADDRESS*]
Chula Vista, CA 91910	
Attn: **[*MANAGER*]**	Attn: **[*CONTACT*]**
[*POSITION*]	[*POSITION*]

The parties may designate, in writing, other individuals to whom notice is to be given. Notices shall be deemed to be received upon personal delivery to the addresses above; if sent by overnight delivery, upon delivery as shown by delivery service records; if sent by facsimile, upon receipt as confirmed by the sending facsimile equipment; if by United States Postal Service, five days after deposit in the mail.

15.3 Severability. The unenforceability, invalidity or illegality of any provision(s) of this Agreement shall not render other provisions of this Agreement unenforceable, invalid or illegal.

15.4 Integration. This Agreement represents the entire understanding of the Authority and the Consultant as to those matters contained herein, and supersedes and cancels any prior oral or written understanding, promises, or representations with respect to those matters covered hereunder. This Agreement may not be modified or altered except in writing, signed by both parties hereto. This is an integrated Agreement.

15.5 Survival. All rights and obligations hereunder that by their nature are to continue after any expiration or termination of this Agreement, including, but not limited to, the indemnification obligations, shall survive any such expiration or termination.

15.6 Time is of the Essence. Time shall be of the essence as to all dates and times of performance contained in this Agreement.

15.7 Third Party Rights. Nothing in this Agreement shall be construed to give any rights or benefits to anyone other than the Authority and Consultant.

15.8 Disputes. If any disputes should arise between the Parties concerning the work to be done under this Agreement, the payments to be made, or the manner of accomplishment of the work, Consultant shall nevertheless proceed to perform the work as directed by the Authority pending settlement of the dispute.

15.9 Laws, Venue, and Attorneys' Fees. This Agreement shall be interpreted in accordance with the laws of the State of California. If any action is brought to interpret or enforce any term of this Agreement, the action shall be brought in a state or federal court situated in the County of San Diego, State of California. In the event of any such litigation between the parties, the prevailing party shall be entitled to recover all reasonable costs incurred, including reasonable attorney's fees, as determined by the court.

AGREEMENT FOR SERVICES
BETWEEN SWEETWATER AUTHORITY
AND
[*CONSULTANT NAME*]

IN WITNESS WHEREOF, the parties have executed this Agreement as of the date first written above.

SWEETWATER AUTHORITY		**[*CONSULTANT NAME*]**	
By:	______________________	By:	______________________ (Authorized Representative of Consultant)
Name:	Carlos Quintero	Name:	**[*NAME*]**
Title:	General Manager	Title:	**[*TITLE*]**
Dated:	______________________	Dated:	______________________

Approved as to form: (only required when contract template is modified)

____
Paula C. P. de Sousa
Legal Counsel
SWEETWATER AUTHORITY

AGREEMENT FOR SERVICES
BETWEEN SWEETWATER AUTHORITY
AND
[*CONSULTANT NAME*]

EXHIBIT "A"
SCOPE OF WORK

[*INSERT PROPOSED SCOPE OF WORK*]

**AGREEMENT FOR SERVICES
BETWEEN SWEETWATER AUTHORITY
AND
[*CONSULTANT NAME*]**

**EXHIBIT “B”
SCHEDULE OF CHARGES**

AGREEMENT FOR SERVICES
BETWEEN SWEETWATER AUTHORITY
AND
[*CONSULTANT NAME*]

EXHIBIT “C”
ACTIVITY SCHEDULE